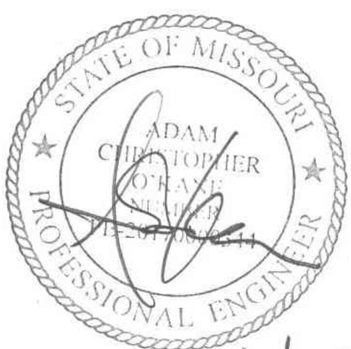
LEIGH & O'KANE L.L.C. – STRUCTURAL CALCULATIONS FOR

BAILEY FARMS CLUBHOUSE

LEE'S SUMMIT, MO



08/13/25

Adam C. O'Kane, P.E. & Reagan Holden, E.I.
8-13-2025

Leigh & O'Kane
209 SE Douglas
Lee's Summit, MO 64063

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LEIGH + O'KANE
209 SE DOUGLAS
LEE'S SUMMIT, MO 64063
816.444.3144
WWW.LEOK.COM

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GENERAL DESIGN CONSIDERATIONS

The calculations provided are for a new clubhouse structure for Bailey Farms in Lee's Summit, Missouri. The building is designed with prefabricated wood trusses, wood load bearing walls, and concrete trench footings. The lateral system is provided by wood framed shear walls. Fixed base steel columns are used to control the lateral force on the covered patio portion and support a custom timber truss.

REFERENCED DESIGN STANDARDS

- International Building Code 2018 Edition – IBC 2018
- AISC Steel Construction Manual – Fifteenth Edition
- Minimum Design Loads for Buildings and Other Structures – ASCE 7-16
- Building Code Requirements for Structural Concrete – ACI 318-14
- National Design Specification for Wood Construction – NDS 2018

DESIGN CRITERIA

Building Classification	
Risk Category	III
Snow Importance Factor, I_s	1.10
Ice Importance Factor – Wind, I_w	1.00
Seismic Importance Factor, I_e	1.25
Slab on Grade Floor Loads	
Live Load	100 psf
Roof Loads	
Dead Load Top Chord	10 psf
Dead Load Bottom Chord	10 psf
Roof Live Load Top Chord	20 psf
Roof Live Load Bottom Chord	0 psf
Snow Loads	
Ground Snow Load, P_g	20 paf
Flat Roof Snow Load, P_f	15.4 psf
Minimum Snow Load, P_m	20 psf
Thermal Factor, C_t	1.0
Slope Factor, C_s	1.0
Drift	Per Code
Wind Loads	
Basic Wind Speed	117 mph
Exposure Category	B
Internal Pressure Coefficient, GC_{pi}	+/- 0.18

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Seismic Loads

S_S	0.100
S_1	0.068
S_{DS}	0.106
S_{D1}	0.109
Site Class	D
Seismic Design Category	B
Seismic Analysis Procedure	Equivalent Lateral Force Procedure
Seismic Force Resisting System	Wood walls sheathed with wood structural panels rated for shear
Design Base Shear	C_sW
Design Response Coefficient, C_s	0.017
Response Modification Coefficient, R	6.5

Rain Loads

60-min Duration/100 Year Rain Intensity, i	3.91 in/hr
15-min Duration/100 Year Rain Intensity, i	8.30 in/hr

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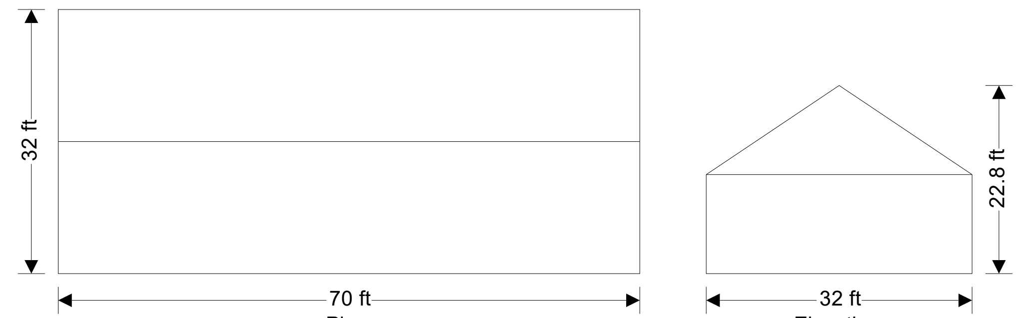
MAIN WIND FORCE RESISTING SYSTEM

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.14



Plan　　Elevation

Building data

Type of roof;	Gable
Length of building;	b = **70.00** ft
Width of building;	d = **32.00** ft
Height to eaves;	H = **12.00** ft
Pitch of roof;	α_0 = **34.0** deg
Mean height;	h = **17.40** ft

General wind load requirements

Basic wind speed;	V = **117.0** mph
Risk category;	III
Velocity pressure exponent coef (Table 26.6-1);	K_d = **0.85**
Ground elevation above sea level;	z_{gl} = **0** ft
Ground elevation factor;	K_e = exp(-0.0000362 ´ z_{gl}/1ft) = **1.00**
Exposure category (cl 26.7.3);	B
Enclosure classification (cl.26.12);	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1);	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1);	GC_{pi_n} = **-0.18**

Gust effect factor for rigid structures

Terrain exposure constants (Table 26.11-1)	
Integral length scale factor;	l = **320.0** ft
Turbulence intensity factor;	c = **0.30**
Minimum equivalent height;	z_{min} = **30.0** ft
Peak factor for background response;	g_Q = **3.400**
Peak factor for wind response;	g_v = **3.400**
Integral length scale power law exponent;	$\bar{\varepsilon}$ = **0.333**
Equivalent height of the structure;	$\bar{z}$ = max (0.6 ´ h, z_{min}) = **30.00** ft
Intensity of turbulence (Eqn. 26.11-7);	$I_{\bar{z}}$ = c ´ (33 ft / $\bar{z}$)$^{1/6}$ = **0.30**

Integral length scale of turbulence (Eqn. 26.11-9); $L_{\bar{z}} = l ´ (\bar{z} / 33 \text{ ft})^{\bar{\varepsilon}}$ = **310.00** ft

Background response (Eqn. 26.11-8); $Q = \sqrt{(1 / (1 + 0.63 ´ ((\min(B, L) + h) / L_{\bar{z}})^{0.63}))}$ = **0.914**

Gust effect factor (Eqn. 26.11-6); $G = G_f = 0.925 ´ (1 + 1.7 ´ g_Q ´ I_{\bar{z}} ´ Q) / (1 + 1.7 ´ g_v ´ I_{\bar{z}})$ = **0.87**

Minimum design wind loading (cl.27.1.5); p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant; $K_{zt} = 1.0$

Velocity pressure equation; $q = 0.00256 ´ K_z ´ K_{zt} ´ K_d ´ V^2 ´ 1\text{psf/mph}^2$;

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
12.00	0.57	16.98
15.00	0.57	16.98
15.00	0.57	16.98
17.40	0.59	17.69
22.79	0.64	19.13

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); q_i = **17.69** psf

Pressures and forces

Net pressure; $p = q ´ G_f ´ C_{pe} - q_i ´ GC_{pi}$;

Net force; $F_w = p ´ A_{ref}$;

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	17.40	-0.20	17.69	-6.30	1350.96	-8.52
B (-ve)	17.40	-0.60	17.69	-12.46	1350.96	-16.84

Total vertical net force; $F_{w,v}$ = **-21.02** kips

Total horizontal net force; $F_{w,h}$ = **4.65** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	16.98	8.69	840.00	7.30
B	17.40	-0.50	17.69	-10.92	840.00	-9.17
C	17.40	-0.70	17.69	-14.01	556.67	-7.80
D	17.40	-0.70	17.69	-14.01	556.67	-7.80

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b ´ H$ = **840.00** ft^2

Projected vertical area of roof; $A_{vert\_r\_0} = b ´ d/2 ´ \tan(\alpha_0)$ = **755.45** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} ´ A_{vert\_w\_0} + p_{min\_r} ´ A_{vert\_r\_0}$ = **19.48** kips

Leeward net force; $F_l = F_{w,wB}$ = **-9.2** kips

Windward net force; $F_w = F_{w,wA}$ = **7.3** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **21.1** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	17.40	0.27	17.69	7.41	1350.96	10.01
B (+ve)	17.40	-0.60	17.69	-6.09	1350.96	-8.23

Total vertical net force; $F_{w,v}$ = **1.47** kips
Total horizontal net force; $F_{w,h}$ = **10.20** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	12.00	0.80	16.98	15.06	840.00	12.65
B	17.40	-0.50	17.69	-4.55	840.00	-3.82
C	17.40	-0.70	17.69	-7.64	556.67	-4.25
D	17.40	-0.70	17.69	-7.64	556.67	-4.25

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0}$ = b ´ H = **840.00** ft^2
Projected vertical area of roof; $A_{vert\_r\_0}$ = b ´ d/2 ´ tan(α_0) = **755.45** ft^2
Minimum overall horizontal loading; $F_{w,total\_min}$ = p_{min_w} ´ $A_{vert\_w\_0}$ + p_{min_r} ´ $A_{vert\_r\_0}$ = **19.48** kips
Leeward net force; $F_l = F_{w,wB}$ = **-3.8** kips
Windward net force; $F_w = F_{w,wA}$ = **12.6** kips
Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **26.7** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	17.40	-0.90	17.69	-17.10	335.74	-5.74
B (-ve)	17.40	-0.90	17.69	-17.10	335.74	-5.74
C (-ve)	17.40	-0.50	17.69	-10.92	671.47	-7.33
D (-ve)	17.40	-0.30	17.69	-7.82	1358.99	-10.63

Total vertical net force; $F_{w,v}$ = **-24.41** kips
Total horizontal net force; $F_{w,h}$ = **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.98	8.69	466.66	4.05
A_2	15.00	0.80	16.98	8.69	0.00	0.00
A_3	22.79	0.80	19.13	10.19	90.02	0.92

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
B	17.40	-0.29	17.69	-7.68	556.67	-4.27
C	17.40	-0.70	17.69	-14.01	840.00	-11.77
D	17.40	-0.70	17.69	-14.01	840.00	-11.77

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d ´ H + d^2 ´ \tan(\alpha_0) / 4 =$ **556.67** ft^2

Projected vertical area of roof; $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} ´ A_{vert\_w\_90} + p_{min\_r} ´ A_{vert\_r\_90} =$ **8.91** kips

Leeward net force; $F_l = F_{w,wB} =$ **-4.3** kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **5.0** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **9.2** kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	17.40	-0.18	17.69	0.40	335.74	0.13
B (+ve)	17.40	-0.18	17.69	0.40	335.74	0.13
C (+ve)	17.40	-0.18	17.69	0.40	671.47	0.27
D (+ve)	17.40	-0.18	17.69	0.40	1358.99	0.55

Total vertical net force; $F_{w,v} =$ **0.90** kips

Total horizontal net force; $F_{w,h} =$ **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.98	15.06	466.66	7.03
A_2	15.00	0.80	16.98	15.06	0.00	0.00
A_3	22.79	0.80	19.13	16.56	90.02	1.49
B	17.40	-0.29	17.69	-1.31	556.67	-0.73
C	17.40	-0.70	17.69	-7.64	840.00	-6.42
D	17.40	-0.70	17.69	-7.64	840.00	-6.42

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d ´ H + d^2 ´ \tan(\alpha_0) / 4 =$ **556.67** ft^2

Projected vertical area of roof; $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} ´ A_{vert\_w\_90} + p_{min\_r} ´ A_{vert\_r\_90} =$ **8.91** kips

Leeward net force; $F_l = F_{w,wB} =$ **-0.7** kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} =$ **8.5** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **9.2** kips

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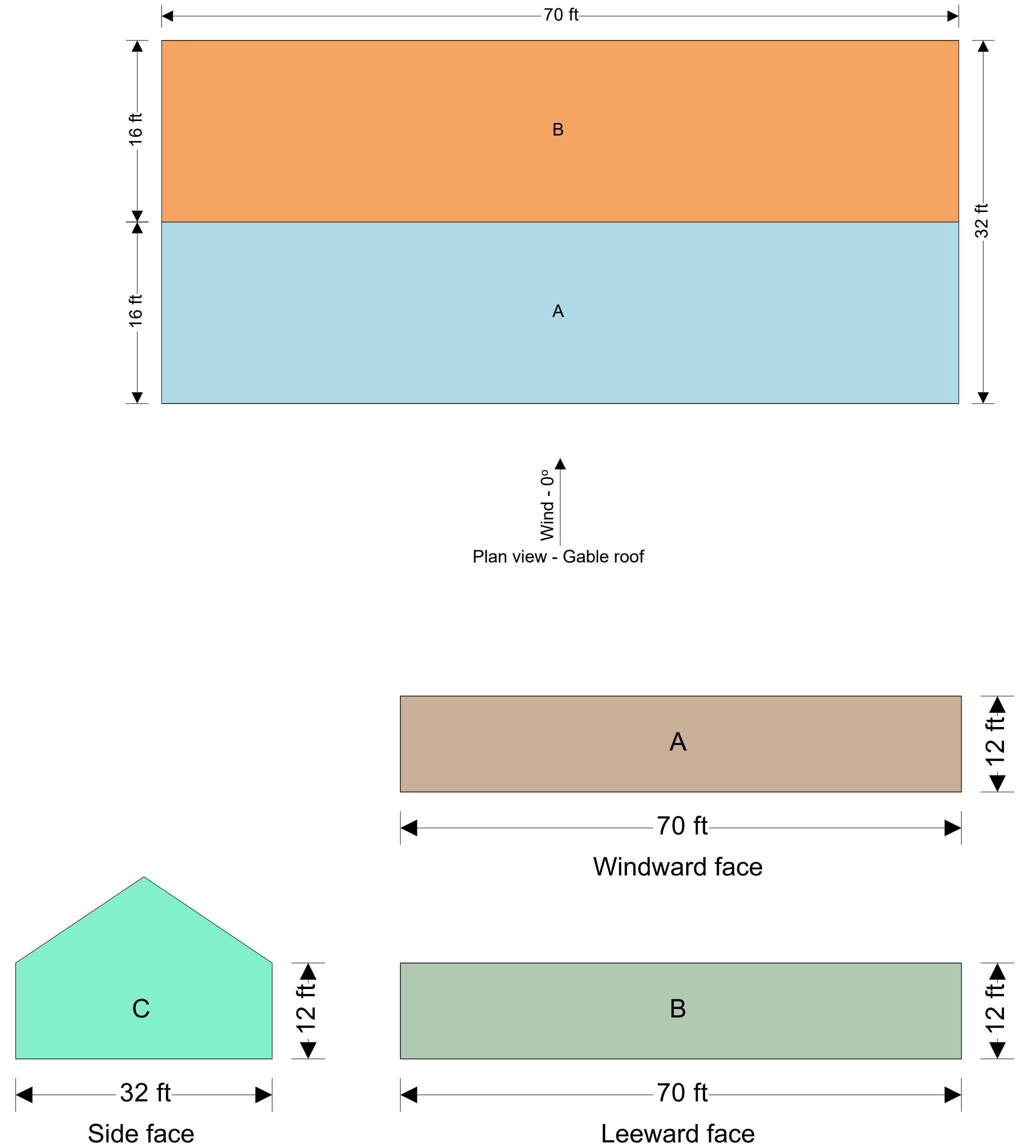
Made by: ACO/RAH	Date: 08/13/2025	Job No. 24-11
Revision:	Date:	Sheet No. 7
Project Name: Bailey Farms Clubhouse		



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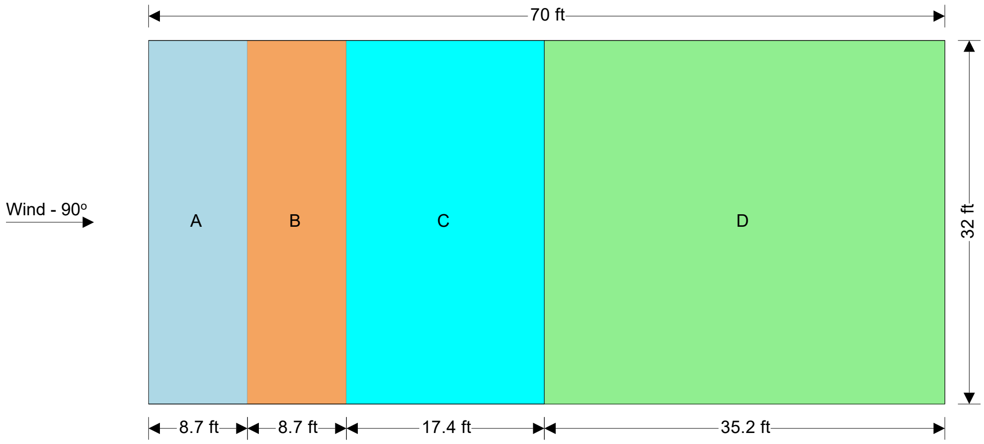
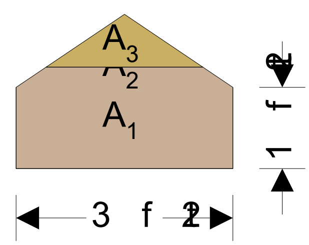
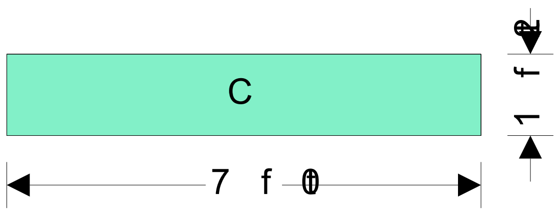


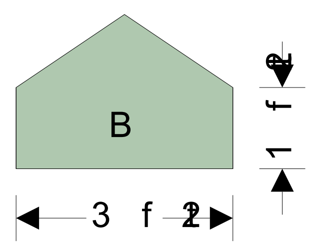
Plan view - Gable roof



Windward face



Side face



Leeward face

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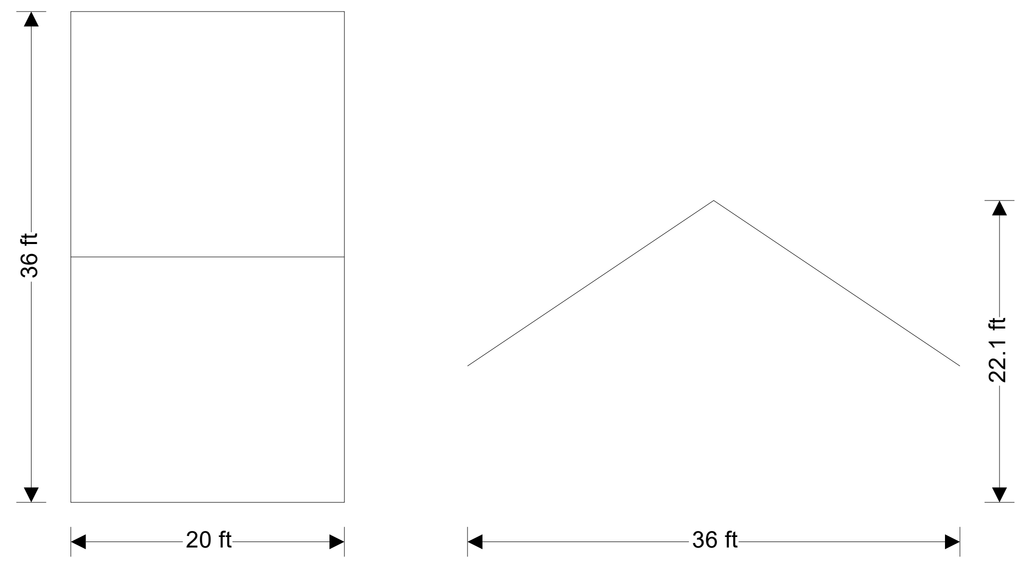
MAIN WIND FORCE RESISTING SYSTEM FOR COVERED PATIO

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.14



Plan

Elevation

Building data

Type of roof; Gable free

Length of building; b = **20.00** ft

Width of building; d = **36.00** ft

Height to eaves; H = **10.00** ft

Pitch of roof; α_0 = **34.0** deg

Mean height; h = **16.07** ft

Wind flow; Clear

General wind load requirements

Basic wind speed; V = **117.0** mph

Risk category; III

Velocity pressure exponent coef (Table 26.6-1); K_d = **0.85**

Ground elevation above sea level; z_{gl} = **0** ft

Ground elevation factor; K_e = exp(-0.0000362 ´ z_{gl}/1ft) = **1.00**

Exposure category (cl 26.7.3); B

Enclosure classification (cl.26.12); Open buildings

Internal pressure coef +ve (Table 26.13-1); GC_{pi_p} = **0.00**

Internal pressure coef –ve (Table 26.13-1); GC_{pi_n} = **0.00**

Gust effect factor for rigid structures

Terrain exposure constants (Table 26.11-1)

Integral length scale factor; l = **320.0** ft

Turbulence intensity factor; c = **0.30**

Minimum equivalent height; z_{min} = **30.0** ft

Peak factor for background response; g_Q = **3.400**

Peak factor for wind response; g_v = **3.400**

Integral length scale power law exponent; $\bar{\varepsilon}$ = **0.333**

Equivalent height of the structure; $\bar{z}$ = max (0.6 ´ h, z_{min}) = **30.00** ft

Intensity of turbulence (Eqn. 26.11-7); $I_{\bar{z}}$ = c ´ (33 ft / $\bar{z}$)$^{1/6}$ = **0.30**

Integral length scale of turbulence (Eqn. 26.11-9); $L_{\bar{z}}$ = l ´ ($\bar{z}$ / 33 ft) $^{\bar{\varepsilon}}$ = **310.00** ft

Background response (Eqn. 26.11-8); Q = √(1 / (1 + 0.63 ´ ((min(B, L) + h) / $L_{\bar{z}}$)$^{0.63}$)) = **0.927**

Gust effect factor (Eqn. 26.11-6); G = G_f = 0.925 ´ (1 + 1.7 ´ g_Q ´ $I_{\bar{z}}$ ´ Q) / (1 + 1.7 ´ g_v ´ $I_{\bar{z}}$) = **0.88**

Topography

Topography factor not significant; K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1); K_z = **0.58**

Velocity pressure; q_h = 0.00256 ´ K_z ´ K_{zt} ´ K_d ´ K_e ´ V^2 ´ 1psf/mph^2 = **17.3** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); q_i = **17.30** psf

Pressures and forces

Net pressure; p = q_h ´ G ´ C_N;

Net force; F_w = p ´ A_{ref};

Minimum design wind loading (cl.27.1.5); p_{min_r} = **16** lb/ft^2

Roof load case 1 - Wind 0 - Loadcase A

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	16.07	1.30	17.30	19.84	434.24	8.61
2 (+ve)	16.07	0.46	17.30	7.02	434.24	3.05

Total vertical net force; $F_{w,v}$ = **9.67** kips

Total horizontal net force; $F_{w,h}$ = **3.11** kips

Minimum loading

Projected vertical area of roof; $A_{vert\_r\_0}$ = b ´ d/2 ´ tan(α_0) = **242.82** ft^2

Minimum overall horizontal loading; $F_{w,total\_min}$ = p_{min_r} ´ $A_{vert\_r\_0}$ = **3.89** kips

Roof load case 2 - Wind 0 - Loadcase B

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	16.07	-0.15	17.30	-2.34	434.24	-1.02
2 (+ve)	16.07	-0.74	17.30	-11.29	434.24	-4.90

Total vertical net force; $F_{w,v}$ = **-4.91** kips

Total horizontal net force; $F_{w,h}$ = **2.17** kips

Minimum loading

Projected vertical area of roof; $A_{vert\_r\_0}$ = b ´ d/2 ´ tan(α_0) = **242.82** ft^2

Minimum overall horizontal loading; $F_{w,total\_min}$ = p_{min_r} ´ $A_{vert\_r\_0}$ = **3.89** kips

Roof load case 3 - Wind 90 - Loadcase A

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	16.07	-0.80	17.30	-12.21	697.85	-8.52
2 (+ve)	16.07	-0.60	17.30	-9.16	170.63	-1.56

Total vertical net force; $F_{w,v}$ = **-8.36** kips

Total horizontal net force; $F_{w,h}$ = **0.00** kips

Minimum loading

Projected vertical area of roof; $A_{vert\_r\_90}$ = **0.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min}$ = p_{min_r} ´ $A_{vert\_r\_90}$ = **0.00** kips

Roof load case 4 - Wind 90 - Loadcase B

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	16.07	0.80	17.30	12.21	697.85	8.52
2 (+ve)	16.07	0.50	17.30	7.63	170.63	1.30

Total vertical net force; $F_{w,v}$ = **8.14** kips

Total horizontal net force; $F_{w,h}$ = **0.00** kips

Minimum loading

Projected vertical area of roof; $A_{vert\_r\_90}$ = **0.00** ft^2

Minimum overall horizontal loading; $F_{w,total\_min}$ = p_{min_r} ´ $A_{vert\_r\_90}$ = **0.00** kips

Horizontal wind loads on open or partially enclosed buildings with transverse frames (cl.27.3.2)

Coefficient for zone 5 load case B (Figure 28.3-1); $GC_{pf\_5\_B}$ = **0.40**

Coefficient for zone 5E load case B (Figure 28.3-1); $GC_{pf\_5E\_B}$ = **0.61**

Coefficient for zone 6 load case B (Figure 28.3-1); $GC_{pf\_6\_B}$ = **-0.29**

Coefficient for zone 6E load case B (Figure 28.3-1); $GC_{pf\_6E\_B}$ = **-0.43**

Area for zone 5 load case B; A_{ref,e_5_B} = **546** ft^2

Area for zone 5E load case B; A_{ref,e_5E_B} = **33** ft^2

Area for zone 6 load case B; A_{ref,e_6_B} = **546** ft^2

Area for zone 6E load case B; A_{ref,e_6E_B} = **33** ft^2

Windward external pressure coefficient; $GC_{pf\_windward}$ = ($GC_{pf\_5\_B}$ ´ A_{ref,e_5_B} + $GC_{pf\_5E\_B}$ ´ A_{ref,e_5E_B}) / (A_{ref,e_5_B} + A_{ref,e_5E_B}) = **0.41**

Leeward external pressure coefficient; $GC_{pf\_leeward}$ = ($GC_{pf\_6\_B}$ ´ A_{ref,e_6_B} + $GC_{pf\_6E\_B}$ ´ A_{ref,e_6E_B}) / (A_{ref,e_6_B} + A_{ref,e_6E_B}) = **-0.30**

Total end wall area for equiv. enclosed building; A_E = A_{ref,e_5_B} + A_{ref,e_5E_B} = **579** ft^2

Effective solid area of the end wall; A_S = **150** ft^2

Solidity ratio; ϕ = A_S / A_E = **0.26**

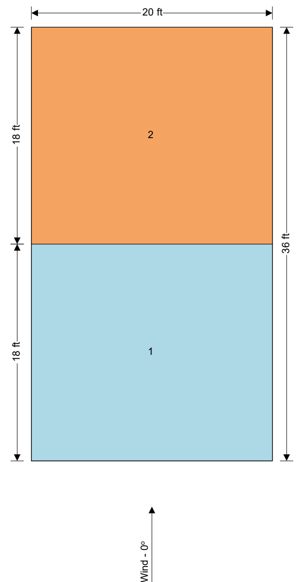
Frame width factor; K_B = 1.8 - 0.01 ´ d/1ft = **1.44**

Number of frames; $n = \mathbf{1}$

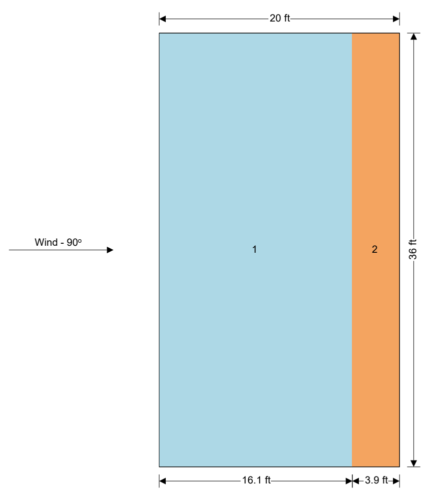
Shielding factor; $K_S = 0.6 + 0.073 \times (\max(n, 3) - 3) + (1.25 \times \phi^{1.8}) = \mathbf{0.71}$

Horiz. pressure in the longitudinal dir. (Eqn.28.3-3); $p_{H\_add} = q_h \times (GC_{pf\_windward} - GC_{pf\_leeward}) \times K_B \times K_S = \mathbf{12.56}$ psf

Add. long. force to be resisted by the MWFRS; $F_{H\_add} = p_{H\_add} \times A_E = \mathbf{7.26}$ kips



Plan view - Gable roof



Plan view - Gable roof

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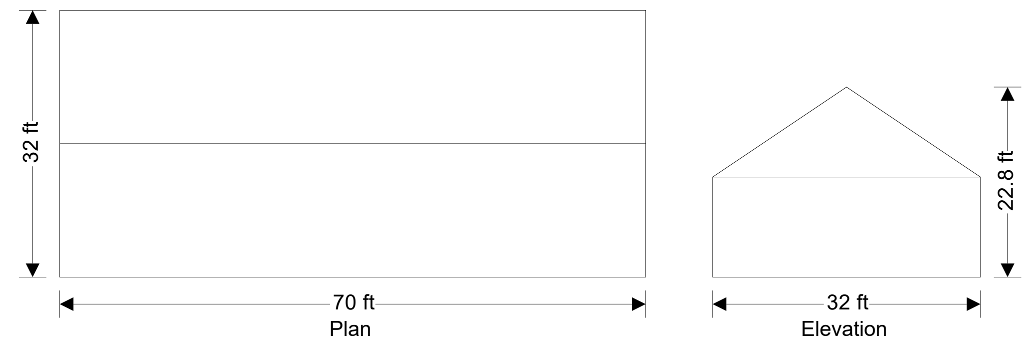
COMPONENTS AND CLADDING WIND

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.16



Building data

Type of roof;	Gable
Length of building;	b = **70.00** ft
Width of building;	d = **32.00** ft
Height to eaves;	H = **12.00** ft
Pitch of roof;	α_0 = **34.0** deg
Mean height;	h = **17.40** ft
End zone width;	a = max(min(0.1´min(b, d), 0.4´h), 0.04´min(b, d), 3ft) = **3.20** ft

General wind load requirements

Basic wind speed;	V = **117.0** mph
Risk category;	III
Velocity pressure exponent coef (Table 26.6-1);	K_d = **0.85**
Ground elevation above sea level;	z_{gl} = **0** ft
Ground elevation factor;	K_e = exp(-0.0000362 ´ z_{gl}/1ft) = **1.00**
Exposure category (cl 26.7.3);	B
Enclosure classification (cl.26.12);	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1);	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1);	GC_{pi_n} = **-0.18**

Gust effect factor for rigid structures

Terrain exposure constants (Table 26.11-1)

Integral length scale factor;	l = **320.0** ft
Turbulence intensity factor;	c = **0.30**
Minimum equivalent height;	z_{min} = **30.0** ft
Peak factor for background response;	g_Q = **3.400**
Peak factor for wind response;	g_v = **3.400**
Integral length scale power law exponent;	$\bar{\varepsilon}$ = **0.333**
Equivalent height of the structure;	$\bar{z}$ = max (0.6 ´ h, z_{min}) = **30.00** ft

Intensity of turbulence (Eqn. 26.11-7); $I_{\bar{z}} = c \times (33\text{ ft} / \bar{z})^{1/6} = \mathbf{0.30}$

Integral length scale of turbulence (Eqn. 26.11-9); $L_{\bar{z}} = l \times (\bar{z} / 33\text{ ft})^{\bar{\varepsilon}} = \mathbf{310.00}$ ft

Background response (Eqn. 26.11-8); $Q = \sqrt{(1 / (1 + 0.63 \times ((\min(B, L) + h) / L_{\bar{z}})^{0.63}))} = \mathbf{0.914}$

Gust effect factor (Eqn. 26.11-6); $G = G_f = 0.925 \times (1 + 1.7 \times g_Q \times I_{\bar{z}} \times Q) / (1 + 1.7 \times g_v \times I_{\bar{z}}) = \mathbf{0.87}$

Topography

Topography factor not significant; $K_{zt} = 1.0$

Velocity pressure

Velocity pressure coefficient (Table 26.10-1); $K_z = \mathbf{0.59}$

Velocity pressure; $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2 = \mathbf{17.7}$ psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); $q_i = \mathbf{17.69}$ psf

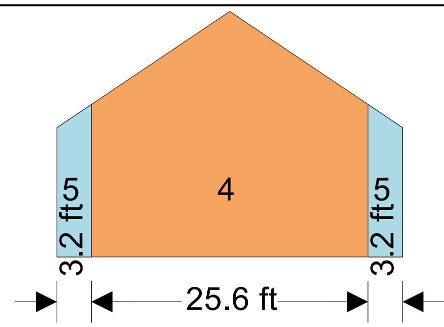
Equations used in tables

Net pressure; $p = q_h \times (GC_p - GC_{pi})$

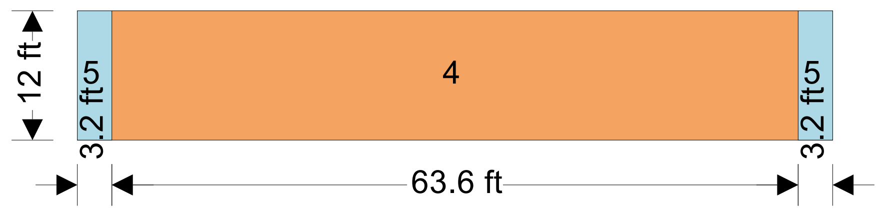
Components and cladding pressures - Wall (Table 30.3-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	1.00	-1.10	20.9	-22.6
50 sf	4	-	-	50.0	0.88	-0.98	18.7	-20.5
200 sf	4	-	-	200.0	0.77	-0.87	16.8	-18.6
>500 sf	4	-	-	500.1	0.70	-0.80	15.6 #	-17.3
<=10 sf	5	-	-	10.0	1.00	-1.40	20.9	-28.0
50 sf	5	-	-	50.0	0.88	-1.15	18.7	-23.6
200 sf	5	-	-	200.0	0.77	-0.94	16.8	-19.8
>500 sf	5	-	-	500.1	0.70	-0.80	15.6 #	-17.3

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Elevation of gable wall



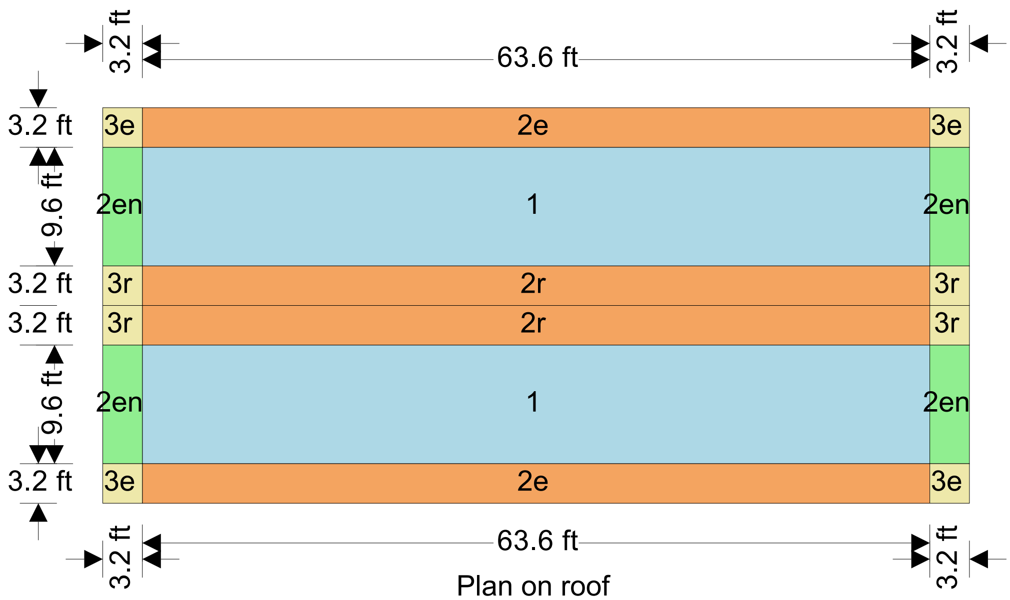
Elevation of side wall

Components and cladding pressures - Roof (Figure 30.3-2D)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.90	-1.80	19.1	-35.0
25 sf	1	-	-	25.0	0.74	-1.40	16.3	-28.0
50 sf	1	-	-	50.0	0.62	-1.10	14.2 #	-22.7
>100 sf	1	-	-	100.1	0.50	-0.80	12.0 #	-17.3
<=10 sf	2e	-	-	10.0	0.90	-1.80	19.1	-35.0
25 sf	2e	-	-	25.0	0.74	-1.40	16.3	-28.0
50 sf	2e	-	-	50.0	0.62	-1.10	14.2 #	-22.7
>100 sf	2e	-	-	100.1	0.50	-0.80	12.0 #	-17.3
<=10 sf	2n	-	-	10.0	0.90	-2.00	19.1	-38.6
50 sf	2n	-	-	50.0	0.62	-1.46	14.2 #	-29.1
100 sf	2n	-	-	100.0	0.50	-1.23	12.0 #	-25.0
>200 sf	2n	-	-	200.1	0.50	-1.00	12.0 #	-20.9
<=10 sf	2r	-	-	10.0	0.90	-1.80	19.1	-35.0
25 sf	2r	-	-	25.0	0.74	-1.40	16.3	-28.0
50 sf	2r	-	-	50.0	0.62	-1.10	14.2 #	-22.7
>100 sf	2r	-	-	100.1	0.50	-0.80	12.0 #	-17.3
<=2 sf	3e	-	-	2.0	0.90	-3.20	19.1	-59.8
10 sf	3e	-	-	10.0	0.90	-2.49	19.1	-47.3

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
100 sf	3e	-	-	100.0	0.50	-1.48	12.0 #	-29.4
>300 sf	3e	-	-	300.1	0.50	-1.00	12.0 #	-20.9
<=10 sf	3r	-	-	10.0	0.90	-2.00	19.1	-38.6
50 sf	3r	-	-	50.0	0.62	-1.46	14.2 #	-29.1
100 sf	3r	-	-	100.0	0.50	-1.23	12.0 #	-25.0
>200 sf	3r	-	-	200.1	0.50	-1.00	12.0 #	-20.9

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof

SHEAR WALL DESIGN

EAST SHEAR WALL DESIGN (NDS)

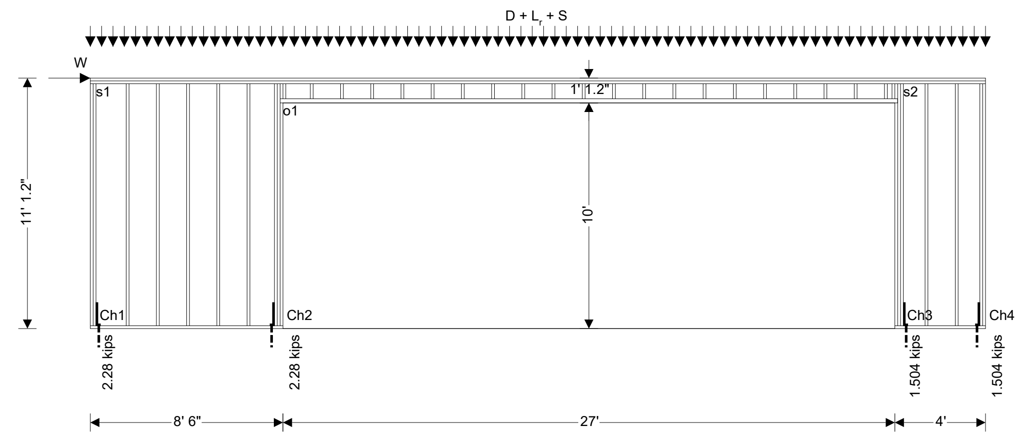
In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Panel details

Structural wood panel sheathing on one side

Panel height;	h = **11.1** ft
Panel length;	b = **39.5** ft



Panel opening details

Width of opening;	w_{o1} = **27** ft
Height of opening;	h_{o1} = **10** ft
Height to underside of lintel over opening;	l_{o1} = **10** ft
Position of opening;	P_{o1} = **8.5** ft
Total area of wall;	A = h ´ b - w_{o1} ´ h_{o1} = **168.45** ft^2

Panel construction

Nominal stud size;	2" x 6"
Dressed stud size;	1.5" x 5.5"
Cross-sectional area of studs;	A_s = **8.25** in^2
Stud spacing;	s = **16** in
Nominal end post size;	2 x 2" x 6"
Dressed end post size;	2 x 1.5" x 5.5"
Cross-sectional area of end posts;	A_e = **16.5** in^2
Hole diameter;	Dia = **1** in
Net cross-sectional area of end posts;	A_{en} = **13.5** in^2
Nominal collector size;	2 x 2" x 6"
Dressed collector size;	2 x 1.5" x 5.5"
Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in

Vertical anchor stiffness;	k_a = **28800** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity;	G = **0.42**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	15/32" wood panel oriented strandboard sheathing
Fastener type;	8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity;	v_n = 1065 plf ´ min[1 - (0.5 - G), 1] = **979.8** lb/ft
Apparent shear wall shear stiffness;	G_a = **19** kips/in

Loading details

Dead load acting on top of panel;	D = **360** lb/ft
Roof live load acting on top of panel;	L_r = **360** lb/ft
Snow load acting on top of panel;	S = **360** lb/ft
Self weight of panel;	S_{wt} = **15** lb/ft^2
In plane wind load acting at head of panel;	W = **3600** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.30**
Size factor for compression – Table 4A;	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3;	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**
Incising factor – cl.4.3.8;	C_i = **1.00**

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Buckling stiffness factor – cl.4.4.2;	C_T = **1.00**
Bearing area factor - cl. 3.10.4;	C_b = **1.0**
Adjusted modulus of elasticity;	$E_{min}' = E_{min}$ ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T = **510000** psi
Critical buckling design value;	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **715** psi
Reference compression design value;	$F_c^* = F_c$ ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i = **2024** psi
For sawn lumber;	c = **0.8**
Column stability factor – eqn.3.7-1;	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.32**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio;	3.5
Segment 1 wall length;	b_1 = **8.5** ft
Shear wall aspect ratio;	h / b_1 = **1.306**
Segment 2 wall length;	b_2 = **4** ft
Shear wall aspect ratio;	h / b_2 = **2.775**

Segmented shear wall capacity - Equal deflection method

Effective segment length;	$b_{1\_eff} = b_1 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **8.13** ft
Effective segment length;	$b_{2\_eff} = b_2 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **3.63** ft
Wind loading:	
Segment 1 vertical unit deflection;	$\Delta_{a1} = h / b_1$ ´ $(1 / k_a + (0.04\text{ in} / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_1 / b_{1\_eff}))$ = **0.053** in/kip
Segment 1 stiffness;	$k_1 = 1 / (2$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1^2) + h / (G_a$ ´ $b_1) + h$ ´ $\Delta_{a1} / b_1)$ = **6.912** kips/in
Unit shear capacity, widest segment;	$v_{sww1} = v_w / 2.0$ = **489.9** plf
Vertical deflction under capacity load;	$\Delta_{a\_Cap} = h$ ´ v_{sww1} ´ $(1 / k_a + (0.04\text{ in} / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_1 / b_{1\_eff}))$ = **0.221** in
Deflection under capacity load;	$\delta_{Cap} = 2$ ´ v_{sww1} ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1) + v_{sww1}$ ´ $h / (G_a) + h$ ´ Δ_{a_Cap} / b_1 = **0.602** in
Segment 2 vertical unit deflection;	$\Delta_{a1} = h / b_2$ ´ $(1 / k_a + (0.04\text{ in} / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_2 / b_{2\_eff}))$ = **0.114** in/kip
Segment 2 stiffness;	$k_2 = 1 / (2$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_2^2) + h / (G_a$ ´ $b_2) + h$ ´ $\Delta_{a1} / b_2)$ = **2.035** kips/in
Segment 2 unit shear at δ_{Cap};	$v_{dsww2} = \delta_{Cap}$ ´ k_2 / b_2 = **306.43** plf
Segment 2 shear capacity;	$v_{sww2} = v_w$ ´ $(1.25 - 0.125$ ´ $h / b_2) / 2.0$ = **442.44** plf
	v_{dsww2} / v_{sww2} = **0.693**
	PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}
Maximum shear force under wind loading;	$V_{w\_max} = 0.6$ ´ W = **2.16** kips
Shear capacity for wind loading;	$V_w = v_{sww1}$ ´ $b_1 + \min(v_{sww2}, v_{dsww2})$ ´ b_2 = **5.39** kips
	V_{w_max} / V_w = **0.401**
	PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio;	h / b_1 = **1.306**
Load combination 5	
Shear force for maximum tension;	V = 0.6 ´ W = **2.16** kips
Axial force for maximum tension;	P = 0 kips = **0** kips
Maximum tensile force in chord;	$T = V$ ´ $(k_1 / \text{sum}(k_1, k_2))$ ´ $h / b_{1\_eff} - P$ = **2.280** kips
Maximum applied tensile stress;	$f_t = T / A_{en}$ = **169** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **936** lb/in^2

f_t / F_t' = **0.180**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **2.16** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.351** kips

Maximum compressive force in chord; $C = V \times (k_1 / \mathrm{sum}(k_1,k_2)) \times h / b_{1\_eff} + P$ = **2.631** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **159** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **653** lb/in^2

f_c / F_c' = **0.244**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **425** lb/in^2

f_c / F_{c_perp}' = **0.375**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 3 and 4

Shear wall aspect ratio; h / b_2 = **2.775**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.16** kips

Axial force for maximum tension; P = 0 kips = **0** kips

Maximum tensile force in chord; $T = V \times (k_2 / \mathrm{sum}(k_1,k_2)) \times h / b_{2\_eff} - P$ = **1.504** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **111** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **936** lb/in^2

f_t / F_t' = **0.119**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **2.16** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.351** kips

Maximum compressive force in chord; $C = V \times (k_2 / \mathrm{sum}(k_1,k_2)) \times h / b_{2\_eff} + P$ = **1.855** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **112** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **653** lb/in^2

f_c / F_c' = **0.172**

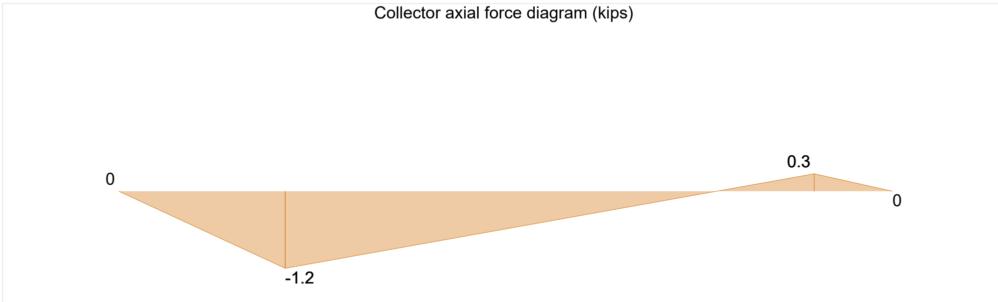
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **425** lb/in^2

f_c / F_{c_perp}' = **0.265**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Collector capacity



Maximum shear force on wall; $V_{max} = V_{w\_max} =$ **2.16** kips

Maximum force in collector; $P_{coll} =$ **1.204** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) =$ **73** lb/in^2

Design tensile stress; $F_t' = F_t$ ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ $C_i =$ **936** lb/in^2

$f_t / F_t' =$ **0.078**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) =$ **73** lb/in^2

Column stability factor; $C_P =$ **1.00**

Design compressive stress; $F_c' = F_c$ ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ $C_P =$ **2024** lb/in^2

$f_c / F_c' =$ **0.036**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; $T_1 =$ **2.28** kips

Chord 2; $T_2 =$ **2.28** kips

Chord 3; $T_3 =$ **1.504** kips

Chord 4; $T_4 =$ **1.504** kips

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv}$ ´ $W =$ **2.16** kips

Deflection limit; $\Delta_{w\_allow} = h / 400 =$ **0.333** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w}$ ´ $(k_1 / sum(k_1,k_2)) / b_1 =$ **196.33** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w}$ ´ h ´ $b_1 / b_{1\_eff}) =$ **2.280** kips

Chord compression force; $C_\delta = max(0\ kips, v_{\delta w}$ ´ h ´ $b_1 / b_{1\_eff}) =$ **2.280** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a =$ **0.079** in

Vertical compression at chord; $\Delta_C = 0.04\ in$ ´ $C_\delta / (A_e$ ´ $F_{c\_perp}) =$ **0.013** in

Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C)$ ´ $(b_1 / b_{1\_eff}) =$ **0.096** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2$ ´ $v_{\delta w}$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1) + v_{\delta w}$ ´ $h / (G_a) + h$ ´ $\Delta_a / b_1 =$ **0.252** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.755**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear;	$v_{\delta w} = V_{\delta w} \times (k_2 / \mathrm{sum}(k_1,k_2)) / b_2 =$ **122.8** lb/ft
Anchor tension force;	$T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h \times b_2 / b_{2\_eff}) =$ **1.504** kips
Chord compression force;	$C_\delta = \max(0 \text{ kips}, v_{\delta w} \times h \times b_2 / b_{2\_eff}) =$ **1.504** kips
Vertical elongation at anchor;	$\Delta_T = T_\delta / k_a =$ **0.052** in
Vertical compression at chord;	$\Delta_C = 0.04 \text{ in} \times C_\delta / (A_e \times F_{c\_perp}) =$ **0.009** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C) \times (b_2 / b_{2\_eff}) =$ **0.067** in
Segment 2 deflection – Eqn. 4.3-1;	$\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.272** in
	$\delta_{sww2} / \Delta_{w\_allow} =$ **0.818**

PASS - Shear wall deflection is less than deflection limit

WEST SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

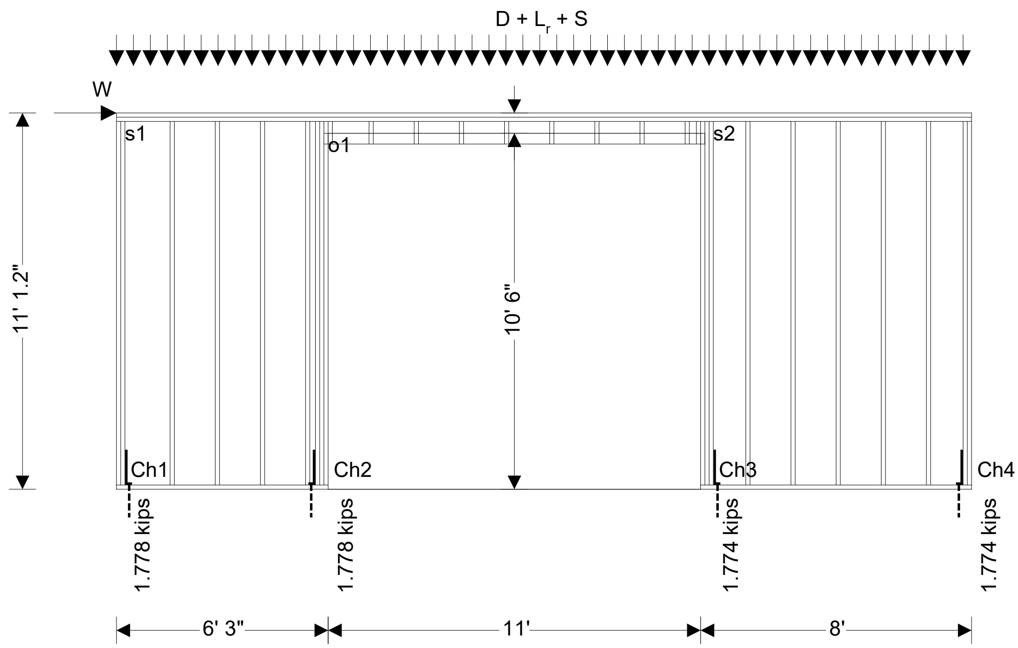
Tedds calculation version 1.2.10

Panel details

Structural wood panel sheathing on one side

Panel height; h = **11.1** ft

Panel length; b = **25.25** ft



Panel opening details

Width of opening; w_{o1} = **11** ft

Height of opening; h_{o1} = **10.5** ft

Height to underside of lintel over opening; l_{o1} = **10.5** ft

Position of opening; P_{o1} = **6.25** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1}$ = **164.775** ft^2

Panel construction

Nominal stud size; 2" x 6"

Dressed stud size; 1.5" x 5.5"

Cross-sectional area of studs; A_s = **8.25** in^2

Stud spacing; s = **16** in

Nominal end post size; 2 x 2" x 6"

Dressed end post size; 2 x 1.5" x 5.5"

Cross-sectional area of end posts; A_e = **16.5** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **13.5** in^2

Nominal collector size; 2 x 2" x 6"

Dressed collector size;	2 x 1.5" x 5.5"
Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in
Vertical anchor stiffness;	k_a = **28800** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity;	G = **0.42**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	15/32" wood panel oriented strandboard sheathing
Fastener type;	8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity;	v_n = 1065 plf ´ min[1 - (0.5 - G), 1] = **979.8** lb/ft
Apparent shear wall shear stiffness;	G_a = **19** kips/in

Loading details

Dead load acting on top of panel;	D = **360** lb/ft
Roof live load acting on top of panel;	L_r = **360** lb/ft
Snow load acting on top of panel;	S = **360** lb/ft
Self weight of panel;	S_{wt} = **15** lb/ft^2
In plane wind load acting at head of panel;	W = **3600** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.30**
Size factor for compression – Table 4A;	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3;	C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8;	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2;	C_T = **1.00**
Bearing area factor - cl. 3.10.4;	C_b = **1.0**
Adjusted modulus of elasticity;	$E_{min}' = E_{min} ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T$ = **510000** psi
Critical buckling design value;	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **715** psi
Reference compression design value;	$F_c^* = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i$ = **2024** psi
For sawn lumber;	c = **0.8**
Column stability factor – eqn.3.7-1;	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.32**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio;	3.5
Segment 1 wall length;	b_1 = **6.25** ft
Shear wall aspect ratio;	h / b_1 = **1.776**
Segment 2 wall length;	b_2 = **8** ft
Shear wall aspect ratio;	h / b_2 = **1.388**

Segmented shear wall capacity - Equal deflection method

Effective segment length;	$b_{1\_eff} = b_1 - 3 / 2 ´ b_{EndPost} - e_{anchor}$ = **5.88** ft
Effective segment length;	$b_{2\_eff} = b_2 - 3 / 2 ´ b_{EndPost} - e_{anchor}$ = **7.62** ft
Wind loading:	
Segment 2 vertical unit deflection;	$\Delta_{a1} = h / b_2 ´ (1 / k_a + (0.04 \text{ in} / (A_e ´ F_{c\_perp})) ´ (b_2 / b_{2\_eff}))$ = **0.056** in/kip
Segment 2 stiffness;	$k_2 = 1 / (2 ´ h^3 / (3 ´ E ´ A_e ´ b_2^2) + h / (G_a ´ b_2) + h ´ \Delta_{a1} / b_2)$ = **6.297** kips/in
Unit shear capacity, widest segment;	$v_{sww2} = v_w / 2.0$ = **489.9** plf
Vertical deflction under capacity load;	$\Delta_{a\_Cap} = h ´ v_{sww2} ´ (1 / k_a + (0.04 \text{ in} / (A_e ´ F_{c\_perp})) ´ (b_2 / b_{2\_eff}))$ = **0.221** in
Deflection under capacity load;	$\delta_{Cap} = 2 ´ v_{sww2} ´ h^3 / (3 ´ E ´ A_e ´ b_2) + v_{sww2} ´ h / (G_a) + h ´ \Delta_{a\_Cap} / b_2$ = **0.622** in
Segment 1 vertical unit deflection;	$\Delta_{a1} = h / b_1 ´ (1 / k_a + (0.04 \text{ in} / (A_e ´ F_{c\_perp})) ´ (b_1 / b_{1\_eff}))$ = **0.072** in/kip
Segment 1 stiffness;	$k_1 = 1 / (2 ´ h^3 / (3 ´ E ´ A_e ´ b_1^2) + h / (G_a ´ b_1) + h ´ \Delta_{a1} / b_1)$ = **4.269** kips/in
Segment 1 unit shear at δ_{Cap};	$v_{dsww1} = \delta_{Cap} ´ k_1 / b_1$ = **425.06** plf
Segment 1 shear capacity;	$v_{sww1} = v_w / 2.0$ = **489.9** plf
	v_{dsww1} / v_{sww1} = **0.868**

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Maximum shear force under wind loading;	$V_{w\_max} = 0.6 ´ W$ = **2.16** kips
Shear capacity for wind loading;	$V_w = \min(v_{sww1}, v_{dsww1}) ´ b_1 + v_{sww2} ´ b_2$ = **6.576** kips
	V_{w_max} / V_w = **0.328**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio;	h / b_1 = **1.776**

Load combination 5

Shear force for maximum tension; V = 0.6 ´ W = **2.16** kips

Axial force for maximum tension; P = 0 kips = **0** kips

Maximum tensile force in chord; T = V ´ (k_1 / sum(k_1,k_2)) ´ h / b_{1_eff} - P = **1.778** kips

Maximum applied tensile stress; f_t = T / A_{en} = **132** lb/in^2

Design tensile stress; F_t' = F_t ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i = **936** lb/in^2

f_t / F_t' = **0.141**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 ´ W = **2.16** kips

Axial force for maximum compression; P = ((D + S_{wt} ´ h)) ´ s / 2 = **0.351** kips

Maximum compressive force in chord; C = V ´ (k_1 / sum(k_1,k_2)) ´ h / b_{1_eff} + P = **2.129** kips

Maximum applied compressive stress; f_c = C / A_e = **129** lb/in^2

Design compressive stress; F_c' = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P = **653** lb/in^2

f_c / F_c' = **0.198**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; F_{c_perp}' = F_{c_perp} ´ C_{Mc} ´ C_{tc} ´ C_i ´ C_b = **425** lb/in^2

f_c / F_{c_perp}' = **0.304**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 3 and 4

Shear wall aspect ratio; h / b_2 = **1.388**

Load combination 5

Shear force for maximum tension; V = 0.6 ´ W = **2.16** kips

Axial force for maximum tension; P = 0 kips = **0** kips

Maximum tensile force in chord; T = V ´ (k_2 / sum(k_1,k_2)) ´ h / b_{2_eff} - P = **1.774** kips

Maximum applied tensile stress; f_t = T / A_{en} = **131** lb/in^2

Design tensile stress; F_t' = F_t ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i = **936** lb/in^2

f_t / F_t' = **0.140**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 ´ W = **2.16** kips

Axial force for maximum compression; P = ((D + S_{wt} ´ h)) ´ s / 2 = **0.351** kips

Maximum compressive force in chord; C = V ´ (k_2 / sum(k_1,k_2)) ´ h / b_{2_eff} + P = **2.125** kips

Maximum applied compressive stress; f_c = C / A_e = **129** lb/in^2

Design compressive stress; F_c' = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P = **653** lb/in^2

f_c / F_c' = **0.197**

PASS - Design compressive stress exceeds maximum applied compressive stress

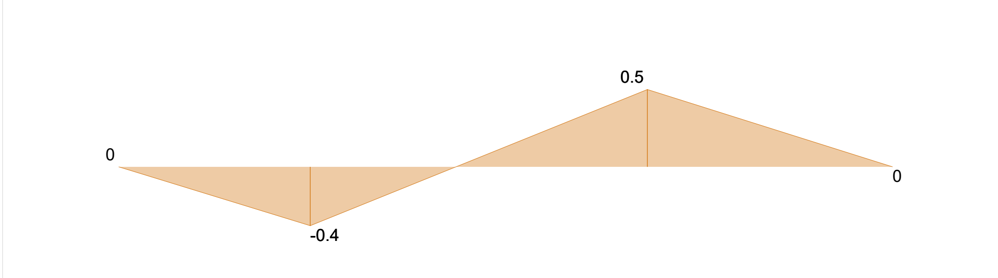
Design bearing compr. stress, bottom plate; F_{c_perp}' = F_{c_perp} ´ C_{Mc} ´ C_{tc} ´ C_i ´ C_b = **425** lb/in^2

f_c / F_{c_perp}' = **0.303**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Collector capacity

Collector axial force diagram (kips)



Maximum shear force on wall; $V_{max} = V_{w\_max} =$ **2.16** kips

Maximum force in collector; $P_{coll} =$ **0.534** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) =$ **32** lb/in^2

Design tensile stress; $F_t' = F_t ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i =$ **936** lb/in^2

$f_t / F_t' =$ **0.035**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) =$ **32** lb/in^2

Column stability factor; $C_P =$ **1.00**

Design compressive stress; $F_c' = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P =$ **2024** lb/in^2

$f_c / F_c' =$ **0.016**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; $T_1 =$ **1.778** kips

Chord 2; $T_2 =$ **1.778** kips

Chord 3; $T_3 =$ **1.774** kips

Chord 4; $T_4 =$ **1.774** kips

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} ´ W =$ **2.16** kips

Deflection limit; $\Delta_{w\_allow} = h / 400 =$ **0.333** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} ´ (k_1 / \mathrm{sum}(k_1,k_2)) / b_1 =$ **150.61** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} ´ h ´ b_1 / b_{1\_eff}) =$ **1.778** kips

Chord compression force; $C_\delta = \max(0 \text{ kips}, v_{\delta w} ´ h ´ b_1 / b_{1\_eff}) =$ **1.778** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a =$ **0.062** in

Vertical compression at chord; $\Delta_C = 0.04 \text{ in} ´ C_\delta / (A_e ´ F_{c\_perp}) =$ **0.010** in

Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C) ´ (b_1 / b_{1\_eff}) =$ **0.076** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 ´ v_{\delta w} ´ h^3 / (3 ´ E ´ A_e ´ b_1) + v_{\delta w} ´ h / (G_a) + h ´ \Delta_a / b_1 =$ **0.235** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.706**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear;	$v_{\delta w} = V_{\delta w}$ ´ $(k_2 / sum(k_1,k_2)) / b_2 =$ **152.34** lb/ft
Anchor tension force;	$T_\delta = max(0\ kips, v_{\delta w}$ ´ h ´ $b_2 / b_{2\_eff}) =$ **1.774** kips
Chord compression force;	$C_\delta = max(0\ kips, v_{\delta w}$ ´ h ´ $b_2 / b_{2\_eff}) =$ **1.774** kips
Vertical elongation at anchor;	$\Delta_T = T_\delta / k_a =$ **0.062** in
Vertical compression at chord;	$\Delta_C = 0.04$ in ´ $C_\delta / (A_e$ ´ $F_{c\_perp}) =$ **0.010** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C)$ ´ $(b_2 / b_{2\_eff}) =$ **0.075** in
Segment 2 deflection – Eqn. 4.3-1;	$\delta_{sww2} = 2$ ´ $v_{\delta w}$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_2) + v_{\delta w}$ ´ $h / (G_a) + h$ ´ $\Delta_a / b_2 =$ **0.202** in
	$\delta_{sww2} / \Delta_{w\_allow} =$ **0.608**

PASS - Shear wall deflection is less than deflection limit

SOUTHWEST SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

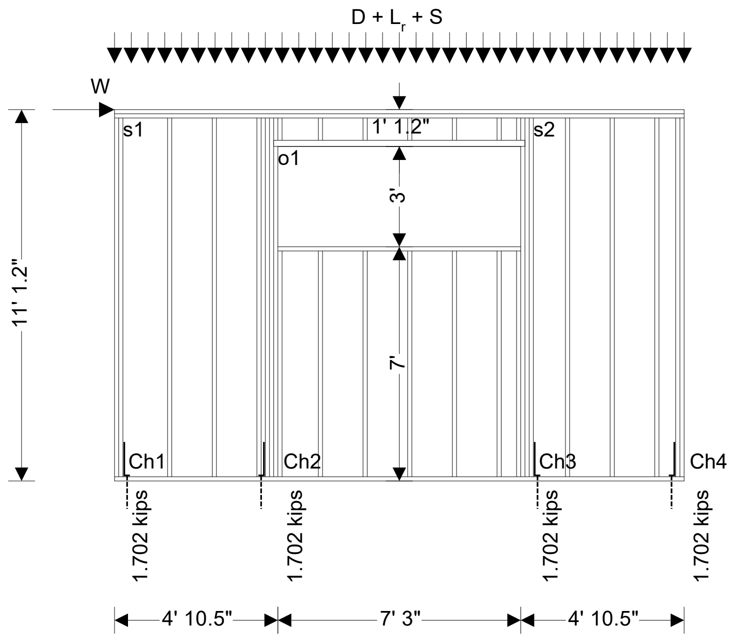
Tedds calculation version 1.2.10

Panel details

Structural wood panel sheathing on one side

Panel height; $h = \mathbf{11.1}$ ft

Panel length; $b = \mathbf{17}$ ft



Panel opening details

Width of opening; $w_{o1} = \mathbf{7.25}$ ft

Height of opening; $h_{o1} = \mathbf{3}$ ft

Height to underside of lintel over opening; $l_{o1} = \mathbf{10}$ ft

Position of opening; $P_{o1} = \mathbf{4.875}$ ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} = \mathbf{166.95}\ ft^2$

Panel construction

Nominal stud size; 2" x 6"

Dressed stud size; 1.5" x 5.5"

Cross-sectional area of studs; $A_s = \mathbf{8.25}\ in^2$

Stud spacing; $s = \mathbf{16}$ in

Nominal end post size; 2 x 2" x 6"

Dressed end post size; 2 x 1.5" x 5.5"

Cross-sectional area of end posts; $A_e = \mathbf{16.5}\ in^2$

Hole diameter; $Dia = \mathbf{1}$ in

Net cross-sectional area of end posts; $A_{en} = \mathbf{13.5}\ in^2$

Nominal collector size; 2 x 2" x 6"

Dressed collector size;	2 x 1.5" x 5.5"
Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in
Vertical anchor stiffness;	k_a = **28800** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity;	G = **0.42**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	15/32" wood panel oriented strandboard sheathing
Fastener type;	8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity;	v_n = 1065 plf ´ min[1 - (0.5 - G), 1] = **979.8** lb/ft
Apparent shear wall shear stiffness;	G_a = **19** kips/in

Loading details

Dead load acting on top of panel;	D = **240** lb/ft
Roof live load acting on top of panel;	L_r = **240** lb/ft
Snow load acting on top of panel;	S = **240** lb/ft
Self weight of panel;	S_{wt} = **15** lb/ft^2
In plane wind load acting at head of panel;	W = **2300** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.30**
Size factor for compression – Table 4A;	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3;	C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8; C_i = **1.00**

Buckling stiffness factor – cl.4.4.2; C_T = **1.00**

Bearing area factor - cl. 3.10.4; C_b = **1.0**

Adjusted modulus of elasticity; $E_{min}' = E_{min} ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T$ = **510000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **715** psi

Reference compression design value; $F_c^* = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i$ = **2024** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.32**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **4.875** ft

Shear wall aspect ratio; h / b_1 = **2.277**

Segment 2 wall length; b_2 = **4.875** ft

Shear wall aspect ratio; h / b_2 = **2.277**

Segmented shear wall capacity - Equal deflection method

Effective segment length; $b_{1\_eff} = b_1 - 3 / 2 ´ b_{EndPost} - e_{anchor}$ = **4.50** ft

Effective segment length; $b_{2\_eff} = b_2 - 3 / 2 ´ b_{EndPost} - e_{anchor}$ = **4.50** ft

Wind loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / b_2 ´ (1 / k_a + (0.04 \text{ in} / (A_e ´ F_{c\_perp})) ´ (b_2 / b_{2\_eff}))$ = **0.093** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 ´ h^3 / (3 ´ E ´ A_e ´ b_2^2) + h / (G_a ´ b_2) + h ´ \Delta_{a1} / b_2)$ = **2.842** kips/in

Unit shear capacity, widest segment; $v_{sww2} = v_w ´ (1.25 - 0.125 ´ h / b_2) / 2.0$ = **472.9** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h ´ v_{sww2} ´ (1 / k_a + (0.04 \text{ in} / (A_e ´ F_{c\_perp})) ´ (b_2 / b_{2\_eff}))$ = **0.215** in

Deflection under capacity load; $\delta_{Cap} = 2 ´ v_{sww2} ´ h^3 / (3 ´ E ´ A_e ´ b_2) + v_{sww2} ´ h / (G_a) + h ´ \Delta_{a\_Cap} / b_2$ = **0.811** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / b_1 ´ (1 / k_a + (0.04 \text{ in} / (A_e ´ F_{c\_perp})) ´ (b_1 / b_{1\_eff}))$ = **0.093** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 ´ h^3 / (3 ´ E ´ A_e ´ b_1^2) + h / (G_a ´ b_1) + h ´ \Delta_{a1} / b_1)$ = **2.842** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} ´ k_1 / b_1$ = **472.94** plf

Segment 1 shear capacity; $v_{sww1} = v_w ´ (1.25 - 0.125 ´ h / b_1) / 2.0$ = **472.94** plf

v_{dsww1} / v_{sww1} = **1.000**

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Maximum shear force under wind loading; $V_{w\_max} = 0.6 ´ W$ = **1.38** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1},v_{dsww1}) ´ b_1 + v_{sww2} ´ b_2$ = **4.611** kips

V_{w_max} / V_w = **0.299**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b_1 = **2.277**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.38** kips

Axial force for maximum tension; $P = 0$ kips $=$ **0** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_1,k_2)) \times h / b_{1\_eff} - P =$ **1.702** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **126** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2

$f_t / F_t' =$ **0.135**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **1.38** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.271** kips

Maximum compressive force in chord; $C = V \times (k_1 / \text{sum}(k_1,k_2)) \times h / b_{1\_eff} + P =$ **1.973** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **120** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **653** lb/in^2

$f_c / F_c' =$ **0.183**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **425** lb/in^2

$f_c / F_{c\_perp}' =$ **0.281**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 3 and 4

Shear wall aspect ratio; $h / b_2 =$ **2.277**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.38** kips

Axial force for maximum tension; $P = 0$ kips $=$ **0** kips

Maximum tensile force in chord; $T = V \times (k_2 / \text{sum}(k_1,k_2)) \times h / b_{2\_eff} - P =$ **1.702** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **126** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2

$f_t / F_t' =$ **0.135**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **1.38** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.271** kips

Maximum compressive force in chord; $C = V \times (k_2 / \text{sum}(k_1,k_2)) \times h / b_{2\_eff} + P =$ **1.973** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **120** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **653** lb/in^2

$f_c / F_c' =$ **0.183**

PASS - Design compressive stress exceeds maximum applied compressive stress

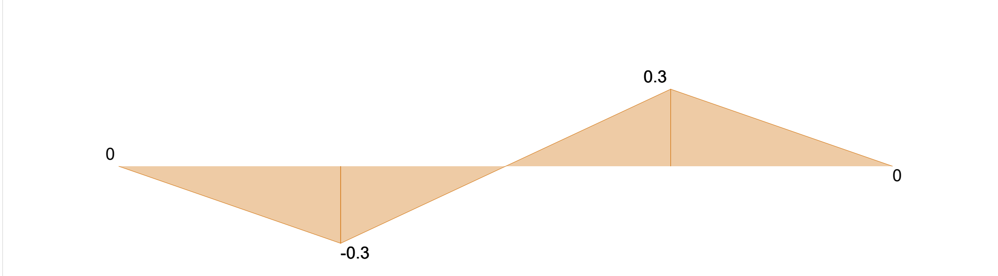
Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **425** lb/in^2

$f_c / F_{c\_perp}' =$ **0.281**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Collector capacity

Collector axial force diagram (kips)



Maximum shear force on wall; $V_{max} = V_{w\_max} = \mathbf{1.38}$ kips

Maximum force in collector; $P_{coll} = \mathbf{0.294}$ kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) = \mathbf{18}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{936}$ lb/in^2

$f_t / F_t' = \mathbf{0.019}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) = \mathbf{18}$ lb/in^2

Column stability factor; $C_P = \mathbf{1.00}$

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{2024}$ lb/in^2

$f_c / F_c' = \mathbf{0.009}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; $T_1 = \mathbf{1.702}$ kips

Chord 2; $T_2 = \mathbf{1.702}$ kips

Chord 3; $T_3 = \mathbf{1.702}$ kips

Chord 4; $T_4 = \mathbf{1.702}$ kips

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W = \mathbf{1.38}$ kips

Deflection limit; $\Delta_{w\_allow} = h / 400 = \mathbf{0.333}$ in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \mathrm{sum}(k_1,k_2)) / b_1 = \mathbf{141.54}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h \times b_1 / b_{1\_eff}) = \mathbf{1.702}$ kips

Chord compression force; $C_\delta = \max(0 \text{ kips}, v_{\delta w} \times h \times b_1 / b_{1\_eff}) = \mathbf{1.702}$ kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a = \mathbf{0.059}$ in

Vertical compression at chord; $\Delta_C = 0.04 \text{ in} \times C_\delta / (A_e \times F_{c\_perp}) = \mathbf{0.010}$ in

Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C) \times (b_1 / b_{1\_eff}) = \mathbf{0.075}$ in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1 = \mathbf{0.266}$ in

$\delta_{sww1} / \Delta_{w\_allow} = \mathbf{0.799}$

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear;	$v_{\delta w} = V_{\delta w} ´ (k_2 / sum(k_1,k_2)) / b_2 =$ **141.54** lb/ft
Anchor tension force;	$T_{\delta} = max(0\ kips, v_{\delta w} ´ h ´ b_2 / b_{2\_eff}) =$ **1.702** kips
Chord compression force;	$C_{\delta} = max(0\ kips, v_{\delta w} ´ h ´ b_2 / b_{2\_eff}) =$ **1.702** kips
Vertical elongation at anchor;	$\Delta_T = T_{\delta} / k_a =$ **0.059** in
Vertical compression at chord;	$\Delta_C = 0.04\ in ´ C_{\delta} / (A_e ´ F_{c\_perp}) =$ **0.010** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C) ´ (b_2 / b_{2\_eff}) =$ **0.075** in
Segment 2 deflection – Eqn. 4.3-1;	$\delta_{sww2} = 2 ´ v_{\delta w} ´ h^3 / (3 ´ E ´ A_e ´ b_2) + v_{\delta w} ´ h / (G_a) + h ´ \Delta_a / b_2 =$ **0.266** in
	$\delta_{sww2} / \Delta_{w\_allow} =$ **0.799**
	PASS - Shear wall deflection is less than deflection limit

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209 SE DOUGLAS
LEE'S SUMMIT, MO 64063
816.444.3144
WWW.LEOK.COM

NORTH SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

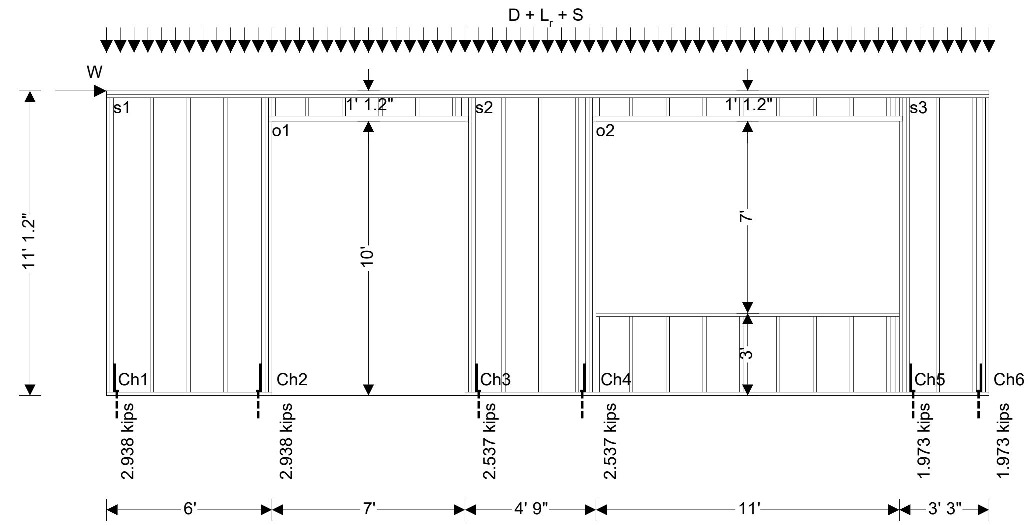
Tedds calculation version 1.2.10

Panel details

Structural wood panel sheathing on one side

Panel height; $h = \mathbf{11.1}$ ft

Panel length; $b = \mathbf{32}$ ft



Panel opening details

Width of opening; $w_{o1} = \mathbf{7}$ ft

Height of opening; $h_{o1} = \mathbf{10}$ ft

Height to underside of lintel over opening; $l_{o1} = \mathbf{10}$ ft

Position of opening; $P_{o1} = \mathbf{6}$ ft

Width of opening; $w_{o2} = \mathbf{11}$ ft

Height of opening; $h_{o2} = \mathbf{7}$ ft

Height to underside of lintel over opening; $l_{o2} = \mathbf{10}$ ft

Position of opening; $P_{o2} = \mathbf{17.75}$ ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} = \mathbf{208.2}\ ft^2$

Panel construction

Nominal stud size; 2" x 6"

Dressed stud size; 1.5" x 5.5"

Cross-sectional area of studs; $A_s = \mathbf{8.25}\ in^2$

Stud spacing; $s = \mathbf{16}$ in

Nominal end post size; 2 x 2" x 6"

Dressed end post size; 2 x 1.5" x 5.5"

Cross-sectional area of end posts; $A_e = \mathbf{16.5}\ in^2$

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; $A_{en} = \mathbf{13.5}\ in^2$

Nominal collector size; 2 x 2" x 6"

Dressed collector size;	2 x 1.5" x 5.5"
Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in
Vertical anchor stiffness;	k_a = **37700** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity;	G = **0.42**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	15/32" wood panel oriented strandboard sheathing
Fastener type;	10d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity;	v_n = 1290 plf ´ min[1 - (0.5 - G), 1] ´ 0.92 = **1091.9** lb/ft
Apparent shear wall shear stiffness;	G_a = **30** kips/in

Loading details

Dead load acting on top of panel;	D = **60** lb/ft
Roof live load acting on top of panel;	L_r = **60** lb/ft
Snow load acting on top of panel;	S = **60** lb/ft
Self weight of panel;	S_{wt} = **15** lb/ft^2
In plane wind load acting at head of panel;	W = **5000** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.30**
Size factor for compression – Table 4A;	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	
	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3;	C_{tt} = **1.00**

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Temperature factor for compression – Table 2.3.3

C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

C_{tE} = **1.00**

Incising factor – cl.4.3.8; C_i = **1.00**

Buckling stiffness factor – cl.4.4.2; C_T = **1.00**

Bearing area factor - cl. 3.10.4; C_b = **1.0**

Adjusted modulus of elasticity; $E_{min}' = E_{min}$ ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T = **510000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **715** psi

Reference compression design value; $F_c^* = F_c$ ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i = **2024** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.32**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **6** ft

Shear wall aspect ratio; h / b_1 = **1.85**

Segment 2 wall length; b_2 = **4.75** ft

Shear wall aspect ratio; h / b_2 = **2.337**

Segment 3 wall length; b_3 = **3.25** ft

Shear wall aspect ratio; h / b_3 = **3.415**

Segmented shear wall capacity - Equal deflection method

Effective segment length; $b_{1\_eff} = b_1 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **5.63** ft

Effective segment length; $b_{2\_eff} = b_2 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **4.38** ft

Effective segment length; $b_{3\_eff} = b_3 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **2.87** ft

Wind loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / b_1$ ´ $(1 / k_a + (0.04 \text{ in} / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_1 / b_{1\_eff}))$ = **0.060** in/kip

Segment 1 stiffness; $k_1 = 1 / (2$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1^2) + h / (G_a$ ´ $b_1) + h$ ´ $\Delta_{a1} / b_1)$ = **5.364** kips/in

Unit shear capacity, widest segment; $v_{sww1} = v_w / 2.0$ = **545.9** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h$ ´ v_{sww1} ´ $(1 / k_a + (0.04 \text{ in} / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_1 / b_{1\_eff}))$ = **0.198** in

Deflection under capacity load; $\delta_{Cap} = 2$ ´ v_{sww1} ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1) + v_{sww1}$ ´ $h / (G_a) + h$ ´ Δ_{a_Cap} / b_1 = **0.611** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / b_2$ ´ $(1 / k_a + (0.04 \text{ in} / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_2 / b_{2\_eff}))$ = **0.076** in/kip

Segment 2 stiffness; $k_2 = 1 / (2$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_2^2) + h / (G_a$ ´ $b_2) + h$ ´ $\Delta_{a1} / b_2)$ = **3.603** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap}$ ´ k_2 / b_2 = **463.19** plf

Segment 2 shear capacity; $v_{sww2} = v_w$ ´ $(1.25 - 0.125$ ´ $h / b_2) / 2.0$ = **522.94** plf

v_{dsww2} / v_{sww2} = **0.886**

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Segment 3 vertical unit deflection; $\Delta_{a1} = h / b_3$ ´ $(1 / k_a + (0.04 \text{ in} / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_3 / b_{3\_eff}))$ = **0.113** in/kip

Segment 3 stiffness; $k_3 = 1 / (2$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_3^2) + h / (G_a$ ´ $b_3) + h$ ´ $\Delta_{a1} / b_3)$ = **1.841** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 = \mathbf{345.83}$ plf

Segment 3 shear capacity; $v_{sww3} = v_w \times (1.25 - 0.125 \times h / b_3) / 2.0 = \mathbf{449.34}$ plf

$v_{dsww3} / v_{sww3} = \mathbf{0.770}$

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W = \mathbf{3}$ kips

Shear capacity for wind loading; $V_w = v_{sww1} \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + \min(v_{sww3},v_{dsww3}) \times b_3 = \mathbf{6.6}$ kips

$V_{w\_max} / V_w = \mathbf{0.455}$

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; $h / b_1 = \mathbf{1.85}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{3}$ kips

Axial force for maximum tension; $P = 0$ kips $= \mathbf{0}$ kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_{1\_eff} - P = \mathbf{2.938}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{218}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{936}$ lb/in^2

$f_t / F_t' = \mathbf{0.233}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W = \mathbf{3}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 = \mathbf{0.151}$ kips

Maximum compressive force in chord; $C = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_{1\_eff} + P = \mathbf{3.089}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{187}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{653}$ lb/in^2

$f_c / F_c' = \mathbf{0.287}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{425}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.441}$

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 3 and 4

Shear wall aspect ratio; $h / b_2 = \mathbf{2.337}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{3}$ kips

Axial force for maximum tension; $P = 0$ kips $= \mathbf{0}$ kips

Maximum tensile force in chord; $T = V \times (k_2 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_{2\_eff} - P = \mathbf{2.537}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{188}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{936}$ lb/in^2

$f_t / F_t' = \mathbf{0.201}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W = \mathbf{3}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 = \mathbf{0.151}$ kips

Maximum compressive force in chord; $C = V \times (k_2 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_{2\_eff} + P = \mathbf{2.688}$ kips

Maximum applied compressive stress; $f_c = C / A_e =$ **163** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **653** lb/in^2

$f_c / F_c' =$ **0.250**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **425** lb/in^2

$f_c / F_{c\_perp}' =$ **0.383**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 5 and 6

Shear wall aspect ratio; $h / b_3 =$ **3.415**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3** kips

Axial force for maximum tension; $P = 0$ kips $=$ **0** kips

Maximum tensile force in chord; $T = V \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_{3\_eff} - P =$ **1.973** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **146** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2

$f_t / F_t' =$ **0.156**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **3** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.151** kips

Maximum compressive force in chord; $C = V \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_{3\_eff} + P =$ **2.124** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **129** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **653** lb/in^2

$f_c / F_c' =$ **0.197**

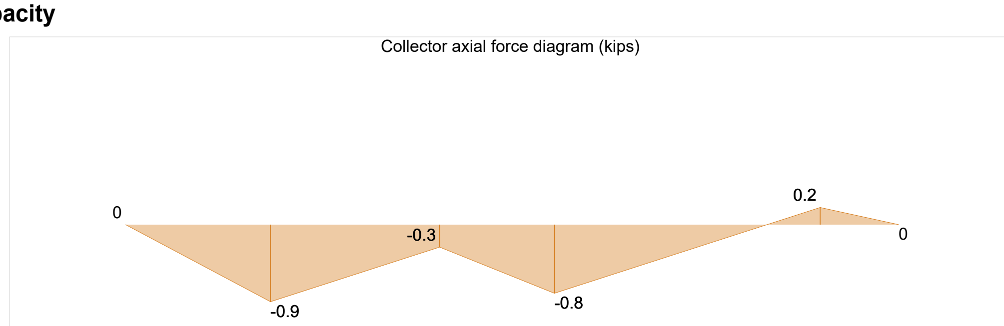
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **425** lb/in^2

$f_c / F_{c\_perp}' =$ **0.303**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Collector capacity



Maximum shear force on wall; $V_{max} = V_{w\_max} =$ **3** kips

Maximum force in collector; $P_{coll} =$ **0.926** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) =$ **56** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2

$f_t / F_t' =$ **0.060**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) =$ **56** lb/in^2

Column stability factor; $C_P =$ **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2024** lb/in^2

$f_c / F_c' =$ **0.028**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; $T_1 =$ **2.938** kips

Chord 2; $T_2 =$ **2.938** kips

Chord 3; $T_3 =$ **2.537** kips

Chord 4; $T_4 =$ **2.537** kips

Chord 5; $T_5 =$ **1.973** kips

Chord 6; $T_6 =$ **1.973** kips

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **3** kips

Deflection limit; $\Delta_{w\_allow} = h / 400 =$ **0.333** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) / b_1 =$ **248.16** lb/ft

Anchor tension force; $T_\delta = \max(0\text{ kips}, v_{\delta w} \times h \times b_1 / b_{1\_eff}) =$ **2.938** kips

Chord compression force; $C_\delta = \max(0\text{ kips}, v_{\delta w} \times h \times b_1 / b_{1\_eff}) =$ **2.938** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a =$ **0.078** in

Vertical compression at chord; $\Delta_C = 0.04\text{ in} \times C_\delta / (A_e \times F_{c\_perp}) =$ **0.017** in

Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C) \times (b_1 / b_{1\_eff}) =$ **0.101** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.298** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.896**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \mathrm{sum}(k_1,k_2,k_3)) / b_2 =$ **210.55** lb/ft

Anchor tension force; $T_\delta = \max(0\text{ kips}, v_{\delta w} \times h \times b_2 / b_{2\_eff}) =$ **2.537** kips

Chord compression force; $C_\delta = \max(0\text{ kips}, v_{\delta w} \times h \times b_2 / b_{2\_eff}) =$ **2.537** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a =$ **0.067** in

Vertical compression at chord; $\Delta_C = 0.04\text{ in} \times C_\delta / (A_e \times F_{c\_perp}) =$ **0.014** in

Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C) \times (b_2 / b_{2\_eff}) =$ **0.089** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.306** in

$\delta_{sww2} / \Delta_{w\_allow} =$ **0.92**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) / b_3 =$ **157.2** lb/ft

Anchor tension force; $T_\delta = \max(0\text{ kips}, v_{\delta w} \times h \times b_3 / b_{3\_eff}) =$ **1.973** kips

Chord compression force; $C_\delta = \max(0\text{ kips}, v_{\delta w} \times h \times b_3 / b_{3\_eff}) =$ **1.973** kips

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Vertical elongation at anchor;	$\Delta_T = T_\delta / k_a =$ **0.052** in
Vertical compression at chord;	$\Delta_C = 0.04 \text{ in } ´\ C_\delta / (A_e\ ´\ F_{c\_perp}) =$ **0.011** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C)\ ´\ (b_3 / b_{3\_eff}) =$ **0.072** in
Segment 3 deflection – Eqn. 4.3-1;	$\delta_{sww3} = 2\ ´\ v_{\delta w}\ ´\ h^3 / (3\ ´\ E\ ´\ A_e\ ´\ b_3) + v_{\delta w}\ ´\ h / (G_a) + h\ ´\ \Delta_a / b_3 =$ **0.327** in
	$\delta_{sww3} / \Delta_{w\_allow} =$ **0.981**

PASS - Shear wall deflection is less than deflection limit

SOUTH SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

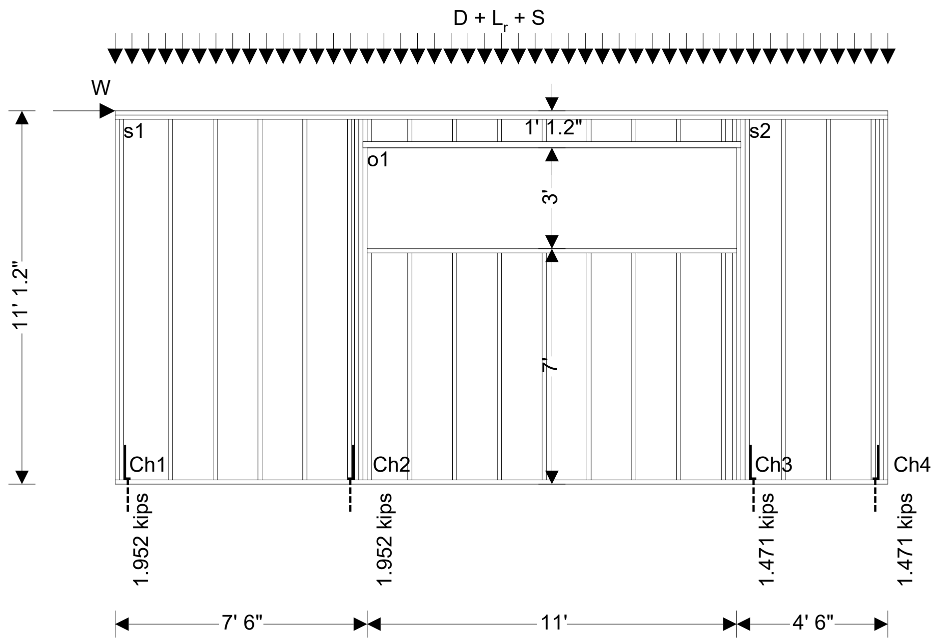
Tedds calculation version 1.2.10

Panel details

Structural wood panel sheathing on one side

Panel height; h = **11.1** ft

Panel length; b = **23** ft



Panel opening details

Width of opening; w_{o1} = **11** ft

Height of opening; h_{o1} = **3** ft

Height to underside of lintel over opening; l_{o1} = **10** ft

Position of opening; P_{o1} = **7.5** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1}$ = **222.3** ft^2

Panel construction

Nominal stud size; 2" x 6"

Dressed stud size; 1.5" x 5.5"

Cross-sectional area of studs; A_s = **8.25** in^2

Stud spacing; s = **16** in

Nominal end post size; 2 x 2" x 6"

Dressed end post size; 2 x 1.5" x 5.5"

Cross-sectional area of end posts; A_e = **16.5** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **13.5** in^2

Nominal collector size; 2 x 2" x 6"

Dressed collector size;	2 x 1.5" x 5.5"
Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in
Vertical anchor stiffness;	k_a = **28800** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity;	G = **0.42**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	15/32" wood panel oriented strandboard sheathing
Fastener type;	8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity;	v_n = 1065 plf ´ min[1 - (0.5 - G), 1] = **979.8** lb/ft
Apparent shear wall shear stiffness;	G_a = **19** kips/in

Loading details

Dead load acting on top of panel;	D = **60** lb/ft
Roof live load acting on top of panel;	L_r = **60** lb/ft
Snow load acting on top of panel;	S = **60** lb/ft
Self weight of panel;	S_{wt} = **15** lb/ft^2
In plane wind load acting at head of panel;	W = **3000** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.30**
Size factor for compression – Table 4A;	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3;	C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3	
	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	
	C_{tE} = **1.00**
Incising factor – cl.4.3.8;	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2;	C_T = **1.00**
Bearing area factor - cl. 3.10.4;	C_b = **1.0**
Adjusted modulus of elasticity;	$E_{min}' = E_{min}$ ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T = **510000** psi
Critical buckling design value;	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **715** psi
Reference compression design value;	$F_c^* = F_c$ ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i = **2024** psi
For sawn lumber;	c = **0.8**
Column stability factor – eqn.3.7-1;	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.32**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio;	3.5
Segment 1 wall length;	b_1 = **7.5** ft
Shear wall aspect ratio;	h / b_1 = **1.48**
Segment 2 wall length;	b_2 = **4.5** ft
Shear wall aspect ratio;	h / b_2 = **2.467**

Segmented shear wall capacity - Equal deflection method

Effective segment length;	$b_{1\_eff} = b_1 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **7.12** ft
Effective segment length;	$b_{2\_eff} = b_2 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **4.13** ft
Wind loading:	
Segment 1 vertical unit deflection;	$\Delta_{a1} = h / b_1$ ´ $(1 / k_a + (0.04\ in / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_1 / b_{1\_eff}))$ = **0.060** in/kip
Segment 1 stiffness;	$k_1 = 1 / (2$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1^2) + h / (G_a$ ´ $b_1) + h$ ´ $\Delta_{a1} / b_1)$ = **5.697** kips/in
Unit shear capacity, widest segment;	$v_{sww1} = v_w / 2.0$ = **489.9** plf
Vertical deflction under capacity load;	$\Delta_{a\_Cap} = h$ ´ v_{sww1} ´ $(1 / k_a + (0.04\ in / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_1 / b_{1\_eff}))$ = **0.221** in
Deflection under capacity load;	$\delta_{Cap} = 2$ ´ v_{sww1} ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1) + v_{sww1}$ ´ $h / (G_a) + h$ ´ Δ_{a_Cap} / b_1 = **0.645** in
Segment 2 vertical unit deflection;	$\Delta_{a1} = h / b_2$ ´ $(1 / k_a + (0.04\ in / (A_e$ ´ $F_{c\_perp}))$ ´ $(b_2 / b_{2\_eff}))$ = **0.101** in/kip
Segment 2 stiffness;	$k_2 = 1 / (2$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_2^2) + h / (G_a$ ´ $b_2) + h$ ´ $\Delta_{a1} / b_2)$ = **2.485** kips/in
Segment 2 unit shear at δ_{Cap};	$v_{dsww2} = \delta_{Cap}$ ´ k_2 / b_2 = **356.2** plf
Segment 2 shear capacity;	$v_{sww2} = v_w$ ´ $(1.25 - 0.125$ ´ $h / b_2) / 2.0$ = **461.32** plf
	v_{dsww2} / v_{sww2} = **0.772**
	PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}
Maximum shear force under wind loading;	$V_{w\_max} = 0.6$ ´ W = **1.8** kips
Shear capacity for wind loading;	$V_w = v_{sww1}$ ´ $b_1 + \min(v_{sww2}, v_{dsww2})$ ´ b_2 = **5.277** kips
	V_{w_max} / V_w = **0.341**
	PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio;	h / b_1 = **1.48**

Load combination 5

Shear force for maximum tension;	$V = 0.6 \times W =$ **1.8** kips
Axial force for maximum tension;	$P = 0$ kips = **0** kips
Maximum tensile force in chord;	$T = V \times (k_1 / sum(k_1,k_2)) \times h / b_{1\_eff} - P =$ **1.952** kips
Maximum applied tensile stress;	$f_t = T / A_{en} =$ **145** lb/in^2
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2
	$f_t / F_t' =$ **0.155**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression;	$V = 0.6 \times W =$ **1.8** kips
Axial force for maximum compression;	$P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.151** kips
Maximum compressive force in chord;	$C = V \times (k_1 / sum(k_1,k_2)) \times h / b_{1\_eff} + P =$ **2.103** kips
Maximum applied compressive stress;	$f_c = C / A_e =$ **127** lb/in^2
Design compressive stress;	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **653** lb/in^2
	$f_c / F_c' =$ **0.195**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate;	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **425** lb/in^2
	$f_c / F_{c\_perp}' =$ **0.300**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 3 and 4

Shear wall aspect ratio; $h / b_2 =$ **2.467**

Load combination 5

Shear force for maximum tension;	$V = 0.6 \times W =$ **1.8** kips
Axial force for maximum tension;	$P = 0$ kips = **0** kips
Maximum tensile force in chord;	$T = V \times (k_2 / sum(k_1,k_2)) \times h / b_{2\_eff} - P =$ **1.471** kips
Maximum applied tensile stress;	$f_t = T / A_{en} =$ **109** lb/in^2
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **936** lb/in^2
	$f_t / F_t' =$ **0.116**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression;	$V = 0.6 \times W =$ **1.8** kips
Axial force for maximum compression;	$P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.151** kips
Maximum compressive force in chord;	$C = V \times (k_2 / sum(k_1,k_2)) \times h / b_{2\_eff} + P =$ **1.622** kips
Maximum applied compressive stress;	$f_c = C / A_e =$ **98** lb/in^2
Design compressive stress;	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **653** lb/in^2
	$f_c / F_c' =$ **0.151**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate;	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **425** lb/in^2
	$f_c / F_{c\_perp}' =$ **0.231**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

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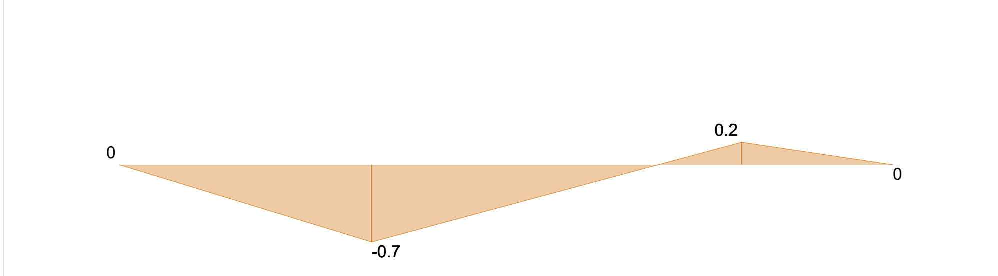
Collector capacity

Collector axial force diagram (kips)



Maximum shear force on wall; $V_{max} = V_{w\_max} =$ **1.8** kips

Maximum force in collector; $P_{coll} =$ **0.666** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) =$ **40** lb/in^2

Design tensile stress; $F_t' = F_t$ ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ $C_i =$ **936** lb/in^2

$f_t / F_t' =$ **0.043**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) =$ **40** lb/in^2

Column stability factor; $C_P =$ **1.00**

Design compressive stress; $F_c' = F_c$ ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ $C_P =$ **2024** lb/in^2

$f_c / F_c' =$ **0.020**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; $T_1 =$ **1.952** kips

Chord 2; $T_2 =$ **1.952** kips

Chord 3; $T_3 =$ **1.471** kips

Chord 4; $T_4 =$ **1.471** kips

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv}$ ´ $W =$ **1.8** kips

Deflection limit; $\Delta_{w\_allow} = h / 400 =$ **0.333** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w}$ ´ $(k_1 / \mathrm{sum}(k_1,k_2)) / b_1 =$ **167.1** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w}$ ´ h ´ $b_1 / b_{1\_eff}) =$ **1.952** kips

Chord compression force; $C_\delta = \max(0 \text{ kips}, v_{\delta w}$ ´ h ´ $b_1 / b_{1\_eff}) =$ **1.952** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a =$ **0.068** in

Vertical compression at chord; $\Delta_C = 0.04 \text{ in}$ ´ $C_\delta / (A_e$ ´ $F_{c\_perp}) =$ **0.011** in

Total vertical deflection; $\Delta_a = (\Delta_T + \Delta_C)$ ´ $(b_1 / b_{1\_eff}) =$ **0.083** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2$ ´ $v_{\delta w}$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_1) + v_{\delta w}$ ´ $h / (G_a) + h$ ´ $\Delta_a / b_1 =$ **0.231** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.694**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear;	$v_{\delta w} = V_{\delta w}$ ´ $(k_2 / sum(k_1,k_2)) / b_2$ = **121.5** lb/ft
Anchor tension force;	$T_{\delta} = max(0\ kips, v_{\delta w}$ ´ h ´ $b_2 / b_{2\_eff})$ = **1.471** kips
Chord compression force;	$C_{\delta} = max(0\ kips, v_{\delta w}$ ´ h ´ $b_2 / b_{2\_eff})$ = **1.471** kips
Vertical elongation at anchor;	$\Delta_T = T_{\delta} / k_a$ = **0.051** in
Vertical compression at chord;	Δ_C = 0.04 in ´ $C_{\delta} / (A_e$ ´ $F_{c\_perp})$ = **0.008** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C)$ ´ (b_2 / b_{2_eff}) = **0.065** in
Segment 2 deflection – Eqn. 4.3-1;	$\delta_{sww2} = 2$ ´ $v_{\delta w}$ ´ $h^3 / (3$ ´ E ´ A_e ´ $b_2) + v_{\delta w}$ ´ $h / (G_a) + h$ ´ Δ_a / b_2 = **0.244** in
	$\delta_{sww2} / \Delta_{w\_allow}$ = **0.732**

PASS - Shear wall deflection is less than deflection limit

EXTERIOR 9' SHEAR WALL DESIGN (NDS)

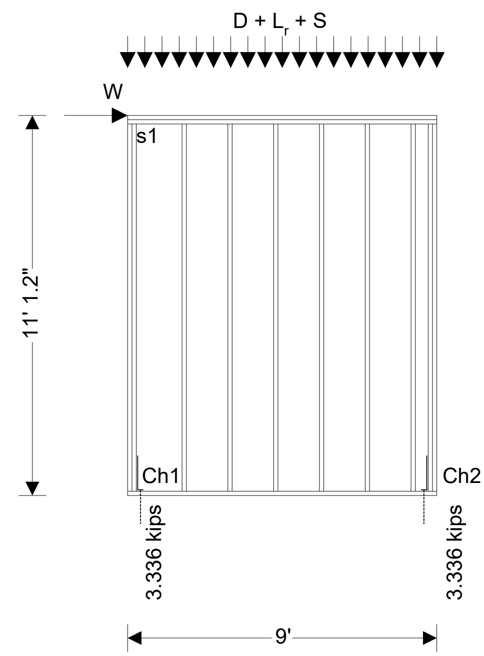
In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Panel details

Structural wood panel sheathing on one side

Panel height;	h = **11.1** ft
Panel length;	b = **9** ft
Total area of wall;	A = h ′ b = **99.9** ft^2



Panel construction

Nominal stud size;	2" x 6"
Dressed stud size;	1.5" x 5.5"
Cross-sectional area of studs;	A_s = **8.25** in^2
Stud spacing;	s = **16** in
Nominal end post size;	2 x 2" x 6"
Dressed end post size;	2 x 1.5" x 5.5"
Cross-sectional area of end posts;	A_e = **16.5** in^2
Hole diameter;	Dia = **1** in
Net cross-sectional area of end posts;	A_{en} = **13.5** in^2
Nominal collector size;	2 x 2" x 6"
Dressed collector size;	2 x 1.5" x 5.5"
Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in

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Vertical anchor stiffness;	k_a = **37700** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification;	Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity;	G = **0.42**
Tension parallel to grain;	F_t = **450** lb/in^2
Compression parallel to grain;	F_c = **1150** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **425** lb/in^2
Modulus of elasticity;	E = **1400000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material;	15/32" wood panel oriented strandboard sheathing
Fastener type;	10d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity;	v_n = 1290 plf ´ min[1 - (0.5 - G), 1] ´ 0.92 = **1091.9** lb/ft
Apparent shear wall shear stiffness;	G_a = **30** kips/in

Loading details

Dead load acting on top of panel;	D = **60** lb/ft
Roof live load acting on top of panel;	L_r = **60** lb/ft
Snow load acting on top of panel;	S = **60** lb/ft
Self weight of panel;	S_{wt} = **15** lb/ft^2
In plane wind load acting at head of panel;	W = **4320** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.30**
Size factor for compression – Table 4A;	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3;	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**
Incising factor – cl.4.3.8;	C_i = **1.00**

Buckling stiffness factor – cl.4.4.2;	C_T = **1.00**
Bearing area factor - cl. 3.10.4;	C_b = **1.0**
Adjusted modulus of elasticity;	$E_{min}' = E_{min} ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T$ = **510000** psi
Critical buckling design value;	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **715** psi
Reference compression design value;	$F_c^* = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i$ = **2024** psi
For sawn lumber;	c = **0.8**
Column stability factor – eqn.3.7-1;	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.32**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio;	3.5
Shear wall length;	b = **9** ft
Shear wall aspect ratio;	h / b = **1.233**

Segmented shear wall capacity

Maximum shear force under wind loading;	$V_{w\_max} = 0.6 ´ W$ = **2.592** kips
Shear capacity for wind loading;	$V_w = v_w ´ b / 2.0$ = **4.913** kips
	V_{w_max} / V_w = **0.528**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio;	h / b = **1.233**
Effective length for chord forces;	$b_{eff} = b - 3 / 2 ´ b_{EndPost} - e_{anchor}$ = **8.63** ft
Load combination 5	
Shear force for maximum tension;	$V = 0.6 ´ W$ = **2.592** kips
Axial force for maximum tension;	P = 0 kips = **0** kips
Maximum tensile force in chord;	$T = V ´ h / b_{eff} - P$ = **3.336** kips
Maximum applied tensile stress;	$f_t = T / A_{en}$ = **247** lb/in^2
Design tensile stress;	$F_t' = F_t ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i$ = **936** lb/in^2
	f_t / F_t' = **0.264**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1	
Shear force for maximum compression;	$V = 0.6 ´ W$ = **2.592** kips
Axial force for maximum compression;	$P = ((D + S_{wt} ´ h)) ´ s / 2$ = **0.151** kips
Maximum compressive force in chord;	$C = V ´ h / b_{eff} + P$ = **3.487** kips
Maximum applied compressive stress;	$f_c = C / A_e$ = **211** lb/in^2
Design compressive stress;	$F_c' = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P$ = **653** lb/in^2
	f_c / F_c' = **0.324**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate;	$F_{c\_perp}' = F_{c\_perp} ´ C_{Mc} ´ C_{tc} ´ C_i ´ C_b$ = **425** lb/in^2
	f_c / F_{c_perp}' = **0.497**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1;	T_1 = **3.336** kips
Chord 2;	T_2 = **3.336** kips

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Wind load deflection

Design shear force;	$V_{\delta w} = f_{Wserv} \times W =$ **2.592** kips
Deflection limit;	$\Delta_{w\_allow} = h / 400 =$ **0.333** in
Induced unit shear;	$v_{\delta w} = V_{\delta w} / b =$ **288** lb/ft
Anchor tension force;	$T_{\delta} = \max(0\text{ kips}, v_{\delta w} \times h \times b / b_{eff}) =$ **3.336** kips
Chord compression force;	$C_{\delta} = \max(0\text{ kips}, v_{\delta w} \times h \times b / b_{eff}) =$ **3.336** kips
Vertical elongation at anchor;	$\Delta_T = T_{\delta} / k_a =$ **0.088** in
Vertical compression at chord;	$\Delta_C = 0.04\text{ in} \times C_{\delta} / (A_e \times F_{c\_perp}) =$ **0.019** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C) \times (b / b_{eff}) =$ **0.112** in
Shear wall deflection – Eqn. 4.3-1;	$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.26** in
	$\delta_{sww} / \Delta_{w\_allow} =$ **0.781**

PASS - Shear wall deflection is less than deflection limit

NORTH INTERIOR SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

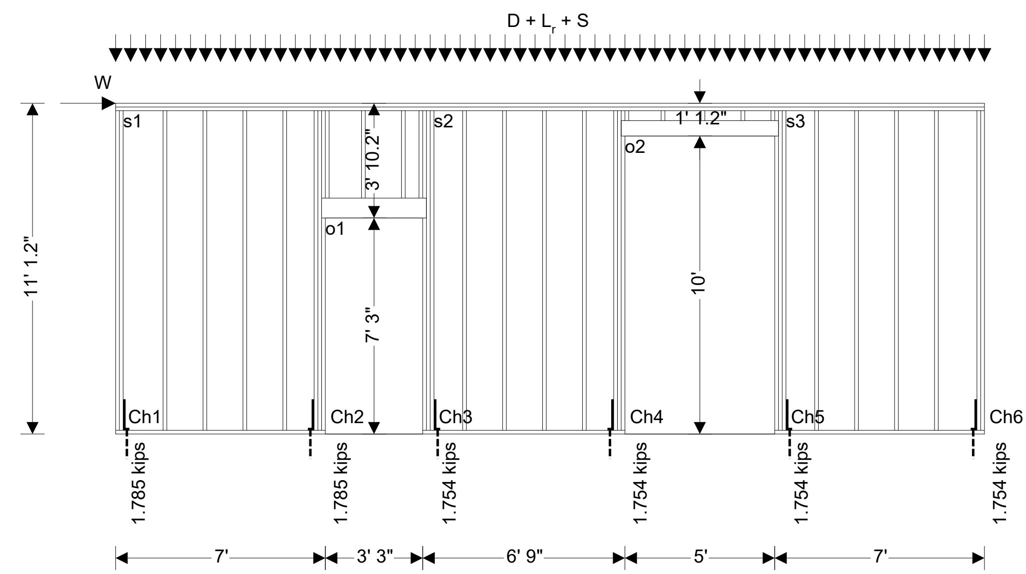
Tedds calculation version 1.2.10

Panel details

gypsum wallboard sheathing on both sides

Panel height; $h = \mathbf{11.1}$ ft

Panel length; $b = \mathbf{29}$ ft



Panel opening details

Width of opening; $w_{o1} = \mathbf{3.25}$ ft

Height of opening; $h_{o1} = \mathbf{7.25}$ ft

Height to underside of lintel over opening; $l_{o1} = \mathbf{7.25}$ ft

Position of opening; $P_{o1} = \mathbf{7}$ ft

Width of opening; $w_{o2} = \mathbf{5}$ ft

Height of opening; $h_{o2} = \mathbf{10}$ ft

Height to underside of lintel over opening; $l_{o2} = \mathbf{10}$ ft

Position of opening; $P_{o2} = \mathbf{17}$ ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} = \mathbf{248.337}$ ft^2

Panel construction

Nominal stud size; 2" x 4"

Dressed stud size; 1.5" x 3.5"

Cross-sectional area of studs; $A_s = \mathbf{5.25}$ in^2

Stud spacing; $s = \mathbf{16}$ in

Nominal end post size; 2 x 2" x 4"

Dressed end post size; 2 x 1.5" x 3.5"

Cross-sectional area of end posts; $A_e = \mathbf{10.5}$ in^2

Hole diameter; $Dia = \mathbf{1}$ in

Net cross-sectional area of end posts; A_{en} = **7.5** in^2
Nominal collector size; 2 x 2" x 4"
Dressed collector size; 2 x 1.5" x 3.5"
Service condition; Dry
Temperature; 100 degF or less
Anchor location; Inside face
Anchor offset; e_{anchor} = **0** in
Vertical anchor stiffness; k_a = **25000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)
Species, grade and size classification; Spruce-Pine-Fir, no.2 grade, 2" & wider
Specific gravity; G = **0.42**
Tension parallel to grain; F_t = **450** lb/in^2
Compression parallel to grain; F_c = **1150** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **425** lb/in^2
Modulus of elasticity; E = **1400000** lb/in^2
Minimum modulus of elasticity; E_{min} = **510000** lb/in^2

Sheathing details
Sheathing material; 1/2" gypsum wallboard with blocking
Fastener type; 5d wallboard nails at 7"centers

From SDPWS Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Gypsum and Portland Cement Plaster
Nominal unit shear capacity; v_{n1} = **250** lb/ft
Apparent shear wall shear stiffness; G_{a1} = **6.5** kips/in

Sheathing details
Sheathing material; 1/2" gypsum wallboard with blocking
Fastener type; 5d wallboard nails at 7"centers

From SDPWS Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Gypsum and Portland Cement Plaster
Nominal unit shear capacity; v_{n2} = **250** lb/ft
Apparent shear wall shear stiffness; G_{a2} = **6.5** kips/in

Combined unit shear capacities (cl. 4.3.5.4.1)
Combined apparent shear wall shear stiffness; $G_{ac} = G_{a1} + G_{a2}$ = **13** kips/in
Minimum shear capacity to stiffness ratio; $K_{min} = \min(v_{n1} / G_{a1}, v_{n2} / G_{a2})$ = **0.003**
Combined nominal unit shear capacity; $v_{nc} = K_{min}$ ´ G_{ac} = **500** lb/ft
Combined nominal unit shear capacity for seismic design
$v_{sc} = v_{nc}$ = **500** lb/ft
Combined nominal unit shear capacity for wind design
$v_{wc} = v_{nc}$ = **500** lb/ft

Loading details
Dead load acting on top of panel; D = **50** lb/ft
Roof live load acting on top of panel; L_r = **50** lb/ft
Snow load acting on top of panel; S = **60** lb/ft
Self weight of panel; S_{wt} = **15** lb/ft^2

In plane wind load acting at head of panel;	W = **5200** lbs
Wind load serviceability factor;	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1;	D + 0.6W
Load combination no.2;	D + 0.7E
Load combination no.3;	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4;	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5;	0.6D + 0.6W
Load combination no.6;	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2;	C_D = **1.60**
Size factor for tension – Table 4A;	C_{Ft} = **1.50**
Size factor for compression – Table 4A;	C_{Fc} = **1.15**
Wet service factor for tension – Table 4A;	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A;	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	
	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3;	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	
	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	
	C_{tE} = **1.00**
Incising factor – cl.4.3.8;	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2;	C_T = **1.00**
Bearing area factor - cl. 3.10.4;	C_b = **1.0**
Adjusted modulus of elasticity;	$E_{min}' = E_{min}$ ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T = **510000** psi
Critical buckling design value;	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **289** psi
Reference compression design value;	$F_c^* = F_c$ ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i = **2116** psi
For sawn lumber;	c = **0.8**
Column stability factor – eqn.3.7-1;	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.13**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio;	2
Segment 1 wall length;	b_1 = **7** ft
Shear wall aspect ratio;	h / b_1 = **1.586**
Segment 2 wall length;	b_2 = **6.75** ft
Shear wall aspect ratio;	h / b_2 = **1.644**
Segment 3 wall length;	b_3 = **7** ft
Shear wall aspect ratio;	h / b_3 = **1.586**

Segmented shear wall capacity - Equal deflection method

Effective segment length;	$b_{1\_eff} = b_1 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **6.62** ft
Effective segment length;	$b_{2\_eff} = b_2 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **6.38** ft
Effective segment length;	$b_{3\_eff} = b_3 - 3 / 2$ ´ $b_{EndPost} - e_{anchor}$ = **6.62** ft
Wind loading:	

Segment 3 vertical unit deflection; $\Delta_{a1} = h / b_3 \times (1 / k_a + (0.04 \text{ in} / (A_e \times F_{c\_perp})) \times (b_3 / b_{3\_eff})) =$ **0.078** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_{ac} \times b_3) + h \times \Delta_{a1} / b_3) =$ **3.823** kips/in

Unit shear capacity, widest segment; $v_{sww3} = v_{wc} / 2.0 =$ **250** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww3} \times (1 / k_a + (0.04 \text{ in} / (A_e \times F_{c\_perp})) \times (b_3 / b_{3\_eff})) =$ **0.137** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww3} \times h^3 / (3 \times E \times A_e \times b_3) + v_{sww3} \times h / (G_{ac}) + h \times \Delta_{a\_Cap} / b_3 =$ **0.458** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / b_1 \times (1 / k_a + (0.04 \text{ in} / (A_e \times F_{c\_perp})) \times (b_1 / b_{1\_eff})) =$ **0.078** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_{ac} \times b_1) + h \times \Delta_{a1} / b_1) =$ **3.823** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1 =$ **250** plf

Segment 1 shear capacity; $v_{sww1} = v_{wc} / 2.0 =$ **250** plf

$v_{dsww1} / v_{sww1} =$ **1.000**

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Segment 2 vertical unit deflection; $\Delta_{a1} = h / b_2 \times (1 / k_a + (0.04 \text{ in} / (A_e \times F_{c\_perp})) \times (b_2 / b_{2\_eff})) =$ **0.081** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_{ac} \times b_2) + h \times \Delta_{a1} / b_2) =$ **3.614** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2 =$ **245.11** plf

Segment 2 shear capacity; $v_{sww2} = v_{wc} / 2.0 =$ **250** plf

$v_{dsww2} / v_{sww2} =$ **0.980**

PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W =$ **3.12** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1}, v_{dsww1}) \times b_1 + \min(v_{sww2}, v_{dsww2}) \times b_2 + v_{sww3} \times b_3 =$ **5.154** kips

$V_{w\_max} / V_w =$ **0.605**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; $h / b_1 =$ **1.586**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.12** kips

Axial force for maximum tension; $P = 0$ kips $=$ **0** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_1,k_2,k_3)) \times h / b_{1\_eff} - P =$ **1.785** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **238** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1080** lb/in^2

$f_t / F_t' =$ **0.220**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **3.12** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.144** kips

Maximum compressive force in chord; $C = V \times (k_1 / \text{sum}(k_1,k_2,k_3)) \times h / b_{1\_eff} + P =$ **1.930** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **184** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **281** lb/in^2

$f_c / F_c' =$ **0.654**

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PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; F_{c_perp}' = F_{c_perp} ´ C_{Mc} ´ C_{tc} ´ C_i ´ C_b = **425** lb/in^2

f_c / F_{c_perp}' = **0.432**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 3 and 4

Shear wall aspect ratio; h / b_2 = **1.644**

Load combination 5

Shear force for maximum tension; V = 0.6 ´ W = **3.12** kips

Axial force for maximum tension; P = 0 kips = **0** kips

Maximum tensile force in chord; T = V ´ (k_2 / sum(k_1,k_2,k_3)) ´ h / b_{2_eff} - P = **1.754** kips

Maximum applied tensile stress; f_t = T / A_{en} = **234** lb/in^2

Design tensile stress; F_t' = F_t ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i = **1080** lb/in^2

f_t / F_t' = **0.217**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 ´ W = **3.12** kips

Axial force for maximum compression; P = ((D + S_{wt} ´ h)) ´ s / 2 = **0.144** kips

Maximum compressive force in chord; C = V ´ (k_2 / sum(k_1,k_2,k_3)) ´ h / b_{2_eff} + P = **1.898** kips

Maximum applied compressive stress; f_c = C / A_e = **181** lb/in^2

Design compressive stress; F_c' = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P = **281** lb/in^2

f_c / F_c' = **0.644**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; F_{c_perp}' = F_{c_perp} ´ C_{Mc} ´ C_{tc} ´ C_i ´ C_b = **425** lb/in^2

f_c / F_{c_perp}' = **0.425**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chords 5 and 6

Shear wall aspect ratio; h / b_3 = **1.586**

Load combination 5

Shear force for maximum tension; V = 0.6 ´ W = **3.12** kips

Axial force for maximum tension; P = 0 kips = **0** kips

Maximum tensile force in chord; T = V ´ (k_3 / sum(k_1,k_2,k_3)) ´ h / b_{3_eff} - P = **1.754** kips

Maximum applied tensile stress; f_t = T / A_{en} = **234** lb/in^2

Design tensile stress; F_t' = F_t ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i = **1080** lb/in^2

f_t / F_t' = **0.217**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 ´ W = **3.12** kips

Axial force for maximum compression; P = ((D + S_{wt} ´ h)) ´ s / 2 = **0.144** kips

Maximum compressive force in chord; C = V ´ (k_3 / sum(k_1,k_2,k_3)) ´ h / b_{3_eff} + P = **1.898** kips

Maximum applied compressive stress; f_c = C / A_e = **181** lb/in^2

Design compressive stress; F_c' = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P = **281** lb/in^2

f_c / F_c' = **0.644**

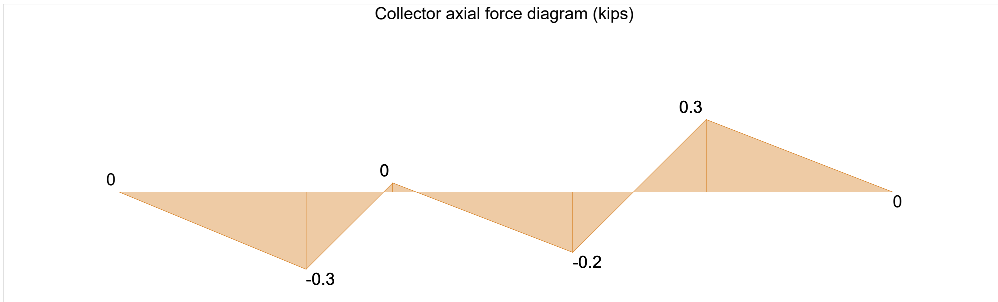
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp}$ ´ C_{Mc} ´ C_{tc} ´ C_i ´ C_b = **425** lb/in^2

f_c / F_{c_perp}' = **0.425**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Collector capacity



Maximum shear force on wall; $V_{max} = V_{w\_max}$ = **3.12** kips

Maximum force in collector; P_{coll} = **0.313** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s)$ = **30** lb/in^2

Design tensile stress; $F_t' = F_t$ ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i = **1080** lb/in^2

f_t / F_t' = **0.028**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s)$ = **30** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c$ ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P = **2116** lb/in^2

f_c / F_c' = **0.014**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; T_1 = **1.785** kips

Chord 2; T_2 = **1.785** kips

Chord 3; T_3 = **1.754** kips

Chord 4; T_4 = **1.754** kips

Chord 5; T_5 = **1.754** kips

Chord 6; T_6 = **1.754** kips

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv}$ ´ W = **3.12** kips

Deflection limit; $\Delta_{w\_allow} = h / 400$ = **0.333** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w}$ ´ $(k_1 / \text{sum}(k_1,k_2,k_3)) / b_1$ = **152.23** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w}$ ´ h ´ $b_1 / b_{1\_eff})$ = **1.785** kips

Chord compression force; $C_\delta = \max(0 \text{ kips}, v_{\delta w}$ ´ h ´ $b_1 / b_{1\_eff})$ = **1.785** kips

Vertical elongation at anchor; $\Delta_T = T_\delta / k_a$ = **0.071** in

Vertical compression at chord; $\Delta_C = 0.04 \text{ in}$ ´ $C_\delta / (A_e$ ´ $F_{c\_perp})$ = **0.016** in

Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C)$ ´ (b_1 / b_{1_eff}) = **0.092** in
Segment 1 deflection – Eqn. 4.3-1;	δ_{sww1} = 2 ´ $v_{\delta w}$ ´ h^3 / (3 ´ E ´ A_e ´ b_1) + $v_{\delta w}$ ´ h / (G_{ac}) + h ´ Δ_a / b_1 = **0.293** in
	$\delta_{sww1} / \Delta_{w\_allow}$ = **0.879**

PASS - Shear wall deflection is less than deflection limit

Segment 2	
Induced unit shear;	$v_{\delta w} = V_{\delta w}$ ´ (k_2 / sum(k_1,k_2,k_3)) / b_2 = **149.25** lb/ft
Anchor tension force;	T_δ = max(0 kips,$v_{\delta w}$ ´ h ´ b_2 / b_{2_eff}) = **1.754** kips
Chord compression force;	C_δ = max(0 kips,$v_{\delta w}$ ´ h ´ b_2 / b_{2_eff}) = **1.754** kips
Vertical elongation at anchor;	$\Delta_T = T_\delta / k_a$ = **0.070** in
Vertical compression at chord;	Δ_C = 0.04 in ´ C_δ / (A_e ´ F_{c_perp}) = **0.016** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C)$ ´ (b_2 / b_{2_eff}) = **0.091** in
Segment 2 deflection – Eqn. 4.3-1;	δ_{sww2} = 2 ´ $v_{\delta w}$ ´ h^3 / (3 ´ E ´ A_e ´ b_2) + $v_{\delta w}$ ´ h / (G_{ac}) + h ´ Δ_a / b_2 = **0.293** in
	$\delta_{sww2} / \Delta_{w\_allow}$ = **0.881**

PASS - Shear wall deflection is less than deflection limit

Segment 3	
Induced unit shear;	$v_{\delta w} = V_{\delta w}$ ´ (k_3 / sum(k_1,k_2,k_3)) / b_3 = **149.57** lb/ft
Anchor tension force;	T_δ = max(0 kips,$v_{\delta w}$ ´ h ´ b_3 / b_{3_eff}) = **1.754** kips
Chord compression force;	C_δ = max(0 kips,$v_{\delta w}$ ´ h ´ b_3 / b_{3_eff}) = **1.754** kips
Vertical elongation at anchor;	$\Delta_T = T_\delta / k_a$ = **0.070** in
Vertical compression at chord;	Δ_C = 0.04 in ´ C_δ / (A_e ´ F_{c_perp}) = **0.016** in
Total vertical deflection;	$\Delta_a = (\Delta_T + \Delta_C)$ ´ (b_3 / b_{3_eff}) = **0.091** in
Segment 3 deflection – Eqn. 4.3-1;	δ_{sww3} = 2 ´ $v_{\delta w}$ ´ h^3 / (3 ´ E ´ A_e ´ b_3) + $v_{\delta w}$ ´ h / (G_{ac}) + h ´ Δ_a / b_3 = **0.288** in
	$\delta_{sww3} / \Delta_{w\_allow}$ = **0.863**

PASS - Shear wall deflection is less than deflection limit

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SOUTH INTERIOR SHEAR WALL DESIGN (NDS)

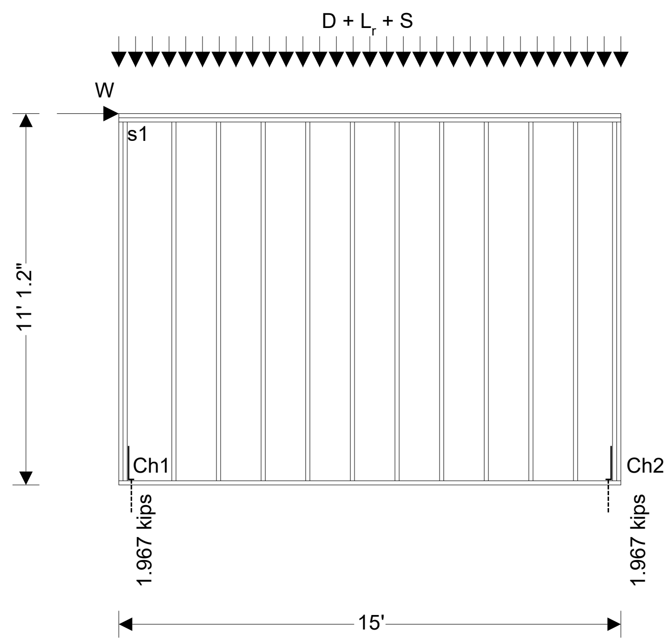
In accordance with NDS2018 and SDPWS2021 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Panel details

gypsum wallboard sheathing on both sides

Panel height;	h = **11.1** ft
Panel length;	b = **15** ft
Total area of wall;	A = h ´ b = **166.5** ft^2



Panel construction

Nominal stud size;	2" x 4"
Dressed stud size;	1.5" x 3.5"
Cross-sectional area of studs;	A_s = **5.25** in^2
Stud spacing;	s = **16** in
Nominal end post size;	2 x 2" x 4"
Dressed end post size;	2 x 1.5" x 3.5"
Cross-sectional area of end posts;	A_e = **10.5** in^2
Hole diameter;	Dia = **1** in
Net cross-sectional area of end posts;	A_{en} = **7.5** in^2
Nominal collector size;	2 x 2" x 4"
Dressed collector size;	2 x 1.5" x 3.5"
Service condition;	Dry
Temperature;	100 degF or less
Anchor location;	Inside face
Anchor offset;	e_{anchor} = **0** in
Vertical anchor stiffness;	k_a = **25000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification; Spruce-Pine-Fir, no.2 grade, 2" & wider

Specific gravity; G = **0.42**

Tension parallel to grain; F_t = **450** lb/in^2

Compression parallel to grain; F_c = **1150** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **425** lb/in^2

Modulus of elasticity; E = **1400000** lb/in^2

Minimum modulus of elasticity; E_{min} = **510000** lb/in^2

Sheathing details

Sheathing material; 1/2" gypsum wallboard with blocking

Fastener type; 5d wallboard nails at 7"centers

From SDPWS Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Gypsum and Portland Cement Plaster

Nominal unit shear capacity; v_{n1} = **250** lb/ft

Apparent shear wall shear stiffness; G_{a1} = **6.5** kips/in

Sheathing details

Sheathing material; 1/2" gypsum wallboard with blocking

Fastener type; 5d wallboard nails at 7"centers

From SDPWS Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Gypsum and Portland Cement Plaster

Nominal unit shear capacity; v_{n2} = **250** lb/ft

Apparent shear wall shear stiffness; G_{a2} = **6.5** kips/in

Combined unit shear capacities (cl. 4.3.5.4.1)

Combined apparent shear wall shear stiffness; $G_{ac} = G_{a1} + G_{a2}$ = **13** kips/in

Minimum shear capacity to stiffness ratio; $K_{min} = \min(v_{n1} / G_{a1}, v_{n2} / G_{a2})$ = **0.003**

Combined nominal unit shear capacity; $v_{nc} = K_{min} ´ G_{ac}$ = **500** lb/ft

Combined nominal unit shear capacity for seismic design

$v_{sc} = v_{nc}$ = **500** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = v_{nc}$ = **500** lb/ft

Loading details

Dead load acting on top of panel; D = **50** lb/ft

Roof live load acting on top of panel; L_r = **50** lb/ft

Snow load acting on top of panel; S = **60** lb/ft

Self weight of panel; S_{wt} = **15** lb/ft^2

In plane wind load acting at head of panel; W = **4320** lbs

Wind load serviceability factor; f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1; D + 0.6W

Load combination no.2; D + 0.7E

Load combination no.3; $D + 0.45W + 0.75L_f + 0.75(L_r \text{ or } S \text{ or } R)$

Load combination no.4; $D + 0.525E + 0.75L_f + 0.75S$

Load combination no.5; 0.6D + 0.6W

Load combination no.6; 0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2; C_D = **1.60**

Size factor for tension – Table 4A; C_{Ft} = **1.50**

Size factor for compression – Table 4A; C_{Fc} = **1.15**

Wet service factor for tension – Table 4A; C_{Mt} = **1.00**

Wet service factor for compression – Table 4A; C_{Mc} = **1.00**

Wet service factor for modulus of elasticity – Table 4A
C_{ME} = **1.00**

Temperature factor for tension – Table 2.3.3; C_{tt} = **1.00**

Temperature factor for compression – Table 2.3.3
C_{tc} = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3
C_{tE} = **1.00**

Incising factor – cl.4.3.8; C_i = **1.00**

Buckling stiffness factor – cl.4.4.2; C_T = **1.00**

Bearing area factor - cl. 3.10.4; C_b = **1.0**

Adjusted modulus of elasticity; E_{min}' = E_{min} ´ C_{ME} ´ C_{tE} ´ C_i ´ C_T = **510000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **289** psi

Reference compression design value; F_c^* = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i = **2116** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.13**

From SDPWS Table 4.3.3 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 2

Shear wall length; b = **15** ft

Shear wall aspect ratio; h / b = **0.74**

Segmented shear wall capacity

Maximum shear force under wind loading; V_{w_max} = 0.6 ´ W = **2.592** kips

Shear capacity for wind loading; V_w = v_{wc} ´ b / 2.0 = **3.75** kips

V_{w_max} / V_w = **0.691**

PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b = **0.74**

Effective length for chord forces; b_{eff} = b - 3 / 2 ´ $b_{EndPost}$ - e_{anchor} = **14.63** ft

Load combination 5

Shear force for maximum tension; V = 0.6 ´ W = **2.592** kips

Axial force for maximum tension; P = 0 kips = **0** kips

Maximum tensile force in chord; T = V ´ h / b_{eff} - P = **1.967** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **262** lb/in^2

Design tensile stress; F_t' = F_t ´ C_D ´ C_{Mt} ´ C_{tt} ´ C_{Ft} ´ C_i = **1080** lb/in^2

f_t / F_t' = **0.243**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression;	V = 0.6 ´ W = **2.592** kips
Axial force for maximum compression;	P = ((D + S_{wt} ´ h)) ´ s / 2 = **0.144** kips
Maximum compressive force in chord;	C = V ´ h / b_{eff} + P = **2.112** kips
Maximum applied compressive stress;	f_c = C / A_e = **201** lb/in^2
Design compressive stress;	F_c' = F_c ´ C_D ´ C_{Mc} ´ C_{tc} ´ C_{Fc} ´ C_i ´ C_P = **281** lb/in^2
	f_c / F_c' = **0.716**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate;	F_{c_perp}' = F_{c_perp} ´ C_{Mc} ´ C_{tc} ´ C_i ´ C_b = **425** lb/in^2
	f_c / F_{c_perp}' = **0.473**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1;	T_1 = **1.967** kips
Chord 2;	T_2 = **1.967** kips

Wind load deflection

Design shear force;	$V_{\delta w}$ = f_{Wserv} ´ W = **2.592** kips
Deflection limit;	Δ_{w_allow}= h / 400 = **0.333** in
Induced unit shear;	$v_{\delta w}$ = $V_{\delta w}$ / b = **172.8** lb/ft
Anchor tension force;	T_δ = max(0 kips,$v_{\delta w}$ ´ h ´ b / b_{eff}) = **1.967** kips
Chord compression force;	C_δ = max(0 kips,$v_{\delta w}$ ´ h ´ b / b_{eff}) = **1.967** kips
Vertical elongation at anchor;	Δ_T = T_δ / k_a = **0.079** in
Vertical compression at chord;	Δ_C = 0.04 in ´ C_δ / (A_e ´ F_{c_perp}) = **0.018** in
Total vertical deflection;	Δ_a = (Δ_T + Δ_C) ´ (b / b_{eff}) = **0.099** in
Shear wall deflection – Eqn. 4.3-1;	δ_{sww} = 2 ´ $v_{\delta w}$ ´ h^3 / (3 ´ E ´ A_e ´ b) + $v_{\delta w}$ ´ h / (G_{ac}) + h ´ Δ_a / b = **0.229** in
	δ_{sww} / Δ_{w_allow} = **0.688**

PASS - Shear wall deflection is less than deflection limit

STUD WALL DESIGN

WOOD MEMBER ANALYSIS & DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the LRFD method

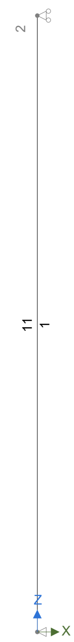
Tedds calculation version 2.2.22

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 2"x6"



Span	Length (ft)	Section	Start Support	End Support
1	11	2"x6"	Pinned	Roller Pin X
2"x6": Area 8 in^2, Inertia Major 21 in^4, Inertia Minor 2 in^4, Shear area parallel to Minor 7 in^2, Shear area parallel to Major 7 in^2				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft^3, Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C^{-1}				

Loading

Self weight included

Made by: ACO/RAH	Date: 08/13/2025	Job No. 24-11
Revision:	Date:	Sheet No. 64
Project Name: Bailey Farms Clubhouse		

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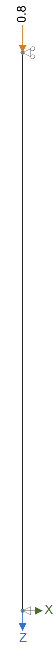
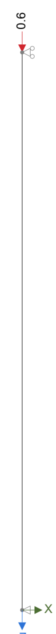
Dead - Loading (kips)



Snow - Loading (kips)



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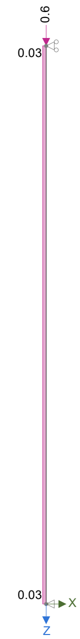
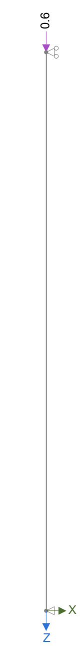
Revision:	Date:	Sheet No. 65
Project Name: Bailey Farms Clubhouse		

Wind - Loading (kips/ft,kips)



Roof Live - Loading (kips)



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Load combination factors

Load combination	Self Weight	Dead	Snow	Wind	Roof Live
1.2D + 1.6Lr + 0.5W (Strength)	1.20	1.20		0.50	1.60
1.0D + 1.0Lr + 0.5W (Service)	1.00	1.00		0.50	1.00
1.2D + 1.0S + 0.5W (Strength)	1.20	1.20	1.00	0.50	
1.0D + 0.7S + 0.5W (Service)	1.00	1.00	1.00	0.70	
0.9D + 1.0W (Strength)	0.90	0.90		1.00	
0.6D + 0.6W (Service)	0.60	0.60		0.60	

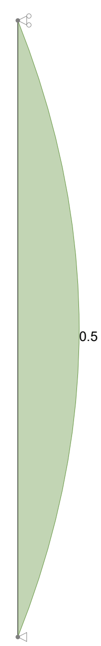
Member Loads

Member	Load case	Load Type	Orientation	Description
Column	Dead	Point load	GlobalZ	0.8 kips at 11 ft
Column	Snow	Point load	GlobalZ	0.6 kips at 11 ft
Column	Wind	Point load	GlobalZ	0.6 kips at 11 ft
Column	Wind	UDL	GlobalX	0.03 kips/ft
Column	Roof Live	Point load	GlobalZ	0.6 kips at 11 ft

Results

Forces

Strength combinations - Moment envelope (kip_ft)



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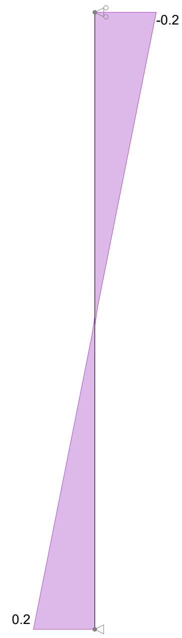
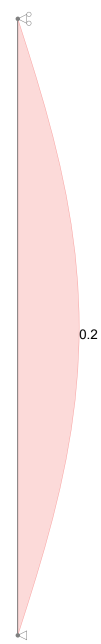
Strength combinations - Shear envelope (kips)



Service combinations - Deflection envelope (in)



;

Column - Span 1

Member details

Service condition; Dry

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Sawn lumber section details

Number of sections in member; $N = \mathbf{1}$

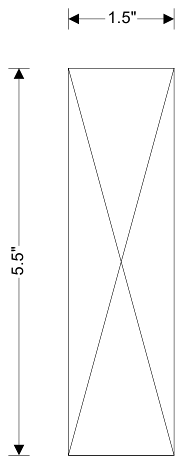
Nominal breadth of sections; $b_{nom} = \mathbf{2}$ in

Breadth of sections; $b = \mathbf{1.5}$ in

Nominal depth of sections; $d_{nom} = \mathbf{6}$ in

Depth of sections; $d = \mathbf{5.5}$ in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



2"x6" sawn lumber section
Cross-sectional area, A, 8.25 in^2
Section modulus, S_x, 7.6 in^3
Section modulus, S_y, 2.1 in^3
Second moment of area, I_x, 20.8 in^4
Second moment of area, I_y, 1.5 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 0.433 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length; $L_b = \mathbf{4}$ in

<u>Consider Combination 5 - 0.9D + 1.0W (Strength)</u>

Adjustment factors - Table 4.3.1

Size factor for bending - Table 4A; $C_{Fb} = \mathbf{1.3}$

Size factor for compression - Table 4A; $C_{Fc} = \mathbf{1.1}$

Repetitive member factor - Table 4.3.9; $C_r = \mathbf{1.15}$

Reference compression design value; $F_c^* = F_c \times C_{Fc} \times K_{Fc} \times \phi_c \times \lambda = \mathbf{2732}$ lb/in^2

Adjusted modulus of elasticity; $E_{min}' = E_{min} \times K_{FE} \times \phi_s = \mathbf{762960}$ lb/in^2

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 = \mathbf{1089}$ lb/in^2

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8} = \mathbf{0.358}$$

Format conversion factor for bending - Table 2.3.5; $K_{Fb} = \mathbf{2.54}$

Format conversion factor for shear - Table 2.3.5; $K_{Fv} = \mathbf{2.88}$

Format conversion factor for comp. - Table 2.3.5; $K_{Fc} = \mathbf{2.4}$

Format conv.factor for perp.comp. - Table 2.3.5; $K_{Fc\_perp} = \mathbf{1.67}$

Form.conv.factor for mod.of elasticity - Table 2.3.5; $K_{FE} = \mathbf{1.76}$

Resistance factor for bending - Table 2.3.6; $\phi_b = \mathbf{0.85}$

Resistance factor for shear - Table 2.3.6; $\phi_v = \mathbf{0.75}$

Resistance factor for compression - Table 2.3.6; $\phi_c = \mathbf{0.9}$

Resistance factor for mod.of elasticity - Table 2.3.6; $\phi_s = \mathbf{0.85}$

Time effect factor - Table N.3; $\lambda = \mathbf{1}$

Check design at start of span

Compression members - General - cl.3.6

Design axial compression force; $P = \mathbf{1337}$ lb

Design compression parallel to grain - Table 4.3.1; $F_c' = F_c \times C_{Fc} \times C_P \times K_{Fc} \times \phi_c \times \lambda = \mathbf{979}$ lb/in^2

Actual compression parallel to grain; $f_c = P / (b \times d) = \mathbf{162}$ lb/in^2

$f_c / F_c' = \mathbf{0.165}$

PASS - Design compression stress exceeds actual compression stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = \mathbf{165}$ lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times K_{Fv} \times \phi_v \times \lambda = \mathbf{292}$ lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = \mathbf{30}$ lb/in^2

$f_{v,x} / F_{v,x}' = \mathbf{0.103}$

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; $R_x = \mathbf{165}$ lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp} \times K_{Fc\_perp} \times \phi_c = \mathbf{639}$ lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b) = \mathbf{28}$ lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}' = \mathbf{0.043}$

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 5ft 6in along span

Compression members - General - cl.3.6

Design axial compression force; $P = \mathbf{1328}$ lb

Design compression parallel to grain - Table 4.3.1; $F_c' = F_c \times C_{Fc} \times C_P \times K_{Fc} \times \phi_c \times \lambda = \mathbf{979}$ lb/in^2

Actual compression parallel to grain; $f_c = P / (b \times d) = \mathbf{161}$ lb/in^2

$f_c / F_c' = \mathbf{0.164}$

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = \mathbf{454}$ lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_{Fb} \times C_r \times K_{Fb} \times \phi_b \times \lambda = \mathbf{2824}$ lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = \mathbf{720}$ lb/in^2

$f_{b,x} / F_{b,x}' = \mathbf{0.255}$

PASS - Design bending stress exceeds actual bending stress

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis; $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 = \mathbf{1089}$ lb/in^2

Critical buckling design value in y-axis; $F_{cE2} = 0.822 \times E_{min}' = \mathbf{627153}$ lb/in^2

Bending and compression check - eqs.3.9-3 and 3.9-4

$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2})) = \mathbf{0.326}$; < 1.0

PASS - Combined bending and compressive stresses are within permissible limits

Check design at end of span

Compression members - General - cl.3.6

Design axial compression force; $P = \mathbf{1320}$ lb

Design compression parallel to grain - Table 4.3.1; $F_c' = F_c \times C_{Fc} \times C_P \times K_{Fc} \times \phi_c \times \lambda$ = **979** lb/in^2

Actual compression parallel to grain; $f_c = P / (b \times d)$ = **160** lb/in^2

f_c / F_c' = **0.163**

PASS - Design compression stress exceeds actual compression stress

Bending members - Shear - cl.3.4

Design shear force; V_x = **165** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times K_{Fv} \times \phi_v \times \lambda$ = **292** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d)$ = **30** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.103**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **165** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp} \times K_{Fc\_perp} \times \phi_c$ = **639** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b)$ = **28** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.043**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Consider Combination 2 - 1.0D + 1.0Lr + 0.5W (Service)

Check design 5ft 6in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.174** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 250$ = **0.528** in

$\delta_x / \delta_{x,Allowable}$ = **0.33**

PASS - Allowable deflection exceeds instantaneous deflection

TOP PLATE DESIGN

WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation; 0.967

Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	1504	1455	0.967	PASS
Shear stress	lb/in^2	155	150	0.966	PASS
Bearing stress	lb/in^2	425	100	0.235	PASS

Design section 1

User note: Check beam at mid-span

Member details

Service condition; Dry

Load duration - Table 2.3.2; Two months

Sawn lumber section details

Number of sections in member; N = **2**

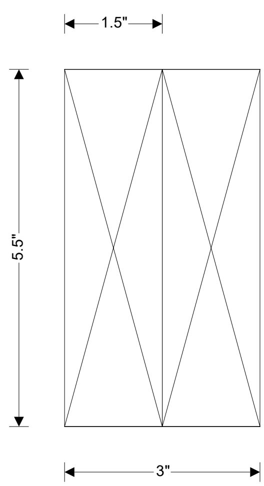
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **6** in

Depth of sections; d = **5.5** in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



2/2"x6" sawn lumber sections
Cross-sectional area, A, 16.5 in^2
Section modulus, S_x, 15.1 in^3
Section modulus, S_y, 8.3 in^3
Second moment of area, I_x, 41.6 in^4
Second moment of area, I_y, 12.4 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 0.866 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Unbraced length - Major axis; L_x = **1.333** ft

Effective bending length - Major axis; $L_{e,x}$ = 1.63 ´ L_x + 3 ´ N ´ b = **2.923** ft

Column buckling length - Major axis; $L_{b,x}$ = L_x = **1.333** ft

Unbraced length - Minor axis; L_y = **1.333** ft
Effective bending length - Minor axis; $L_{e,y}$ = 1.63 ´ L_y + 3 ´ d = **3.548** ft
Column buckling length - Minor axis; $L_{b,y}$ = L_y = **1.333** ft
Bearing length; L_b = **4** in

Analysis results

Design bending moment - Minor axis; M_y = **1000** lb_ft
Design shear force - Minor axis; V_y = **1650** lb
Design perpendicular compression - Minor axis; R_y = **2200** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2; C_D = **1.15**
Size factor for bending - Table 4A; C_{Fb} = **1.3**
Flat use factor - Table 4A; C_{fu} = **1.15**

Bending members - Flexure - cl.3.3

Design bending moment; M_y = **1000** lb_ft
Design bending stress - Table 4.3.1; $F_{b,y}'$ = F_b ´ C_D ´ C_{Fb} ´ C_{fu} = **1504** lb/in^2
Actual bending stress - eq.3.3-2; $f_{b,y}$ = M_y / S_y = **1455** lb/in^2
$f_{b,y}$ / $F_{b,y}'$ = **0.967**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; V_y = **1650** lb
Design shear stress - Table 4.3.1; $F_{v,y}'$ = F_v ´ C_D = **155** lb/in^2
Actual shear stress - eq.3.4-2; $f_{v,y}$ = 3 ´ V_y / (2 ´ N ´ b ´ d) = **150** lb/in^2
$f_{v,y}$ / $F_{v,y}'$ = **0.966**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_y = **2200** lb
Design bearing stress - Table 4.3.1; $F_{c\_perp,y}'$ = F_{c_perp} = **425** lb/in^2
Actual bearing stress; $f_{c\_perp,y}$ = R_y / (d ´ L_b) = **100** lb/in^2
$f_{c\_perp,y}$ / $F_{c\_perp,y}'$ = **0.235**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

DORMER JOIST DESIGN

WOOD MEMBER ANALYSIS & DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

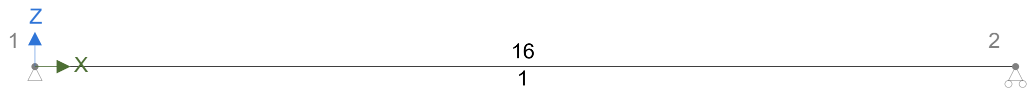
Tedds calculation version 2.2.20

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 2"x10"

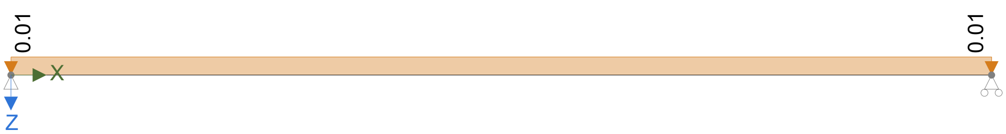


Span	Length (ft)	Section	Start Support	End Support
1	16	2"x10"	Pinned	Roller Pin X
2"x10": Area 14 in^2, Inertia Major 99 in^4, Inertia Minor 3 in^4, Shear area parallel to Minor 12 in^2, Shear area parallel to Major 12 in^2				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft^3, Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C^{-1}				

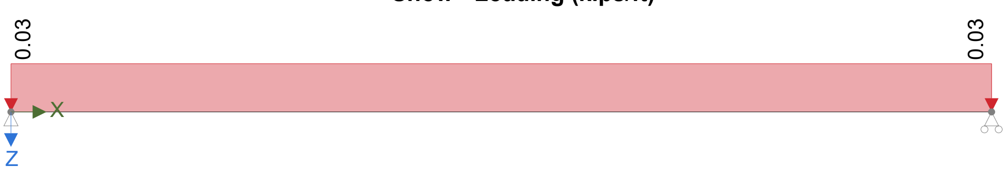
Loading

Self weight included

Dead - Loading (kips/ft)



Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Snow
1.0D + 1.0S (Strength)	1.00	1.00	1.00

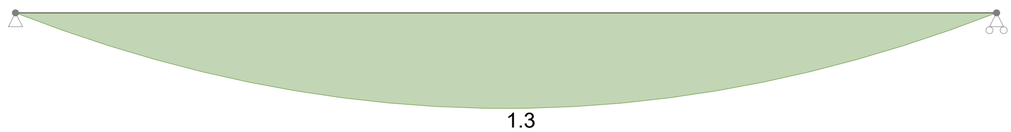
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.01 kips/ft
Beam	Snow	UDL	GlobalZ	0.03 kips/ft

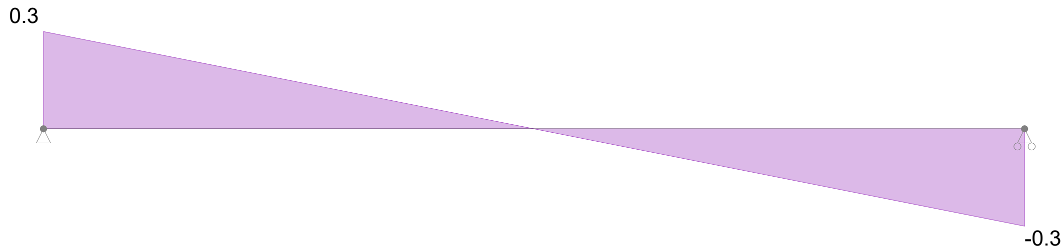
Results

Forces

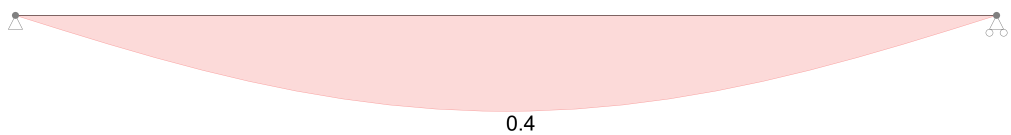
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Strength combinations - Deflection envelope (in)



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Beam - Span 1

Member details

Service condition;	Dry
Load duration - Table 2.3.2;	Ten years

Sawn lumber section details

Number of sections in member;	N = **1**
Nominal breadth of sections;	b_{nom} = **2** in
Breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **10** in
Depth of sections;	d = **9.25** in
Material;	**Spruce-Pine-Fir, 2" && wider, No.2 grade**

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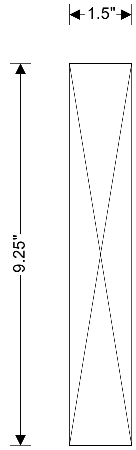


2"x10" sawn lumber section
Cross-sectional area, A, 13.875 in^2
Section modulus, S_x, 21.4 in^3
Section modulus, S_y, 3.5 in^3
Second moment of area, I_x, 98.9 in^4
Second moment of area, I_y, 2.6 in^4
Radius of gyration, r_x, 2.67 in
Radius of gyration, r_y, 0.433 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length; L_b = **2** in

<u>Consider Combination 1 - 1.0D + 1.0S (Strength)</u>

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2; C_D = **1.15**
Size factor for bending - Table 4A; C_{Fb} = **1.1**
Repetitive member factor - Table 4.3.9; C_r = **1.15**
Depth-to-breadth ratio; d_{nom} / b_{nom} = **5.00**
Slenderness ratio - eq.3.3-5; $R_B = \sqrt{(L_{e,y} \times d / b^2)}$ = **20.874**
Reference bending design value; $F_b^* = F_b \times C_D \times C_{Fb} \times C_r$ = **1273** lb/in^2
Adjusted modulus of elasticity; $E_{min}' = E_{min}$ = **510000** lb/in^2
Critical buckling design value - cl.3.3.3.8; $F_{bE} = 1.2 \times E_{min}' / R_B^2$ = **1405** lb/in^2
Beam stability factor - eq.3.3-6

$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]}$ = **0.854**

Time dependent deformation factor - cl.3.5.2; K_{cr} = **1.5**

<u>Check design at start of span</u>

Bending members - Shear - cl.3.4

Design shear force; V_x = **318** lb
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D$ = **155** lb/in^2
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d)$ = **34** lb/in^2
$f_{v,x} / F_{v,x}'$ = **0.222**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **318** lb
Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2
Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b)$ = **106** lb/in^2
$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.250**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 8ft along span

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **1274** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb} \times C_r$ = **1087** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **715** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.657**

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Shear - cl.3.4

Design shear force; V_x = **318** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D$ = **155** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d)$ = **34** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.222**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **318** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b)$ = **106** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.250**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 8ft along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.439** in

Final deflection; $\delta_{x,Final} = K_{cr} \times \delta_x$ = **0.658** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 250$ = **0.768** in

$\delta_{x,Final} / \delta_{x,Allowable}$ = **0.857**

PASS - Allowable deflection exceeds final deflection

NORTH COVERED PATIO JOIST

WOOD MEMBER ANALYSIS & DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.25

Beam span 1 results summary	**Unit**	**Capacity**	**Maximum**	**Utilization**	**Result**
Bending stress	lb/in^2	1263	930	0.736	PASS
Shear stress	lb/in^2	155	60	0.389	PASS
Bearing stress	lb/in^2	425	221	0.520	PASS
Deflection	in	0.588	0.573	0.975	PASS
Beam span 2 results summary	**Unit**	**Capacity**	**Maximum**	**Utilization**	**Result**
Bending stress	lb/in^2	1263	334	0.264	PASS
Shear stress	lb/in^2	155	31	0.200	PASS
Bearing stress	lb/in^2	425	221	0.520	PASS
Deflection	in	0.325	0.241	0.742	PASS

ANALYSIS

Tedds calculation version 1.0.38

Geometry

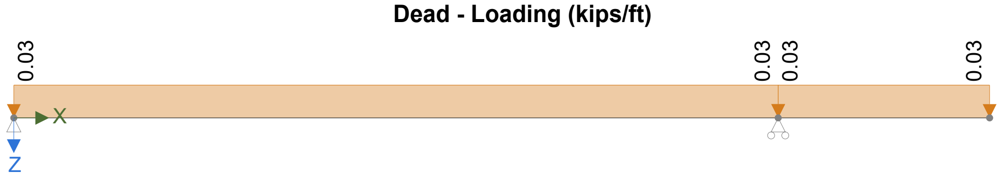
Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 2"x8"

Span	Length (ft)	Section	Start Support	End Support
1	11.75	2"x8"	Pinned	Roller Pin X
2	3.25	2"x8"	Roller Pin X	Free
2"x8": Area 11 in^2, Inertia Major 48 in^4, Inertia Minor 2 in^4, Shear area parallel to Minor 9 in^2, Shear area parallel to Major 9 in^2				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft^3, Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C^{-1}				

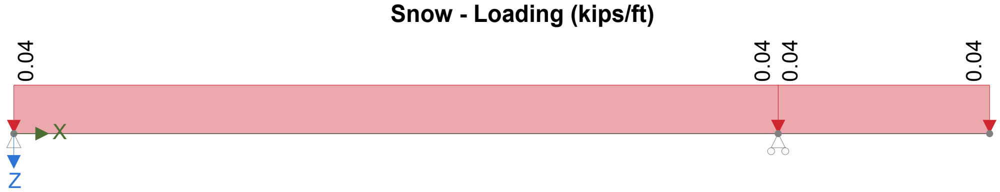
Loading

Self weight included

Dead - Loading (kips/ft)



Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Snow
1.0D + 1.0S (Strength)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.03 kips/ft
Beam	Snow	UDL	GlobalZ	0.04 kips/ft

Results

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0.012	0
2	0	0.021	0

Load case: Dead

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0.146	0
2	0	0.259	0

Load case: Snow

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0.217	0
2	0	0.383	0

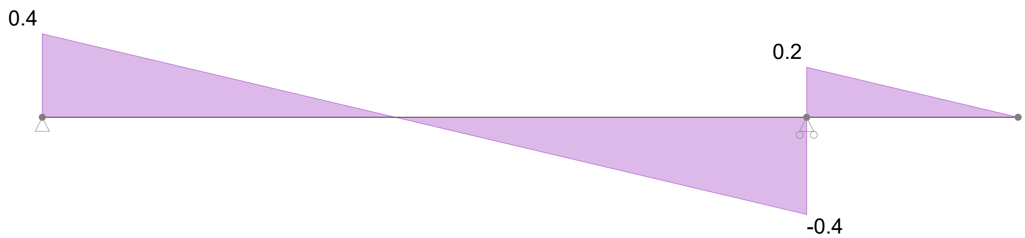
Load combination: 1.0D + 1.0S (Strength)

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
1	0	0.375	0
2	0	0.663	0

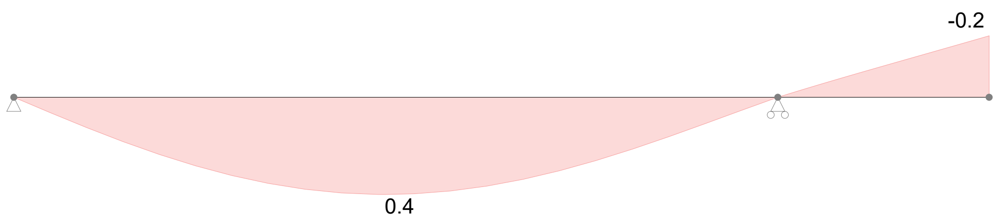
Forces

Strength combinations - Moment envelope (kip_ft)

Strength combinations - Shear envelope (kips)



Strength combinations - Deflection envelope (in)



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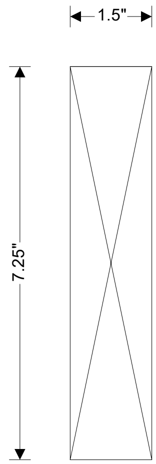
<u>Beam - Span 1</u>

Member details

Service condition;	Dry

Sawn lumber section details

Number of sections in member;	N = **1**
Nominal breadth of sections;	b_{nom} = **2** in
Breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **8** in
Depth of sections;	d = **7.25** in
Material;	**Spruce-Pine-Fir, 2" && wider, No.2 grade**



2"x8" sawn lumber section
Cross-sectional area, A, 10.875 in^2
Section modulus, S_x, 13.1 in^3
Section modulus, S_y, 2.7 in^3
Second moment of area, I_x, 47.6 in^4
Second moment of area, I_y, 2 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.433 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length; L_b = **2** in

<u>Consider Combination 1 - 1.0D + 1.0S (Strength)</u>

Adjustment factors - Table 4.3.1

Two months load duration factor - Table 2.3.2; C_D = **1.15**
Size factor for bending - Table 4A; C_{Fb} = **1.2**
Repetitive member factor - Table 4.3.9; C_r = **1.15**
Depth-to-breadth ratio; d_{nom} / b_{nom} = **4.00**
Slenderness ratio - eq.3.3-5; $R_B = \sqrt{(L_{e,y}\ ´\ d / b^2)}$ = **17.950**
Reference bending design value; $F_b^* = F_b\ ´\ C_D\ ´\ C_{Fb}\ ´\ C_r$ = **1389** lb/in^2
Adjusted modulus of elasticity; $E_{min}' = E_{min}$ = **510000** lb/in^2
Critical buckling design value - cl.3.3.3.8; $F_{bE} = 1.2\ ´\ E_{min}' / R_B^2$ = **1900** lb/in^2
Beam stability factor - eq.3.3-6

$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]}$ = **0.91**

Time dependent deformation factor - cl.3.5.2; K_{cr} = **1.5**

<u>Check design at start of span</u>

Bending members - Shear - cl.3.4

Design shear force; V_x = **375** lb
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v\ ´\ C_D$ = **155** lb/in^2
Actual shear stress - eq.3.4-2; $f_{v,x} = 3\ ´\ V_x / (2\ ´\ b\ ´\ d)$ = **52** lb/in^2
$f_{v,x} / F_{v,x}'$ = **0.334**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **375** lb
Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2
Actual bearing stress; $f_{c\_perp,x} = R_x / (b\ ´\ L_b)$ = **125** lb/in^2
$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.294**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 5ft 5.106in along span

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **1018** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb} \times C_r$ = **1263** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **930** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.736**

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **365** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb} \times C_r$ = **1263** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **334** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.264**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; V_x = **438** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D$ = **155** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d)$ = **60** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.389**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **663** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b)$ = **221** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.520**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 5ft 8.463in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.382** in

Final deflection; $\delta_{x,Final} = K_{cr} \times \delta_x$ = **0.573** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 240$ = **0.588** in

$\delta_{x,Final} / \delta_{x,Allowable}$ = **0.975**

PASS - Allowable deflection exceeds final deflection

Beam - Span 2

Member details

Service condition; Dry

Sawn lumber section details

Number of sections in member; N = **1**

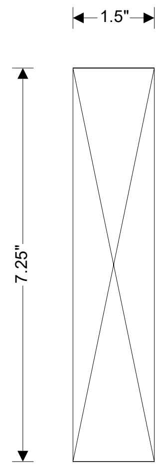
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **8** in

Depth of sections; $d = \mathbf{7.25}$ in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



2"x8" sawn lumber section
Cross-sectional area, A, 10.875 in^2
Section modulus, S_x, 13.1 in^3
Section modulus, S_y, 2.7 in^3
Second moment of area, I_x, 47.6 in^4
Second moment of area, I_y, 2 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.433 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length; $L_b = \mathbf{2}$ in

<u>Consider Combination 1 - 1.0D + 1.0S (Strength)</u>

Adjustment factors - Table 4.3.1

Two months load duration factor - Table 2.3.2; $C_D = \mathbf{1.15}$

Size factor for bending - Table 4A; $C_{Fb} = \mathbf{1.2}$

Repetitive member factor - Table 4.3.9; $C_r = \mathbf{1.15}$

Depth-to-breadth ratio; $d_{nom} / b_{nom} = \mathbf{4.00}$

Slenderness ratio - eq.3.3-5; $R_B = \sqrt{(L_{e,y} \times d / b^2)} = \mathbf{17.950}$

Reference bending design value; $F_b^* = F_b \times C_D \times C_{Fb} \times C_r = \mathbf{1389}$ lb/in^2

Adjusted modulus of elasticity; $E_{min}' = E_{min} = \mathbf{510000}$ lb/in^2

Critical buckling design value - cl.3.3.3.8; $F_{bE} = 1.2 \times E_{min}' / R_B^2 = \mathbf{1900}$ lb/in^2

Beam stability factor - eq.3.3-6

$$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]} = \mathbf{0.91}$$

<u>Check design at start of span</u>

Bending members - Flexure - cl.3.3

Design bending moment; $M_x = \mathbf{365}$ lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb} \times C_r = \mathbf{1263}$ lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = \mathbf{334}$ lb/in^2

$f_{b,x} / F_{b,x}' = \mathbf{0.264}$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; $V_x = \mathbf{225}$ lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D = \mathbf{155}$ lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times b \times d) = \mathbf{31}$ lb/in^2

$f_{v,x} / F_{v,x}' = \mathbf{0.200}$

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PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **663** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (b \times L_b)$ = **221** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.520**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

<u>Check design at end of span</u>

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.241** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s2} / 120$ = **0.325** in

$\delta_x / \delta_{x,Allowable}$ = **0.742**

PASS - Allowable deflection exceeds instantaneous deflection

RIDGE BEAM DESIGN

WOOD MEMBER ANALYSIS & DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

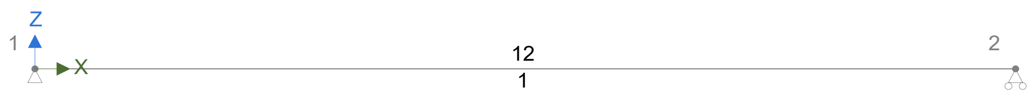
Tedds calculation version 2.2.20

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 3/2"x14"

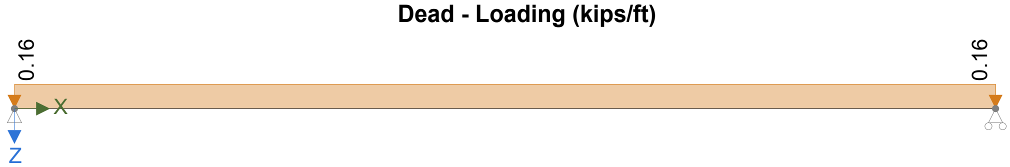


Span	Length (ft)	Section	Start Support	End Support
1	12	3/2"x14"	Pinned	Roller Pin X
3/2"x14": Area 60 in^2, Inertia Major 872 in^4, Inertia Minor 101 in^4, Shear area parallel to Minor 50 in^2, Shear area parallel to Major 50 in^2				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft^3, Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C^{-1}				

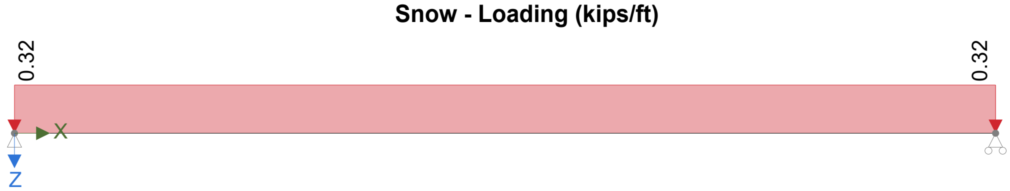
Loading

Self weight included

Dead - Loading (kips/ft)



Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Snow
1.0D + 1.0S (Strength)	1.00	1.00	1.00

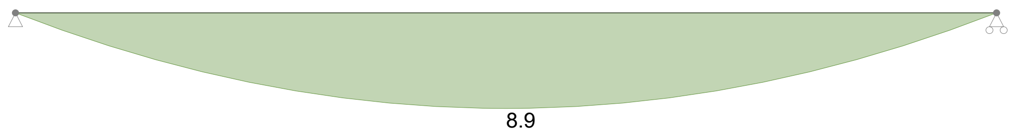
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.16 kips/ft
Beam	Snow	UDL	GlobalZ	0.32 kips/ft

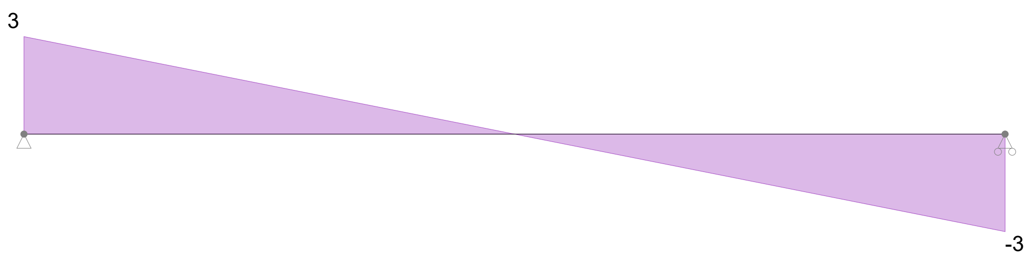
Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



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Beam - Span 1

Member details

Service condition; Dry

Load duration - Table 2.3.2; Ten years

Sawn lumber section details

Number of sections in member; N = **3**

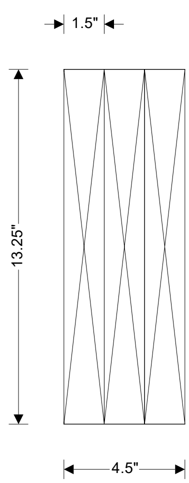
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **14** in

Depth of sections; d = **13.25** in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



3/2"x14" sawn lumber sections
Cross-sectional area, A, 59.625 in^2
Section modulus, S_x, 131.7 in^3
Section modulus, S_y, 44.7 in^3
Second moment of area, I_x, 872.3 in^4
Second moment of area, I_y, 100.6 in^4
Radius of gyration, r_x, 3.825 in
Radius of gyration, r_y, 1.299 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length; L_b = **2** in

Consider Combination 1 - 1.0D + 1.0S (Strength)

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2; C_D = **1.15**

Size factor for bending - Table 4A; C_{Fb} = **0.9**

Repetitive member factor - Table 4.3.9; C_r = **1.15**

Depth-to-breadth ratio; $d_{nom} / (N ´ b_{nom})$ = **2.33**

Slenderness ratio - eq.3.3-5; $R_B = \sqrt{(L_{e,y} ´ d / (N ´ b)^2)}$ = **8.787**

Reference bending design value; $F_b^* = F_b ´ C_D ´ C_{Fb} ´ C_r$ = **1041** lb/in^2

Adjusted modulus of elasticity; $E_{min}' = E_{min}$ = **510000** lb/in^2

Critical buckling design value - cl.3.3.3.8; $F_{bE} = 1.2 ´ E_{min}' / R_B^2$ = **7927** lb/in^2

Beam stability factor - eq.3.3-6

$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]}$ = **0.993**

Time dependent deformation factor - cl.3.5.2; K_{cr} = **1.5**

Check design at start of span

Bending members - Shear - cl.3.4

Design shear force; V_x = **2952** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v ´ C_D$ = **155** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 ´ V_x / (2 ´ N ´ b ´ d)$ = **74** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.478**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **2952** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (N ´ b ´ L_b)$ = **328** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.772**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 6ft along span

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **8857** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b ´ C_D ´ C_L ´ C_{Fb} ´ C_r$ = **1034** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **807** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.781**

PASS - Design bending stress exceeds actual bending stress

Check design 6ft along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.212** in

Final deflection; $\delta_{x,Final} = K_{cr} ´ \delta_x$ = **0.319** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 250$ = **0.576** in

$\delta_{x,Final} / \delta_{x,Allowable}$ = **0.553**

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PASS - Allowable deflection exceeds final deflection

HEADER DESIGN

TYPICAL HEADER DESIGN (< 4'-0")

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

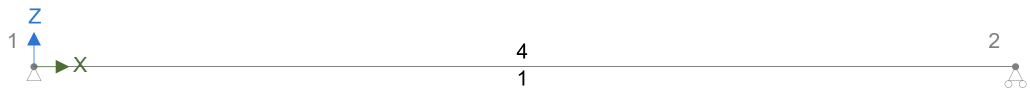
Tedds calculation version 2.2.20

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 2/2"x8"

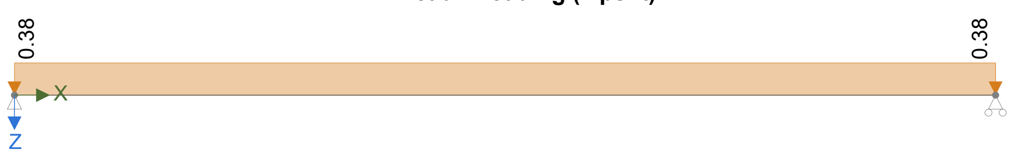


Span	Length (ft)	Section	Start Support	End Support
1	4	2/2"x8"	Pinned	Roller Pin X
2/2"x8": Area 22 in^2, Inertia Major 95 in^4, Inertia Minor 16 in^4, Shear area parallel to Minor 18 in^2, Shear area parallel to Major 18 in^2				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft^3, Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C^{-1}				

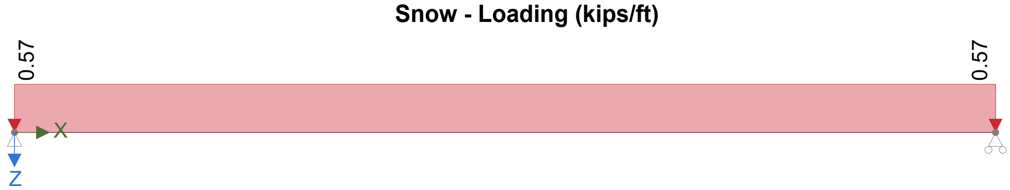
Loading

Self weight included

Dead - Loading (kips/ft)



Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Snow
1.0D + 1.0S (Strength)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.38 kips/ft
Beam	Snow	UDL	GlobalZ	0.57 kips/ft

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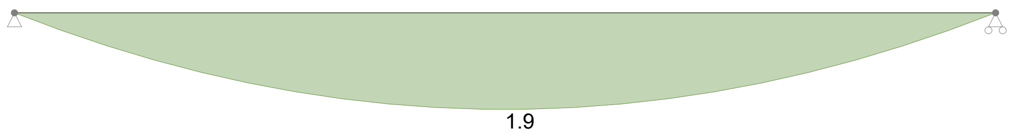
Made by: ACO/RAH	Date: 08/13/2025	Job No. 24-11
Revision:	Date:	Sheet No. 89
Project Name: Bailey Farms Clubhouse		

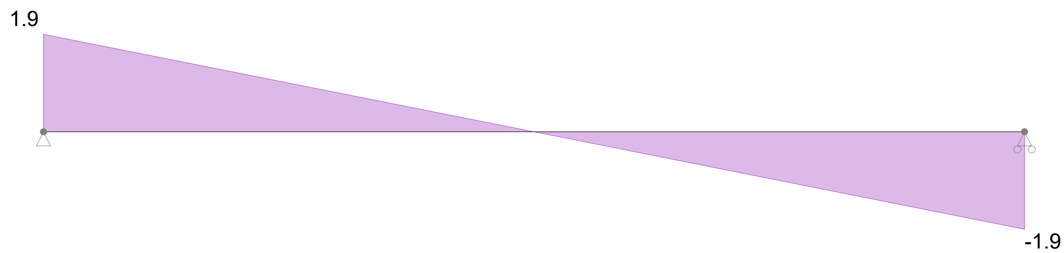
Results

Forces

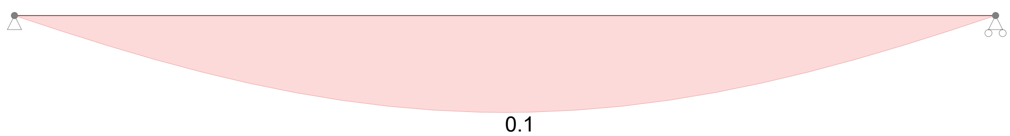
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Strength combinations - Deflection envelope (in)



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Beam - Span 1

Member details

Service condition;	Dry
Load duration - Table 2.3.2;	Ten years

Sawn lumber section details

Number of sections in member;	N = **2**
Nominal breadth of sections;	b_{nom} = **2** in
Breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **8** in
Depth of sections;	d = **7.25** in
Material;	**Spruce-Pine-Fir, 2" && wider, No.2 grade**

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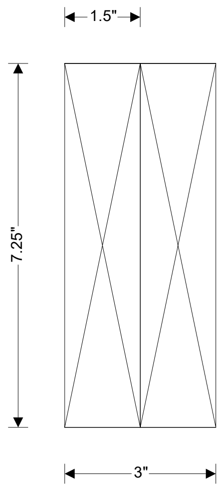


2/2"x8" sawn lumber sections
Cross-sectional area, A, 21.75 in^2
Section modulus, S_x, 26.3 in^3
Section modulus, S_y, 10.9 in^3
Second moment of area, I_x, 95.3 in^4
Second moment of area, I_y, 16.3 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.866 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length; L_b = **2** in

<u>Consider Combination 1 - 1.0D + 1.0S (Strength)</u>

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2; C_D = **1.15**
Size factor for bending - Table 4A; C_{Fb} = **1.2**
Repetitive member factor - Table 4.3.9; C_r = **1.15**
Depth-to-breadth ratio; d_{nom} / (N ´ b_{nom}) = **2.00**
Slenderness ratio - eq.3.3-5; R_B = √($L_{e,y}$ ´ d / (N ´ b)^2) = **8.975**
Reference bending design value; F_b^* = F_b ´ C_D ´ C_{Fb} ´ C_r = **1389** lb/in^2
Adjusted modulus of elasticity; E_{min}' = E_{min} = **510000** lb/in^2
Critical buckling design value - cl.3.3.3.8; F_{bE} = 1.2 ´ E_{min}' / R_B^2 = **7598** lb/in^2
Beam stability factor - eq.3.3-6

C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - √[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95] = **0.989**

Time dependent deformation factor - cl.3.5.2; K_{cr} = **1.5**

<u>Check design at start of span</u>

Bending members - Shear - cl.3.4

Design shear force; V_x = **1909** lb
Design shear stress - Table 4.3.1; $F_{v,x}'$ = F_v ´ C_D = **155** lb/in^2
Actual shear stress - eq.3.4-2; $f_{v,x}$ = 3 ´ V_x / (2 ´ N ´ b ´ d) = **132** lb/in^2
$f_{v,x}$ / $F_{v,x}'$ = **0.848**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **1909** lb
Design bearing stress - Table 4.3.1; $F_{c\_perp,x}'$ = F_{c_perp} = **425** lb/in^2
Actual bearing stress; $f_{c\_perp,x}$ = R_x / (N ´ b ´ L_b) = **318** lb/in^2
$f_{c\_perp,x}$ / $F_{c\_perp,x}'$ = **0.749**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 2ft along span

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **1909** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb} \times C_r$ = **1373** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **872** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.635**

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Shear - cl.3.4

Design shear force; V_x = **1909** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D$ = **155** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d)$ = **132** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.848**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **1909** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (N \times b \times L_b)$ = **318** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.749**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 2ft along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.056** in

Final deflection; $\delta_{x,Final} = K_{cr} \times \delta_x$ = **0.083** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 240$ = **0.2** in

$\delta_{x,Final} / \delta_{x,Allowable}$ = **0.417**

PASS - Allowable deflection exceeds final deflection

7'-0" HEADER DESIGN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

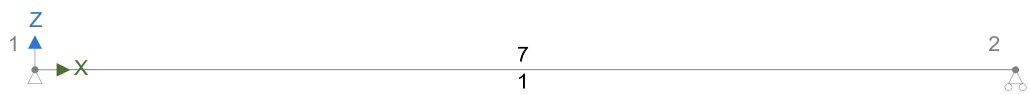
Tedds calculation version 2.2.20

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 3/2"x8"

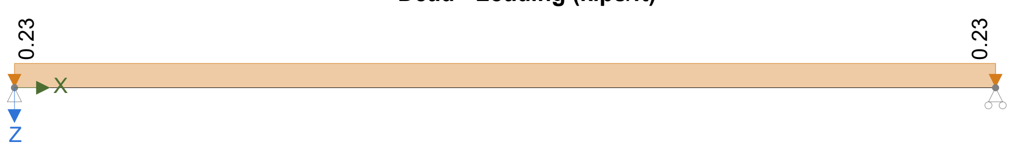


Span	Length (ft)	Section	Start Support	End Support
1	7	3/2"x8"	Pinned	Roller Pin X
3/2"x8": Area 33 in^2, Inertia Major 143 in^4, Inertia Minor 55 in^4, Shear area parallel to Minor 27 in^2, Shear area parallel to Major 27 in^2				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft^3, Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C^{-1}				

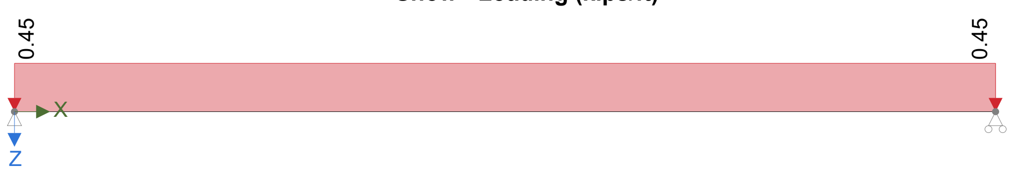
Loading

Self weight included

Dead - Loading (kips/ft)



Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Snow
1.0D + 1.0S (Strength)	1.00	1.00	1.00

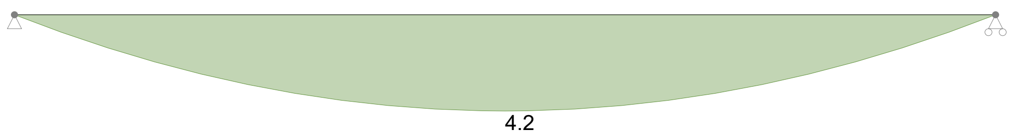
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.23 kips/ft
Beam	Snow	UDL	GlobalZ	0.45 kips/ft

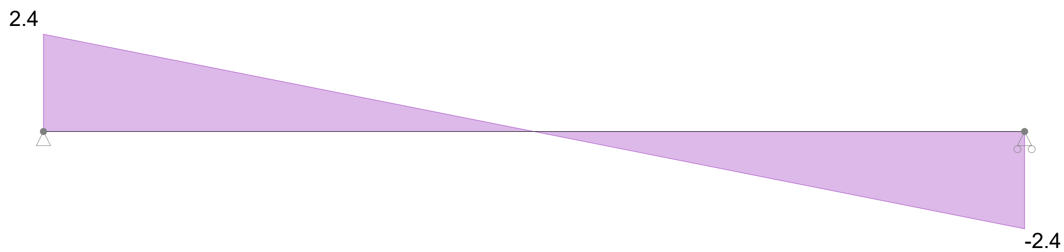
Results

Forces

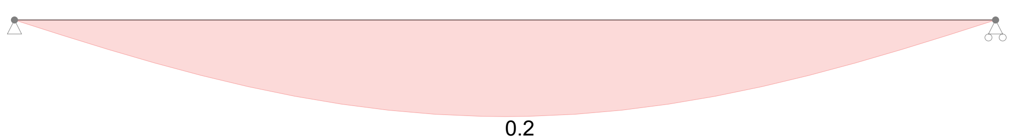
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Strength combinations - Deflection envelope (in)



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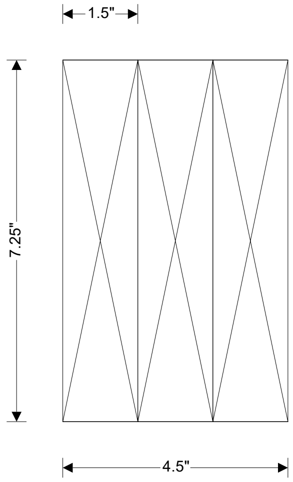
Beam - Span 1

Member details

Service condition;	Dry
Load duration - Table 2.3.2;	Ten years

Sawn lumber section details

Number of sections in member;	N = **3**
Nominal breadth of sections;	b_{nom} = **2** in
Breadth of sections;	b = **1.5** in
Nominal depth of sections;	d_{nom} = **8** in
Depth of sections;	d = **7.25** in
Material;	**Spruce-Pine-Fir, 2" && wider, No.2 grade**



3/2"x8" sawn lumber sections
Cross-sectional area, A, 32.625 in^2
Section modulus, S_x, 39.4 in^3
Section modulus, S_y, 24.5 in^3
Second moment of area, I_x, 142.9 in^4
Second moment of area, I_y, 55.1 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 1.299 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length;	L_b = **2** in

<u>Consider Combination 1 - 1.0D + 1.0S (Strength)</u>

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2;	C_D = **1.15**
Size factor for bending - Table 4A;	C_{Fb} = **1.2**
Repetitive member factor - Table 4.3.9;	C_r = **1.15**
Depth-to-breadth ratio;	$d_{nom} / (N \times b_{nom})$ = **1.33**
Slenderness ratio - eq.3.3-5;	$R_B = \sqrt{(L_{e,y} \times d / (N \times b)^2)}$ = **5.983**
Reference bending design value;	$F_b^* = F_b \times C_D \times C_{Fb} \times C_r$ = **1389** lb/in^2
Adjusted modulus of elasticity;	$E_{min}' = E_{min}$ = **510000** lb/in^2
Critical buckling design value - cl.3.3.3.8;	$F_{bE} = 1.2 \times E_{min}' / R_B^2$ = **17096** lb/in^2
Beam stability factor - eq.3.3-6	
	$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]}$ = **0.996**
Time dependent deformation factor - cl.3.5.2;	K_{cr} = **1.5**

<u>Check design at start of span</u>

Bending members - Shear - cl.3.4

Design shear force;	V_x = **2386** lb
Design shear stress - Table 4.3.1;	$F_{v,x}' = F_v \times C_D$ = **155** lb/in^2
Actual shear stress - eq.3.4-2;	$f_{v,x} = 3 \times V_x / (2 \times N \times b \times d)$ = **110** lb/in^2
	$f_{v,x} / F_{v,x}'$ = **0.706**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;	R_x = **2386** lb
Design bearing stress - Table 4.3.1;	$F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2
Actual bearing stress;	$f_{c\_perp,x} = R_x / (N \times b \times L_b)$ = **265** lb/in^2
	$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.624**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 3ft 6in along span

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **4175** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb} \times C_r$ = **1383** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **1271** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.919**

PASS - Design bending stress exceeds actual bending stress

Check design 3ft 6in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.205** in

Final deflection; $\delta_{x,Final} = K_{cr} \times \delta_x$ = **0.308** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 240$ = **0.35** in

$\delta_{x,Final} / \delta_{x,Allowable}$ = **0.879**

PASS - Allowable deflection exceeds final deflection

12'-6" HEADER DESIGN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

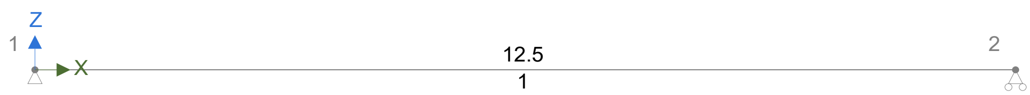
Tedds calculation version 2.2.20

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Microllam(2.0E LVL) - 3/1.75"x11.875"

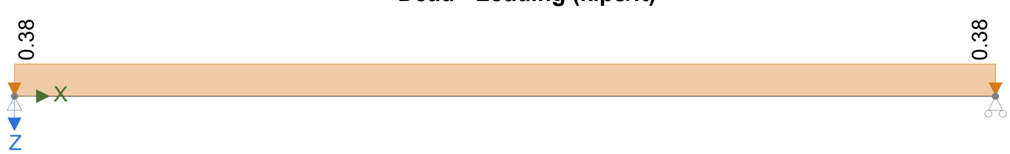


Span	Length (ft)	Section	Start Support	End Support
1	12.5	3/1.75"x11.875"	Pinned	Roller Pin X
3/1.75"x11.875": Area 62 in^2, Inertia Major 733 in^4, Inertia Minor 143 in^4, Shear area parallel to Minor 52 in^2, Shear area parallel to Major 52 in^2				
Microllam(2.0E LVL): Density 42 lbm/ft^3, Youngs 2000 ksi, Shear 125 ksi, Thermal 0 °C^{-1}				

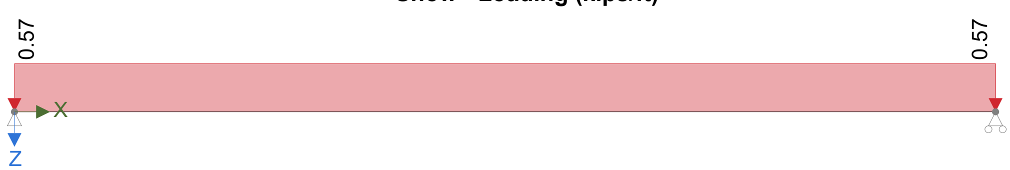
Loading

Self weight included

Dead - Loading (kips/ft)



Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Snow
1.0D + 1.0S (Strength)	1.00	1.00	1.00

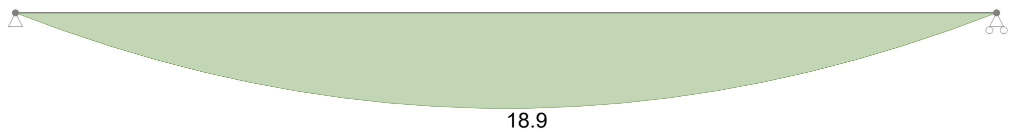
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.38 kips/ft
Beam	Snow	UDL	GlobalZ	0.57 kips/ft

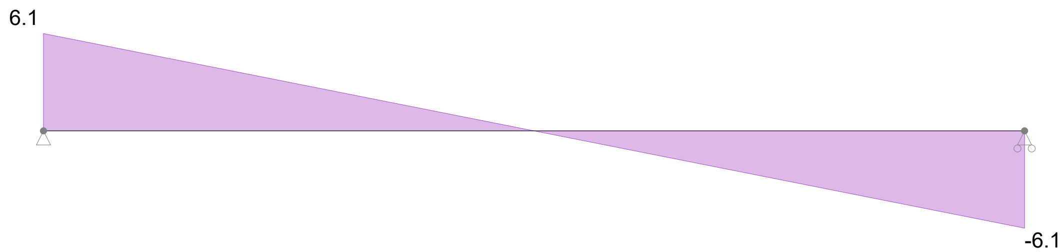
Results

Forces

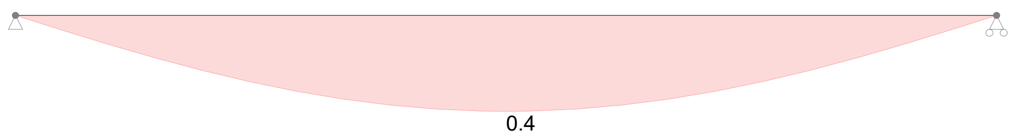
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Strength combinations - Deflection envelope (in)



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Beam - Span 1

Member details

Service condition;	Dry
Load duration - Table 2.3.2;	Ten years

Composite section details

Number of sawn lumber sections in member;	N = **3**
Breadth of sections;	b = **1.75** in
Depth of sections;	d = **11.875** in
Material;	**Microllam, 2.0E LVL grade**

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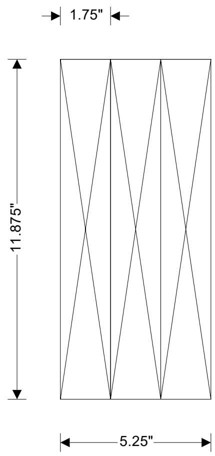
Made by: ACO/RAH	Date: 08/13/2025	Job No. 24-11
Revision:	Date:	Sheet No. 98
Project Name: Bailey Farms Clubhouse		



3/1.75"x11.875" composite sections
Cross-sectional area, A, 62.344 in^2
Section modulus, S_x, 123.4 in^3
Section modulus, S_y, 54.6 in^3
Second moment of area, I_x, 732.6 in^4
Second moment of area, I_y, 143.2 in^4
Radius of gyration, r_x, 3.428 in
Radius of gyration, r_y, 1.516 in
Microllam, x, 2.0E LVL grade
Bending about x-x axis, F_{bx}, 2600 psi
Bending about y-y axis, F_{by}, 2690 psi
Shear parallel to grain, bending about x-x axis, F_{vx}, 285 psi
Shear parallel to grain, bending about y-y axis, F_{vy}, 190 psi
Compression parallel to grain, F_c, 2510 psi
Compression perpendicular to grain, F_{c_perpx}, 750 psi
Compression perpendicular to grain, F_{c_perpy}, 680 psi
Tension parallel to grain, F_t, 1895 psi
Modulus of elasticity, E, 2000000 psi
Minimum modulus of elasticity, E_{min}, 1016535 psi
Density, ρ, 42 lbm/ft^3

Span details

Bearing length; L_b = **2** in

<u>Consider Combination 1 - 1.0D + 1.0S (Strength)</u>

Adjustment factors - Table 8.3.1

Load duration factor - Table 2.3.2; C_D = **1.15**

Repetitive member factor - Table 8.3.7; C_r = **1.04**

Depth-to-breadth ratio; d / (N × b) = **2.26**

Slenderness ratio - eq.3.3-5; $R_B = \sqrt{(L_{e,y} \times d / (N \times b)^2)}$ = **7.004**

Reference bending design value; $F_b^* = F_{bx} \times C_D \times C_{Vx} \times C_r$ = **3114** lb/in^2

Adjusted modulus of elasticity; $E_{min}' = E_{min}$ = **1016535** lb/in^2

Critical buckling design value - cl.3.3.3.8; $F_{bE} = 1.2 \times E_{min}' / R_B^2$ = **24866** lb/in^2

Beam stability factor - eq.3.3-6

$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]}$ = **0.993**

Volume factor maj.axis bending - Code evaluation; $C_{Vx} = (12\ in / d)^{0.136}$ = **1.001**

Time dependent deformation factor - cl.3.5.2; K_{cr} = **1.5**

<u>Check design at start of span</u>

Bending members - Shear - cl.3.4

Design shear force; V_x = **6051** lb

Design shear stress - Table 8.3.1; $F_{v,x}' = F_{vx} \times C_D$ = **328** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d)$ = **146** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.444**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **6051** lb

Design bearing stress - Table 8.3.1; $F_{c\_perp,x}' = F_{c\_perpx}$ = **750** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (N \times b \times L_b)$ = **576** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.768**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

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Check design 6ft 3in along span

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **18910** lb_ft

Design bending stress - Table 8.3.1; $F_{b,x}' = F_{bx} \times C_D \times C_L \times C_{Vx} \times C_r$ = **3092** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **1839** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.595**

PASS - Design bending stress exceeds actual bending stress

Check design at end of span

Bending members - Shear - cl.3.4

Design shear force; V_x = **6051** lb

Design shear stress - Table 8.3.1; $F_{v,x}' = F_{vx} \times C_D$ = **328** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d)$ = **146** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.444**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **6051** lb

Design bearing stress - Table 8.3.1; $F_{c\_perp,x}' = F_{c\_perpx}$ = **750** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (N \times b \times L_b)$ = **576** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.768**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Check design 6ft 3in along span

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.398** in

Final deflection; $\delta_{x,Final} = K_{cr} \times \delta_x$ = **0.597** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 240$ = **0.625** in

$\delta_{x,Final} / \delta_{x,Allowable}$ = **0.955**

PASS - Allowable deflection exceeds final deflection

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HEADER AT DORMER WINDOWS

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

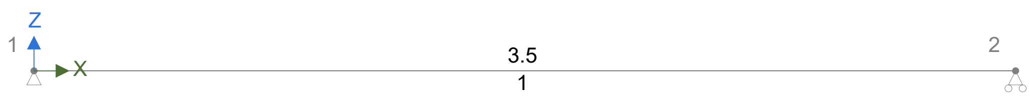
Tedds calculation version 2.2.20

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (ft) - Spruce-Pine-Fir(2" && wider(No.2)) - 3/2"x4"

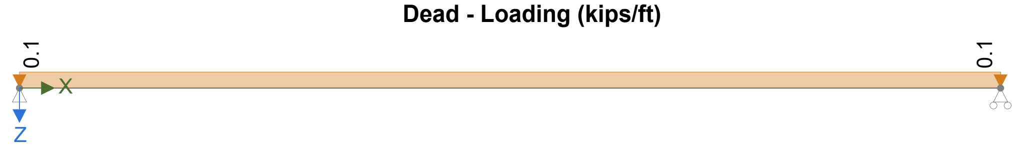


Span	Length (ft)	Section	Start Support	End Support
1	3.5	3/2"x4"	Pinned	Roller Pin X
3/2"x4": Area 16 in^2, Inertia Major 16 in^4, Inertia Minor 27 in^4, Shear area parallel to Minor 13 in^2, Shear area parallel to Major 13 in^2				
Spruce-Pine-Fir(2" && wider(No.2)): Density 29.1 lbm/ft^3, Youngs 1400 ksi, Shear 87 ksi, Thermal 0 °C^{-1}				

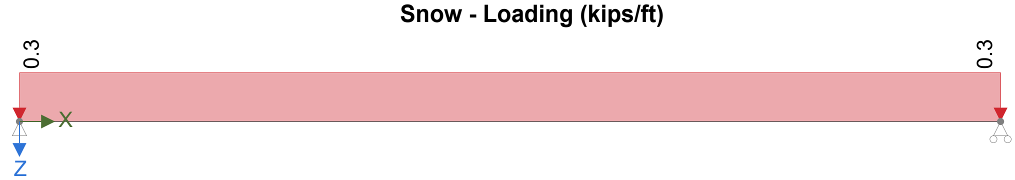
Loading

Self weight included

Dead - Loading (kips/ft)



Snow - Loading (kips/ft)



Load combination factors

Load combination	Self Weight	Dead	Snow
1.0D + 1.0S (Strength)	1.00	1.00	1.00

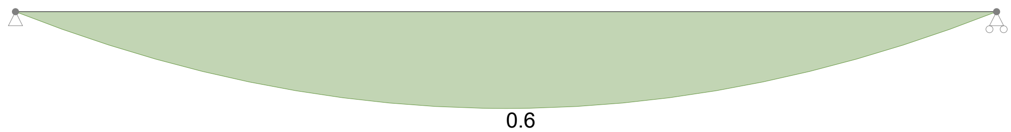
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.1 kips/ft
Beam	Snow	UDL	GlobalZ	0.3 kips/ft

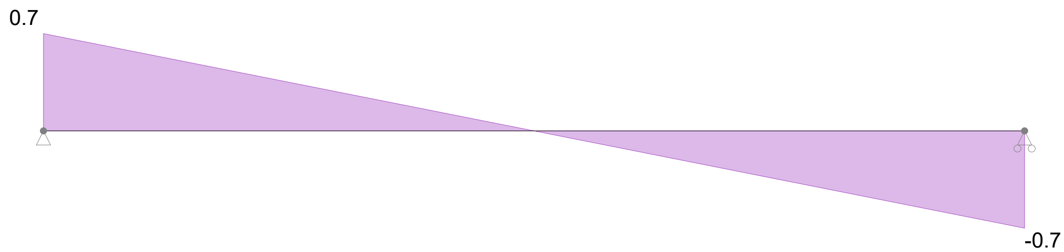
Results

Forces

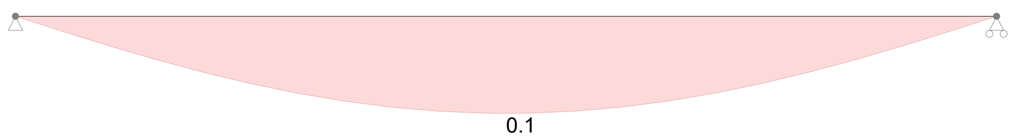
Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



Strength combinations - Deflection envelope (in)



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Beam - Span 1

Member details

Service condition; Dry

Load duration - Table 2.3.2; Ten years

Sawn lumber section details

Number of sections in member; N = **3**

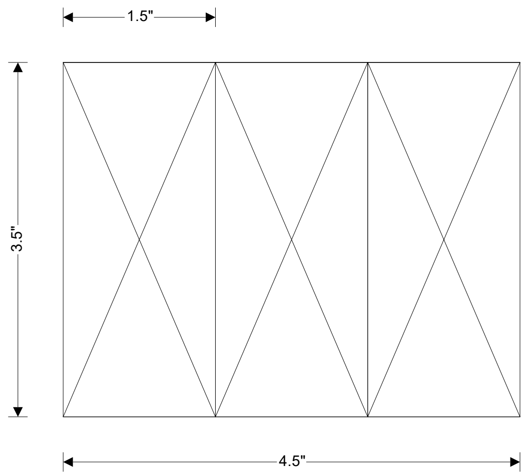
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **4** in

Depth of sections; d = **3.5** in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



3/2"x4" sawn lumber sections
Cross-sectional area, A, 15.75 in^2
Section modulus, S_x, 9.2 in^3
Section modulus, S_y, 11.8 in^3
Second moment of area, I_x, 16.1 in^4
Second moment of area, I_y, 26.6 in^4
Radius of gyration, r_x, 1.01 in
Radius of gyration, r_y, 1.299 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Bearing length; L_b = **2** in

__**Consider Combination 1 - 1.0D + 1.0S (Strength)**__

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2; C_D = **1.15**
Size factor for bending - Table 4A; C_{Fb} = **1.5**
Repetitive member factor - Table 4.3.9; C_r = **1.15**
Time dependent deformation factor - cl.3.5.2; K_{cr} = **1.5**

__**Check design at start of span**__

Bending members - Shear - cl.3.4

Design shear force; V_x = **706** lb
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v$ ´ C_D = **155** lb/in^2
Actual shear stress - eq.3.4-2; $f_{v,x}$ = 3 ´ V_x / (2 ´ N ´ b ´ d) = **67** lb/in^2
$f_{v,x} / F_{v,x}'$ = **0.433**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **706** lb
Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2
Actual bearing stress; $f_{c\_perp,x} = R_x$ / (N ´ b ´ L_b) = **78** lb/in^2
$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.184**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

__**Check design 1ft 9in along span**__

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **617** lb_ft
Design bending stress - Table 4.3.1; $F_{b,x}' = F_b$ ´ C_D ´ C_{Fb} ´ C_r = **1736** lb/in^2
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **806** lb/in^2
$f_{b,x} / F_{b,x}'$ = **0.465**

PASS - Design bending stress exceeds actual bending stress

<u>Check design at end of span</u>

Bending members - Shear - cl.3.4

Design shear force; V_x = **706** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v$ ´ C_D = **155** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x} = 3$ ´ V_x / (2 ´ N ´ b ´ d) = **67** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.433**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **706** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ = **425** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x$ / (N ´ b ´ L_b) = **78** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.184**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

<u>Check design 1ft 9in along span</u>

Bending members - Deflection - cl.3.5

Instantaneous deflection; δ_x = **0.067** in

Final deflection; $\delta_{x,Final} = K_{cr}$ ´ δ_x = **0.1** in

Allowable deflection; $\delta_{x,Allowable} = L_{m1\_s1} / 240$ = **0.175** in

$\delta_{x,Final} / \delta_{x,Allowable}$ = **0.574**

PASS - Allowable deflection exceeds final deflection

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COLUMN DESIGN

TYPICAL JACK STUD

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation; 0.765

Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	1308	793	0.607	PASS
Shear stress	lb/in^2	155	27	0.176	PASS
Compressive stress	lb/in^2	631	121	0.192	PASS
Bending and axial force				0.765	PASS

Design section 1

User note: Check column at base

Member details

Service condition; Dry

Load duration - Table 2.3.2; Two months

Sawn lumber section details

Number of sections in member; N = **2**

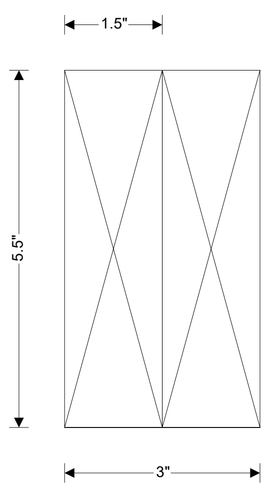
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **6** in

Depth of sections; d = **5.5** in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



2/2"x6" sawn lumber sections
Cross-sectional area, A, 16.5 in^2
Section modulus, S_x, 15.1 in^3
Section modulus, S_y, 8.3 in^3
Second moment of area, I_x, 41.6 in^4
Second moment of area, I_y, 12.4 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 0.866 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Unbraced length - Major axis; L_x = **11** ft

Effective bending length - Major axis; $L_{e,x}$ = 1.63 ´ L_x + 3 ´ N ´ b = **18.68** ft

Column buckling length - Major axis; $L_{b,x} = L_x = \mathbf{11}$ ft
Unbraced length - Minor axis; $L_y = \mathbf{0}$ ft
Bearing length; $L_b = \mathbf{4}$ in

Analysis results
Design bending moment - Major axis; $M_x = \mathbf{1000}$ lb_ft
Design shear force - Major axis; $V_x = \mathbf{300}$ lb
Design axial compression force; $P = \mathbf{2000}$ lb

Adjustment factors - Table 4.3.1
Load duration factor - Table 2.3.2; $C_D = \mathbf{1.15}$
Size factor for bending - Table 4A; $C_{Fb} = \mathbf{1.3}$
Size factor for compression - Table 4A; $C_{Fc} = \mathbf{1.1}$
Reference compression design value; $F_c^* = F_c \times C_D \times C_{Fc} = \mathbf{1455}$ lb/in^2
Adjusted modulus of elasticity; $E_{min}' = E_{min} = \mathbf{510000}$ lb/in^2
Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 = \mathbf{728}$ lb/in^2
Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8} = \mathbf{0.434}$$

Compression members - General - cl.3.6
Design axial compression force; $P = \mathbf{2000}$ lb
Design compression parallel to grain - Table 4.3.1; $F_c' = F_c \times C_D \times C_{Fc} \times C_P = \mathbf{631}$ lb/in^2
Actual compression parallel to grain; $f_c = P / (N \times b \times d) = \mathbf{121}$ lb/in^2
$f_c / F_c' = \mathbf{0.192}$

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3
Design bending moment; $M_x = \mathbf{1000}$ lb_ft
Design bending stress - Table 4.3.1; $F_{b,x}' = F_b \times C_D \times C_{Fb} = \mathbf{1308}$ lb/in^2
Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x = \mathbf{793}$ lb/in^2
$f_{b,x} / F_{b,x}' = \mathbf{0.607}$

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4
Design shear force; $V_x = \mathbf{300}$ lb
Design shear stress - Table 4.3.1; $F_{v,x}' = F_v \times C_D = \mathbf{155}$ lb/in^2
Actual shear stress - eq.3.4-2; $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d) = \mathbf{27}$ lb/in^2
$f_{v,x} / F_{v,x}' = \mathbf{0.176}$

PASS - Design shear stress exceeds actual shear stress

Combined bending and axial loading - cl.3.9
Critical buckling design value in x-axis; $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 = \mathbf{728}$ lb/in^2
Critical buckling design value in y-axis; $F_{cE2} = 0.822 \times E_{min}' = \mathbf{419220}$ lb/in^2
Bending and compression check - eqs.3.9-3 and 3.9-4

$$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2})) = \mathbf{0.765}; < 1.0$$

PASS - Combined bending and compressive stresses are within permissible limits

(3) 2X6 COLUMN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation; 0.854

Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	2065	719	0.348	PASS
Shear stress	lb/in^2	216	30	0.140	PASS
Compressive stress	lb/in^2	276	174	0.629	PASS
Bending and axial force				0.854	PASS

Design section 1

User note: Check column at base

Member details

Service condition; Dry

Load duration - Table 2.3.2; Ten minutes

Sawn lumber section details

Number of sections in member; N = **3**

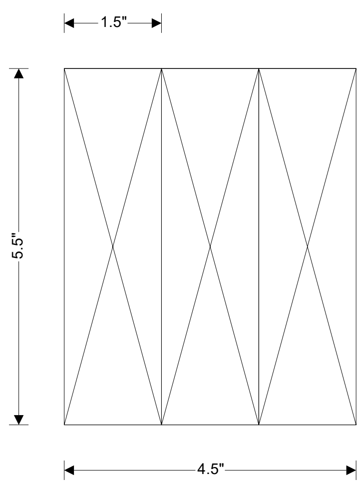
Nominal breadth of sections; b_{nom} = **2** in

Breadth of sections; b = **1.5** in

Nominal depth of sections; d_{nom} = **6** in

Depth of sections; d = **5.5** in

Material; **Spruce-Pine-Fir, 2" && wider, No.2 grade**



3/2"x6" sawn lumber sections
Cross-sectional area, A, 24.75 in^2
Section modulus, S_x, 22.7 in^3
Section modulus, S_y, 18.6 in^3
Second moment of area, I_x, 62.4 in^4
Second moment of area, I_y, 41.8 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 1.299 in
Spruce-Pine-Fir, 2" & wider, No.2 grade
Bending, F_b, 875 psi
Shear parallel to grain, F_v, 135 psi
Compression parallel to grain, F_c, 1150 psi
Compression perpendicular to grain, F_{c_perp}, 425 psi
Tension parallel to grain, F_t, 450 psi
Modulus of elasticity, E, 1400000 psi
Minimum modulus of elasticity, E_{min}, 510000 psi
Density, ρ, 29.098 lbm/ft^3
Specific gravity, G, 0.42

Span details

Unbraced length - Major axis; L_x = **11** ft

Effective bending length - Major axis; $L_{e,x}$ = 1.63 ´ L_x + 3 ´ N ´ b = **19.055** ft

Column buckling length - Major axis; $L_{b,x}$ = L_x = **11** ft

Unbraced length - Minor axis;	L_y = **11** ft
Effective bending length - Minor axis;	$L_{e,y} = 1.63 \times L_y + 3 \times d$ = **19.305** ft
Column buckling length - Minor axis;	$L_{b,y} = L_y$ = **11** ft
Bearing length;	L_b = **4** in

Analysis results

Design bending moment - Major axis;	M_x = **1360** lb_ft
Design shear force - Major axis;	V_x = **500** lb
Design axial compression force;	P = **4300** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2;	C_D = **1.6**
Size factor for bending - Table 4A;	C_{Fb} = **1.3**
Size factor for compression - Table 4A;	C_{Fc} = **1.1**
Repetitive member factor - Table 4.3.9;	C_r = **1.15**
Depth-to-breadth ratio;	$d_{nom} / (N \times b_{nom})$ = **1.00**
Slenderness ratio - eq.3.3-5;	$R_B = \sqrt{(L_{e,y} \times d / (N \times b)^2)}$ = **7.932**
Reference bending design value;	$F_b^* = F_b \times C_D \times C_{Fb} \times C_r$ = **2093** lb/in^2
Adjusted modulus of elasticity;	$E_{min}' = E_{min}$ = **510000** lb/in^2
Critical buckling design value - cl.3.3.3.8;	$F_{bE} = 1.2 \times E_{min}' / R_B^2$ = **9727** lb/in^2
Beam stability factor - eq.3.3-6	

$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]}$ = **0.987**

Column fixing factor for nails - cl.15.3.2;	K_f = **0.6**
Reference compression design value;	$F_c^* = F_c \times C_D \times C_{Fc}$ = **2024** lb/in^2
Adjusted modulus of elasticity;	$E_{min}' = E_{min}$ = **510000** lb/in^2
Critical buckling design value;	$F_{cE} = 0.822 \times E_{min}' / (L_{b,y} / (N \times b))^2$ = **487** lb/in^2
Column stability factor - eq.3.7-1	

$C_P = K_f \times ((1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{(((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8)})$ = **0.136**

Compression members - General - cl.3.6

Design axial compression force;	P = **4300** lb
Design compression parallel to grain - Table 4.3.1;	$F_c' = F_c \times C_D \times C_{Fc} \times C_P$ = **276** lb/in^2
Actual compression parallel to grain;	$f_c = P / (N \times b \times d)$ = **174** lb/in^2
	f_c / F_c' = **0.629**

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment;	M_x = **1360** lb_ft
Design bending stress - Table 4.3.1;	$F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb} \times C_r$ = **2065** lb/in^2
Actual bending stress - eq.3.3-2;	$f_{b,x} = M_x / S_x$ = **719** lb/in^2
	$f_{b,x} / F_{b,x}'$ = **0.348**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;	V_x = **500** lb
Design shear stress - Table 4.3.1;	$F_{v,x}' = F_v \times C_D$ = **216** lb/in^2
Actual shear stress - eq.3.4-2;	$f_{v,x} = 3 \times V_x / (2 \times N \times b \times d)$ = **30** lb/in^2
	$f_{v,x} / F_{v,x}'$ = **0.140**

PASS - Design shear stress exceeds actual shear stress

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis; $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2$ = **728** lb/in^2

Critical buckling design value in y-axis; $F_{cE2} = 0.822 \times E_{min}' / (L_{b,y} / (N \times b))^2$ = **487** lb/in^2

Bending and compression check - eqs.3.9-3 and 3.9-4

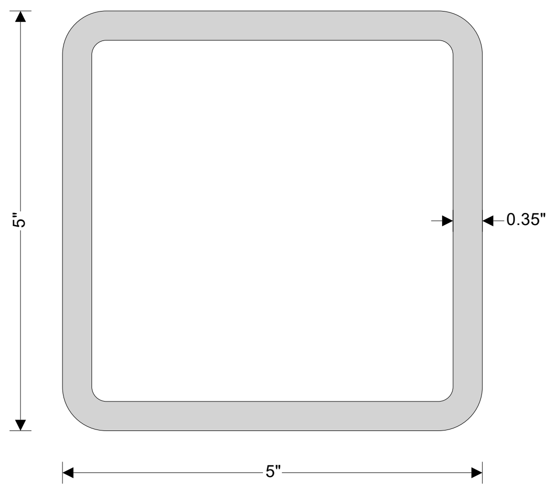
$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2}))$ = **0.854**; < 1.0

PASS - Combined bending and compressive stresses are within permissible limits

STEEL COLUMNS

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section; **HSS 5x5x3/8**

Design loading

Required axial strength; P_r = **8** kips; (Compression)

Moment about x axis at end 1; M_{x1} = **18.5** kips_ft

Moment about x axis at end 2; M_{x2} = **0.0** kips_ft

Single curvature bending about x axis

Maximum moment about x axis; M_x = max(abs(M_{x1}), abs(M_{x2})) = **18.5** kips_ft

Moment about y axis at end 1; M_{y1} = **0.0** kips_ft

Moment about y axis at end 2; M_{y2} = **0.0** kips_ft

Maximum moment about y axis; M_y = max(abs(M_{y1}), abs(M_{y2})) = **0.0** kips_ft

Maximum shear force parallel to y axis; V_{ry} = **0.0** kips

Maximum shear force parallel to x axis; V_{rx} = **0.0** kips

Material details

Steel grade; **A500 Gr. B**

Yield strength; F_y = **46** ksi

Ultimate strength; F_u = **58** ksi

Modulus of elasticity; E = **29000** ksi

Shear modulus of elasticity; G = **11200** ksi

Unbraced lengths

For buckling about x axis; L_x = **132** in

For buckling about y axis; L_y = **132** in

For torsional buckling; L_z = **132** in

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Effective length factors

For buckling about x axis; K_x = **1.20**

For buckling about y axis; K_y = **1.20**

For torsional buckling; K_z = **1.00**

Effective unbraced lengths

For buckling about x axis; $L_{cx} = L_x \times K_x$ = **158** in

For buckling about y axis; $L_{cy} = L_y \times K_y$ = **158** in

For torsional buckling; $L_{cz} = L_z \times K_z$ = **132** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width; $b = b_f - 3 \times t$ = **3.953** in

Critical web width; $h = d - 3 \times t$ = **3.953** in

Width to thickness ratio of flange (compression); $\lambda_{f\_c} = b / t$ = **11.327**

Width to thickness ratio of web (compression); $\lambda_{w\_c} = h / t$ = **11.327**

Width to thickness ratio of flange (major flexure); $\lambda_{f\_fx} = b / t$ = **11.327**

Width to thickness ratio of web (major flexure); $\lambda_{w\_fx} = h / t$ = **11.327**

Width to thickness ratio of flange (minor flexure); $\lambda_{f\_fy} = h / t$ = **11.327**

Width to thickness ratio of web (minor flexure); $\lambda_{w\_fy} = b / t$ = **11.327**

Compression

Limit for nonslender section; $\lambda_{r\_c} = 1.40 \times \sqrt{(E / F_y)}$ = **35.152**

The section is nonslender in compression

Flexure

Limit for compact flange; $\lambda_{pf\_f} = 1.12 \times \sqrt{(E / F_y)}$ = **28.121**

Limit for noncompact flange; $\lambda_{rf\_f} = 1.40 \times \sqrt{(E / F_y)}$ = **35.152**

Limit for compact web; $\lambda_{pw\_f} = 2.42 \times \sqrt{(E / F_y)}$ = **60.762**

Limit for noncompact web; $\lambda_{rw\_f} = 5.70 \times \sqrt{(E / F_y)}$ = **143.118**

The section is compact in flexure about the major axis

Slenderness

Member slenderness

Slenderness ratio about x axis; $SR_x = L_{cx} / r_x$ = **84.7**

Slenderness ratio about y axis; $SR_y = L_{cy} / r_y$ = **84.7**

Second order effects

Second order effects for bending about y axis (cl. C2.1b)

Second order effects are already included or do not need to be considered therefore:-

P-δ amplifier; $B_{1x} = B_{1y}$ = **1.0**

Required flexural strength (x axis); $M_{rx} = B_{1x} \times M_x$ = **18.5** kips_ft

Required flexural strength (y axis); $M_{ry} = B_{1y} \times M_y$ = **0.0** kips_ft

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress; $F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **39.9** ksi

Flexural buckling stress; $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **28.4** ksi

Nominal compressive strength for flexural buckling; $P_{nx} = F_{crx} \times A =$ **175.4** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress; $F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **39.9** ksi

Flexural buckling stress; $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y =$ **28.4** ksi

Nominal compressive strength for flexural buckling; $P_{ny} = F_{cry} \times A =$ **175.4** kips

Allowable compressive strength (cl. E1)

Safety factor for compression; $\Omega_c =$ **1.67**

Allowable compressive strength; $P_c = \min(P_{nx}, P_{ny}) / \Omega_c =$;**105.1**; kips

PASS - The allowable compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F7.1)

Nominal flexural strength; $M_{nx\_yld} = M_{px} = F_y \times Z_x =$ **40.6** kips_ft

Allowable flexural strength about the major axis (cl. F1)

Safety factor for flexure; $\Omega_b =$ **1.67**

Allowable flexural strength; $M_{cx} = \min(M_{nx\_yld}) / \Omega_b =$ **24.331** kip_ft

PASS - The allowable flexural strength about the major axis exceeds the required flexural strength

Library item - Design flex str ASD x RHSCombined forces

Member utilization (cl. H1.1)

Equation H1-1b; $UR = abs(P_r) / (2 \times P_c) + (M_{rx} / M_{cx} + M_{ry} / M_{cy}) =$ **0.798**

PASS - The member is adequate for the combined forces

FOUNDATION DESIGN

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.07

Summary results

Overall design status; PASS;

Overall design utilisation; 0.877

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	12.8			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.421	1.62	0.877	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	3.8	340.4	0.011	Pass
Moment, positive, y-direction	kip_ft	3.3	340.4	0.010	Pass
Min.area of reinf, bot., x-direction	in^2	2.333	2.370		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	14.5		Pass
Min.area of reinf, bot., y-direction	in^2	2.333	2.370		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	14.5		Pass

Pad footing details

Length of footing; L_x = **3** ft

Width of footing; L_y = **3** ft

Footing area; $A = L_x \times L_y$ = **9** ft^2

Depth of footing; h = **36** in

Depth of soil over footing; h_{soil} = **0** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3

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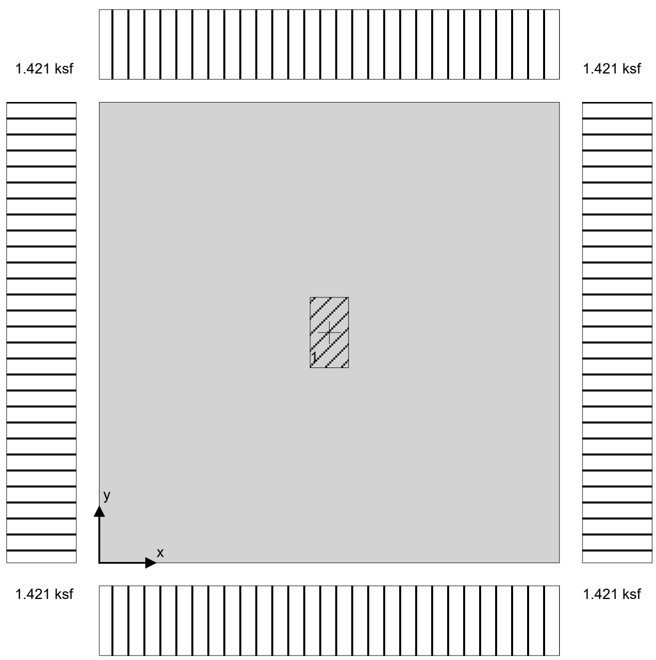


Column no.1 details

Length of column;	l_{x1} = **3.00** in
Width of column;	l_{y1} = **5.50** in
position in x-axis;	x_1 = **18.00** in
position in y-axis;	y_1 = **18.00** in

Soil properties

Net allowable bearing pressure;	q_{allow_Net} = **1.5** ksf; using a soil factor of safety, FS_{soil}, of 3
Density of soil;	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction;	ϕ_b = **30.0** deg
Design base friction angle;	δ_{bb} = **30.0** deg
Coefficient of base friction;	$\tan(\delta_{bb})$ = **0.577**
Dead surcharge load;	F_{Dsur} = **25** psf
Live surcharge load;	F_{Lsur} = **100** psf
Self weight;	$F_{swt} = h \,´\, \gamma_{conc}$ = **450** psf

Column no.1 loads

Dead load in z;	F_{Dz1} = **2.8** kips
Snow load in z;	F_{Sz1} = **4.2** kips
Wind load in z;	F_{Wz1} = **4.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.485)
1.0D + 1.0S (0.773)
1.0D + 0.6W (0.658)
1.0D + 0.75L + 0.75S + 0.45W (0.877)

0.6D + 0.6W (0.464)

Combination 12 results: 1.0D + 0.75L + 0.75S + 0.45W

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_S \times F_{Sz1} + \gamma_W \times F_{Wz1}$ = **12.8** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_S \times (F_{Sz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1)$ = **19.2** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **19.2** kip_ft

Uplift verification

Vertical force; F_{dz} = **12.79** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.421** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.421** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.421** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.421** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.421** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.421** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **1.62** ksf

q_{max} / q_{allow} = **0.877**

PASS - Allowable bearing capacity exceeds design base pressure

<u>FOOTING DESIGN</u>

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.07

Material details

Compressive strength of concrete; f'_c = **4000** psi

Yield strength of reinforcement; f_y = **60000** psi

Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**

Cover to top of footing; c_{nom_t} = **3** in

Cover to side of footing; c_{nom_s} = **3** in

Cover to bottom of footing; c_{nom_b} = **3** in

Concrete type; Normal weight

Concrete modification factor; λ = **1.00**

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.004)

1.2D + 1.0L + 1.6S (0.009)

1.2D + 1.6S + 0.5W (0.011)

1.2D + 1.0L + 0.5S + 1.0W (0.009)

0.9D + 1.0W (0.006)

Combination 9 results: 1.2D + 1.6S + 0.5W

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{Dsur}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_S \times F_{Sz1} + \gamma_W \times F_{Wz1}$ = **17.3** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_S \times (F_{Sz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1)$ = **26.0** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{Dsur}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **26.0** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis; $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.923** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.923** ksf

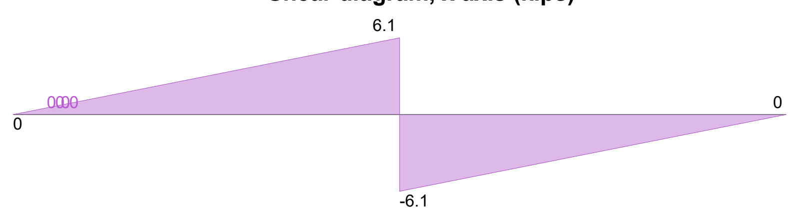
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.923** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.923** ksf

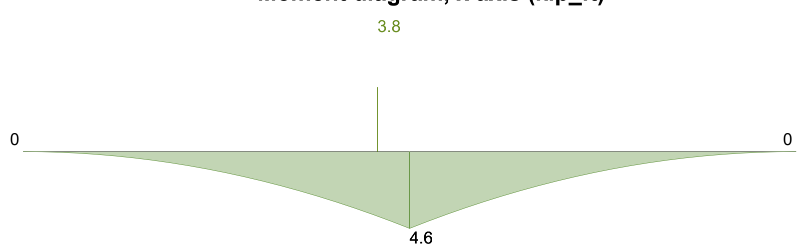
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.923** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.923** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max}$ = **3.84** kip_ft

Tension reinforcement provided; 3 No.8 bottom bars (14.5 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **2.37** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h$ = **2.333** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); s_{max} = min(2 $\times$ h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **32.500** in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **1.162** in

Neutral axis factor; β_1 = **0.85**

Depth to neutral axis; $c = a / \beta_1$ = **1.367** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06834**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **378.242** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n$ = **340.417** kip_ft

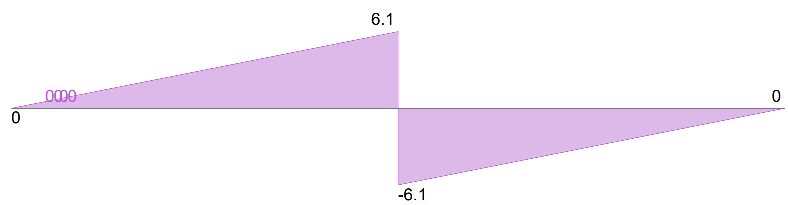
$M_{u.x.max} / \phi M_n$ = **0.011**

PASS - Design moment capacity exceeds ultimate moment load

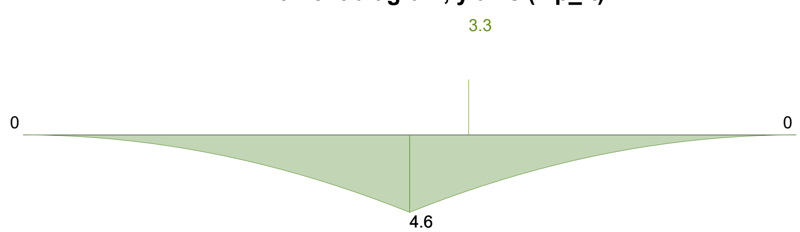
One-way shear design, x direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **3.278** kip_ft

Tension reinforcement provided; 3 No.8 bottom bars (14.5 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **2.37** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **2.333** in^2

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PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); s_{max} = min(2 $\times$ h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **31.500** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **1.162** in

Neutral axis factor; β_1 = **0.85**

Depth to neutral axis; $c = a / \beta_1$ = **1.367** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06614**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **366.392** kip_ft

Flexural strength reduction factor; ϕ_f = min(max(0.65 + 0.25 $\times$ ($\varepsilon_t - \varepsilon_{ty}$) / (0.003), 0.65), 0.9) = **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n$ = **329.752** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.010**

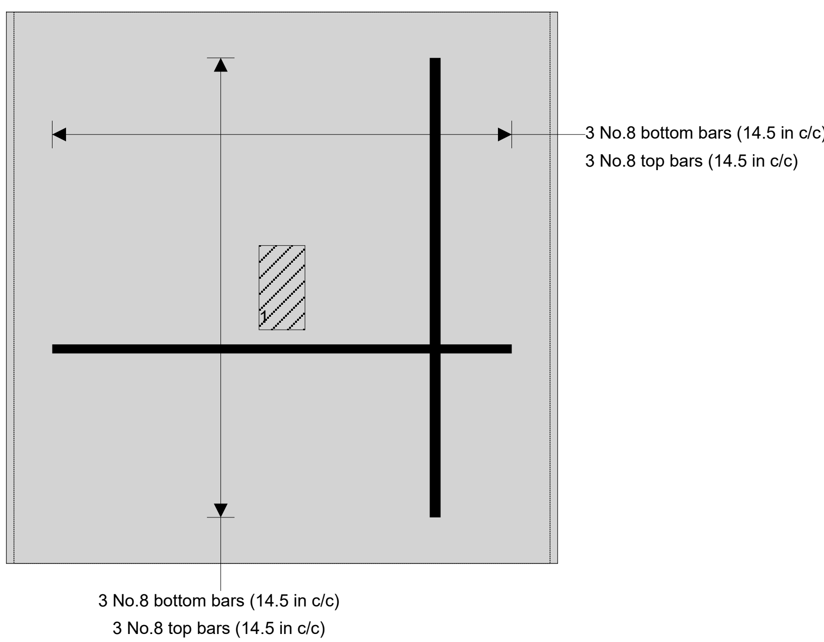
PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

Two-way shear design at column 1

Two-way shear design does not apply. Shear perimeter falls outside extents of foundation.



FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.07

Summary results

Overall design status; PASS;

Overall design utilisation; 0.792

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	1.9			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.283	1.62	0.792	Pass

Strip footing details - considering a one meter strip

Length of footing; L_x = **1** ft

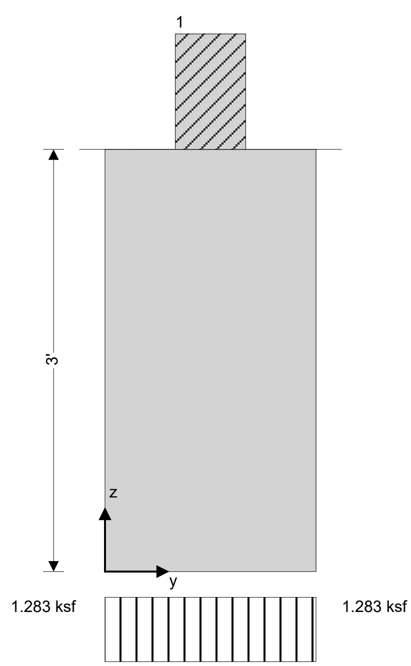
Width of footing; L_y = **1.5** ft

Footing area; $A = L_x \times L_y$ = **1.5** ft^2

Depth of footing; h = **36** in

Depth of soil over footing; h_{soil} = **0** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall; l_{y1} = **6** in

position in y-axis; y_1 = **9** in

Soil properties

Net allowable bearing pressure; q_{allow_Net} = **1.5** ksf; using a soil factor of safety, FS_{soil}, of 3

Density of soil;	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction;	ϕ_b = **30.0** deg
Design base friction angle;	δ_{bb} = **30.0** deg
Coefficient of base friction;	$\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight;	$F_{swt} = h$ ´ γ_{conc} = **450** psf

Wall no.1 loads per linear foot

Dead load in z;	F_{Dz1} = **0.5** kips
Live roof load in z;	F_{Lrz1} = **0.5** kips
Snow load in z;	F_{Sz1} = **0.8** kips
Wind load in z;	F_{Wz1} = **-0.5** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.484)

1.0D + 1.0S (0.792)

0.6D + 0.6W (0.426)

Combination 4 results: 1.0D + 1.0S

Forces on footing per linear foot

Force in z-axis; $F_{dz} = \gamma_D$ ´ A ´ $F_{swt} + \gamma_D$ ´ $(F_{Dz1} - l_{x1}$ ´ l_{y1} ´ h_{soil} ´ $\gamma_{soil}) + \gamma_S$ ´ F_{Sz1} = **1.9** kips

Moments on footing per linear foot

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D$ ´ A ´ F_{swt} ´ $L_y / 2 + \gamma_D$ ´ $(((F_{Dz1} - l_{x1}$ ´ l_{y1} ´ h_{soil} ´ $\gamma_{soil}))$ ´ $y_1) + \gamma_S$ ´ $(F_{Sz1}$ ´ $y_1)$ = **1.4** kip_ft

Uplift verification

Vertical force; F_{dz} = **1.925** kips

PASS - Footing is not subject to uplift

Stability against sliding

Resistance due to base friction; $F_{RFriction} = \max(F_{dz}, 0\ kN)$ ´ $\tan(\delta_{bb})$ = **1.111** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz}$ ´ $(1 - 6$ ´ $e_{dy} / L_y) / (L_y$ ´ $1\ ft)$ = **1.283** ksf

$q_2 = F_{dz}$ ´ $(1 + 6$ ´ $e_{dy} / L_y) / (L_y$ ´ $1\ ft)$ = **1.283** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2)$ = **1.283** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2)$ = **1.283** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Net} + ((h + h_{soil})$ ´ $\gamma_{soil}) / FS_{soil}$ = **1.62** ksf

q_{max} / q_{allow} = **0.792**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.08

Summary results

Overall design status; PASS;

Overall design utilisation; 0.794

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	24.4			Pass
Overturning stability, x	kip_ft	18.50	-73.12	3.95	Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.191	1.5	0.794	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	12.7	296.9	0.043	Pass
Moment, negative, x-direction	kip_ft	3.7	296.9	0.012	Pass
Moment, positive, y-direction	kip_ft	4.5	275.6	0.016	Pass
Shear, one-way, x-direction	kips	5.9	99.0	0.060	Pass
Shear, one-way, y-direction	kips	2.0	92.2	0.022	Pass
Shear, two-way, Col 1	psi	4.365	189.737	0.023	Pass
Min.area of reinf, bot., x-direction	in^2	2.333	4.740		Pass
Min.area of reinf, top, x-direction	in^2	2.333	4.740		Pass
Max.reinf.spacing, top, x-direction	in	18.0	13.0		Pass
Min.area of reinf, bot., y-direction	in^2	2.333	4.740		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	13.0		Pass

Pad footing details

Length of footing; L_x = **6** ft

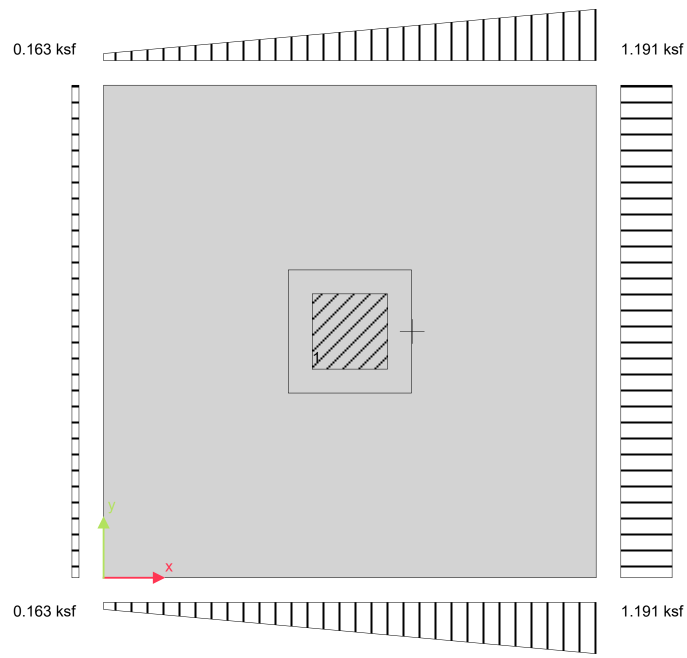
Width of footing; L_y = **6** ft

Footing area; $A = L_x \times L_y$ = **36** ft^2

Depth of footing; h = **18** in

Depth of soil over footing; h_{soil} = **24** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column; l_{x1} = **11.00** in

Width of column; l_{y1} = **11.00** in

position in x-axis; x_1 = **36.00** in

position in y-axis; y_1 = **36.00** in

Height of pedestal; h_{ped1} = **24.00** in

Length of pedestal; $l_{x,ped1}$ = **18.00** in

Width of pedestal; $l_{y,ped1}$ = **18.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **1.5** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight; $F_{swt} = h$ ´ γ_{conc} = **225** psf

Soil weight; $F_{soil} = h_{soil}$ ´ γ_{soil} = **240** psf

Column no.1 loads

Pedestal self weight; F_{SWz1} = **0.7** kips

Dead load in z; F_{Dz1} = **7.5** kips

Wind load in x; F_{Wx1} = **1.7** kips

Dead load moment in x; M_{Dx1} = **18.5** kip_ft

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Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.794)

1.0D + 1.0L (0.794)

Combination 1 results: 1.0D

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D$ ´ A ´ $(F_{swt} + F_{soil}) + \gamma_D$ ´ $(F_{Dz1} + F_{SWz1} - l_{x,ped1}$ ´ $l_{y,ped1}$ ´ h_{soil} ´ $\gamma_{soil}) =$ **24.4** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D$ ´ $(A$ ´ $(F_{swt} + F_{soil})$ ´ $L_x / 2) + \gamma_D$ ´ $(((F_{Dz1} + F_{SWz1} - l_{x,ped1}$ ´ $l_{y,ped1}$ ´ h_{soil} ´ $\gamma_{soil}))$ ´ $x_1+M_{Dx1}) =$ **91.6** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D$ ´ $(A$ ´ $(F_{swt} + F_{soil})$ ´ $L_y / 2) + \gamma_D$ ´ $(((F_{Dz1} + F_{SWz1} - l_{x,ped1}$ ´ $l_{y,ped1}$ ´ h_{soil} ´ $\gamma_{soil}))$ ´ $y_1) =$ **73.1** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **24.375** kips

PASS - Footing is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment; $M_{OTxL} = \gamma_D$ ´ $(M_{Dx1}) =$ **18.5** kip_ft

Resisting moment; $M_{RxL} = -1$ ´ $(\gamma_D$ ´ $(A$ ´ $(F_{swt} + F_{soil})$ ´ $L_x / 2)) + \gamma_D$ ´ $(((F_{Dz1} + F_{SWz1} - l_{x,ped1}$ ´ $l_{y,ped1}$ ´ h_{soil} ´ $\gamma_{soil}))$ ´ $(x_1 - L_x)) =$ **-73.12** kip_ft

Factor of safety; $abs(M_{RxL} / M_{OTxL}) =$ **3.953**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **9.108** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz}$ ´ $(1 - 6$ ´ $e_{dx} / L_x - 6$ ´ $e_{dy} / L_y) / (L_x$ ´ $L_y) =$ **0.163** ksf

$q_2 = F_{dz}$ ´ $(1 - 6$ ´ $e_{dx} / L_x + 6$ ´ $e_{dy} / L_y) / (L_x$ ´ $L_y) =$ **0.163** ksf

$q_3 = F_{dz}$ ´ $(1 + 6$ ´ $e_{dx} / L_x - 6$ ´ $e_{dy} / L_y) / (L_x$ ´ $L_y) =$ **1.191** ksf

$q_4 = F_{dz}$ ´ $(1 + 6$ ´ $e_{dx} / L_x + 6$ ´ $e_{dy} / L_y) / (L_x$ ´ $L_y) =$ **1.191** ksf

Minimum base pressure; $q_{min} = min(q_1,q_2,q_3,q_4) =$ **0.163** ksf

Maximum base pressure; $q_{max} = max(q_1,q_2,q_3,q_4) =$ **1.191** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} =$ **1.5** ksf

$q_{max} / q_{allow} =$ **0.794**

PASS - Allowable bearing capacity exceeds design base pressure

<u>FOOTING DESIGN</u>

In accordance with ACI318-14

Material details

Compressive strength of concrete; f'_c = **4000** psi
Yield strength of reinforcement; f_y = **60000** psi
Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**
Cover to top of footing; c_{nom_t} = **3** in
Cover to side of footing; c_{nom_s} = **3** in
Cover to bottom of footing; c_{nom_b} = **3** in
Concrete type; Normal weight
Concrete modification factor; λ = **1.00**
Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.060)
1.2D + 1.6L + 0.5Lr (0.051)

Combination 1 results: 1.4D

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil})$ = **34.1** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil})) \times x_1 + M_{Dx1})$ = **128.3** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil})) \times y_1)$ = **102.4** kip_ft

Eccentricity of base reaction

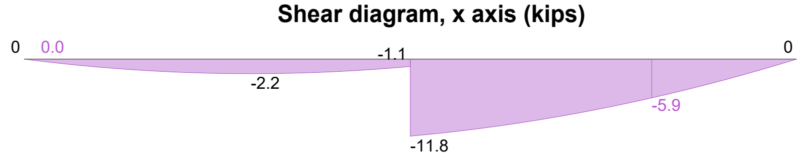
Eccentricity of base reaction in x-axis; $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **9.108** in
Eccentricity of base reaction in y-axis; $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

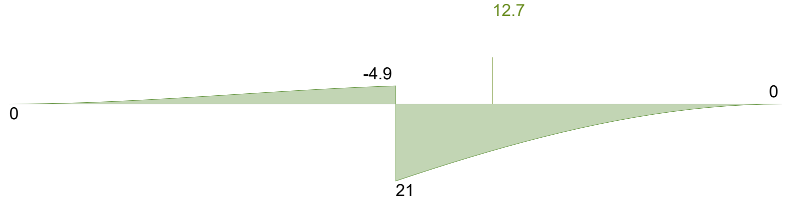
$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.228** ksf
$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.228** ksf
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.667** ksf
$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.667** ksf

Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.228** ksf
Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.667** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment;	$M_{u.x.max}$ = **12.704** kip_ft
Tension reinforcement provided;	6 No.8 bottom bars (13.0 in c/c)
Area of tension reinforcement provided;	$A_{sx.bot.prov}$ = **4.74** in^2
Minimum area of reinforcement (8.6.1.1);	$A_{s.min}$ = 0.0018 ´ L_y ´ h = **2.333** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);	s_{max} = min(2 ´ h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;	d = h - c_{nom_b} - $\phi_{x.bot}$ / 2 = **14.500** in
Depth of compression block;	a = $A_{sx.bot.prov}$ ´ f_y / (0.85 ´ f'_c ´ L_y) = **1.162** in
Neutral axis factor;	β_1 = **0.85**
Depth to neutral axis;	c = a / β_1 = **1.367** in
Strain in tensile reinforcement;	ε_t = 0.003 ´ d / c - 0.003 = **0.02883**
Minimum tensile strain(8.3.3.1);	ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;	M_n = $A_{sx.bot.prov}$ ´ f_y ´ (d - a / 2) = **329.883** kip_ft
Flexural strength reduction factor;	ϕ_f = min(max(0.65 + 0.25 ´ (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.900**
Design moment capacity;	ϕM_n = ϕ_f ´ M_n = **296.895** kip_ft
	$M_{u.x.max}$ / ϕM_n = **0.043**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment;	$M_{u.x.min}$ = **-3.686** kip_ft
Tension reinforcement provided;	6 No.8 top bars (13.0 in c/c)
Area of tension reinforcement provided;	$A_{sx.top.prov}$ = **4.74** in^2
Minimum area of reinforcement (8.6.1.1);	$A_{s.min}$ = 0.0018 ´ L_y ´ h = **2.333** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);	s_{max} = min(2 ´ h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;	d = h - c_{nom_t} - $\phi_{x.top}$ / 2 = **14.500** in
Depth of compression block;	a = $A_{sx.top.prov}$ ´ f_y / (0.85 ´ f'_c ´ L_y) = **1.162** in
Neutral axis factor;	β_1 = **0.85**
Depth to neutral axis;	c = a / β_1 = **1.367** in
Strain in tensile reinforcement;	ε_t = 0.003 ´ d / c - 0.003 = **0.02883**
Minimum tensile strain(8.3.3.1);	ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;	M_n = $A_{sx.top.prov}$ ´ f_y ´ (d - a / 2) = **329.883** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{296.895}$ kip_ft

$\text{abs}(M_{u.x.min}) / \phi M_n = \mathbf{0.012}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x} = \mathbf{5.95}$ kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2) = \mathbf{14.5}$ in

Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

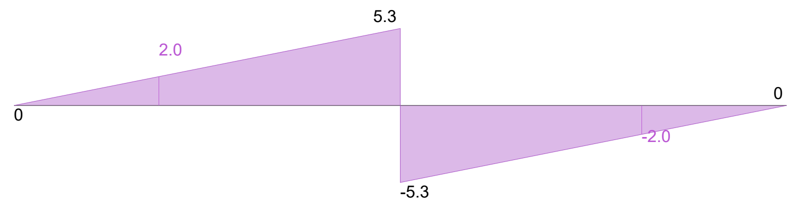
Nominal shear capacity (Eq. 22.5.5.1); $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v = \mathbf{132.057}$ kips

Design shear capacity; $\phi V_n = \phi_v \times V_n = \mathbf{99.043}$ kips

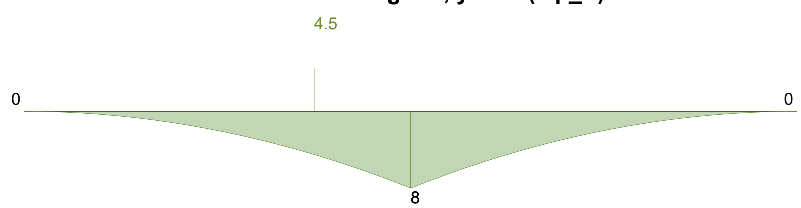
$V_{u.x} / \phi V_n = \mathbf{0.060}$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max} = \mathbf{4.511}$ kip_ft

Tension reinforcement provided; 6 No.8 bottom bars (13.0 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} = \mathbf{4.74}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h = \mathbf{2.333}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); $s_{max} = \min(2 \times h, 18\text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = \mathbf{13.500}$ in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{1.162}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.85}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{1.367}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.02663}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = 0.004 = \mathbf{0.00400}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = \mathbf{306.183}$ kip_ft

Flexural strength reduction factor;	ϕ_f = min(max(0.65 + 0.25 $\times$ (ε_t - ε_{ty}) / (0.005 - ε_{ty}), 0.65), 0.9) = **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n$ = **275.565** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.016**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;	$V_{u.y}$ = **2.005** kips
Depth to reinforcement;	$d_v = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **13.5** in
Shear strength reduction factor;	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v$ = **122.949** kips
Design shear capacity;	$\phi V_n = \phi_v \times V_n$ = **92.212** kips
	$V_{u.y} / \phi V_n$ = **0.022**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;	d_{v2} = **14** in
Shear perimeter length (22.6.4);	l_{xp} = **32.000** in
Shear perimeter width (22.6.4);	l_{yp} = **32.000** in
Shear perimeter (22.6.4);	$b_o = 2 \times (l_{x,ped1} + d_{v2}) + 2 \times (l_{y,ped1} + d_{v2})$ = **128.000** in
Shear area;	$A_p = l_{x,perim} \times l_{y,perim}$ = **1024.000** in^2
Surcharge loaded area;	$A_{sur} = A_p - l_{x,ped1} \times l_{y,ped1}$ = **700.000** in^2
Ultimate bearing pressure at center of shear area;	$q_{up.avg}$ = **0.948** ksf
Ultimate shear load;	$F_{up} = \gamma_D \times (F_{Dz1} + F_{SWz1} - l_{x,ped1} \times l_{y,ped1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **7.822** kips
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi})$ = **4.365** psi
Column geometry factor (Table 22.6.5.2);	$\beta = l_{y,ped1} / l_{x,ped1}$ = **1.00**
Column location factor (22.6.5.3);	α_s =**40**
Concrete shear strength (22.6.5.2);	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **403.190** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **252.982** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **252.982** psi
Shear strength reduction factor;	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2);	$v_n = v_{cp}$ = **252.982** psi
Design shear stress capacity (8.5.1.1(d));	$\phi v_n = \phi_v \times v_n$ = **189.737** psi
	$v_{ug} / \phi v_n$ = **0.023**
	PASS - Design shear stress capacity exceeds ultimate shear stress load

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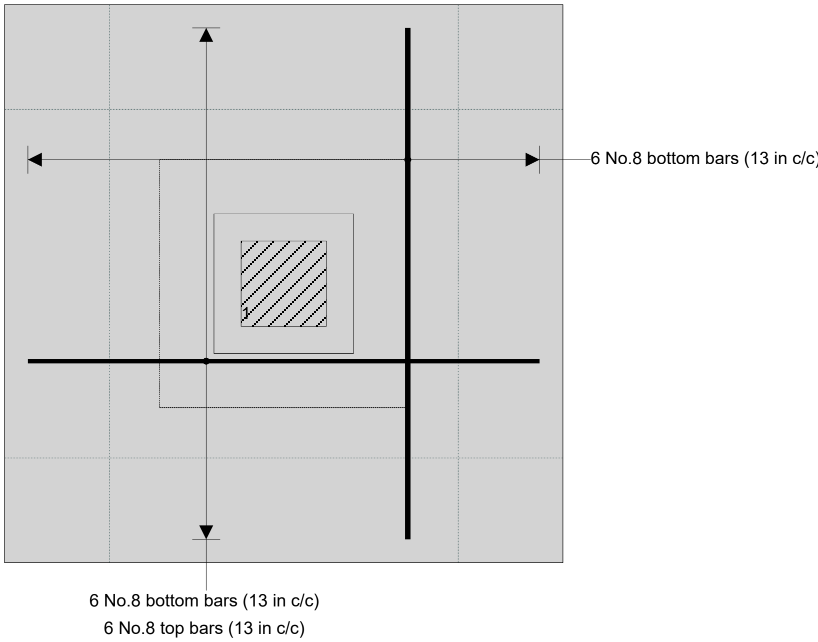
Made by: ACO/RAH	Date: 08/13/2025	Job No. 24-11
Revision:	Date:	Sheet No. 127
Project Name: Bailey Farms Clubhouse		



BASE PLATES

COLUMN BASE PLATE DESIGN

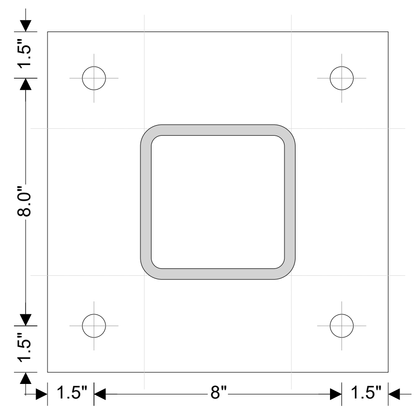
In accordance with AISC Steel Design Guide 1 and AISC 360-16

Tedds calculation version 2.1.12

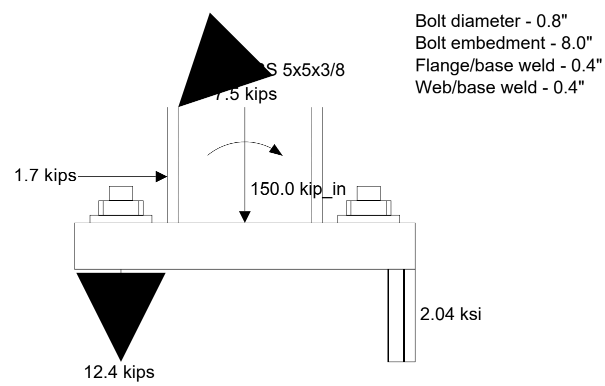
Design summary

Overall design status; PASS

Description	Unit	Required	Provided	Utilization	Result
Plate thickness (comp)	(in)	0.948	1.500	0.632	PASS
Plate thickness (tens)	(in)	0.581	1.500	0.388	PASS
Flange weld strength	(kips/in)	6.027	8.353	0.722	PASS
Tension weld strength	(kips/in)	3.1	8.4	0.370	PASS
Shear weld strength	(kips/in)	0.192	5.568	0.034	PASS



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force; P_u = **7.5** kips; (Compression)

Bending moment; M_u = **150.0** kip_in

Shear force; F_v = **1.7** kips

Eccentricity; e = ABS(M_u / P_u) = **20.000** in

Anchor bolt to center of plate; f = N/2 - e_1 = **4.000** in

Column details

Column section; HSS 5x5x3/8

Depth; d = **5.000** in

Breadth; b_f = **5.000** in

Thickness; t = **0.349** in

Baseplate details

Depth; N = **11.000** in

Breadth; B = **11.000** in

Thickness; t_p = **1.500** in

Design strength; F_y = **36.0** ksi

Foundation geometry

Member thickness; h_a = **16.000** in

Dist center of baseplate to left edge foundation;	x_{ce1} = **12.000** in
Dist center of baseplate to right edge foundation;	x_{ce2} = **12.000** in
Dist center of baseplate to bot edge foundation;	y_{ce1} = **12.000** in
Dist center of baseplate to top edge foundation;	y_{ce2} = **12.000** in

Holding down bolt and anchor plate details

Total number of bolts;	N_{bolt} = **4**
Bolt diameter;	d_o = **0.750** in
Bolt spacing;	s_{bolt} = **8.000** in
Edge distance;	e_1 = **1.500** in
Minimum tensile strength, base plate;	F_y = **36** ksi
Minimum tensile strength, column;	F_{yCol} = **46** ksi
Compressive strength of concrete;	f'_c = **3** ksi

Safety factors

Compression;	Ω_c = **2.50**
Flexure;	Ω_b = **1.67**
Weld shear;	Ω_v = **2.00**

Plate cantilever dimensions

Minimum distance to edge of concrete;	$l_{min} = \min(\min(x_{ce1}, x_{ce2}) - N / 2 , \min(y_{ce1}, y_{ce2}) - B / 2)$ = **6.500** in
Area of base plate;	$A_1 = B ´ N$ = **121.000** in^2
Maximum area of supporting surface;	$A_2 = (N + 2 ´ l_{min})^2 ´ (B / N)$ = **576.000** in^2
Nominal strength of concrete under base plate;	$P_p = 0.85 ´ f'_c ´ A_1 ´ \min(\sqrt{(A_2 / A_1)}, 2)$ = **617.1** kips
Bending line cantilever distance m;	$m = (N - 0.95 ´ d) / 2$ = **3.125** in
Bending line cantilever distance n;	$n = (B - 0.95 ´ b_f) / 2$ = **3.125** in
Maximum bending line cantilever;	$l = \max(m, n)$ = **3.125** in

Check eccentricity

Maximum bearing stress;	$f_{p,max} = 0.85 ´ f'_c / \Omega_c ´ \min(\sqrt{(A_2 / A_1)}, 2)$ = **2.04** ksi
Maximum bearing pressure;	$q_{max} = f_{p,max} ´ B$ = **22.44** kips/in
Critical eccentricity;	$e_{crit} = N / 2 - P_u / (2 ´ q_{max})$ = **5.333** in

e > e_{crit} so loads cannot be resisted by bearing alone. Therefore consider as a large moment

Plate dimensions adequate as $(f + N/2)^2 >= (2 ´ P_u ´ (e + f))/q_{max}$ and a real solution for bearing length exists

Bearing length - quadratic solution 1;	$Y_1 = (f + N/2) + \sqrt{((f + N/2)^2 - (2 ´ P_u ´ (e + f))/q_{max})}$ = **18.114** in
Bearing length - quadratic solution 2;	$Y_2 = (f + N/2) - \sqrt{((f + N/2)^2 - (2 ´ P_u ´ (e + f))/q_{max})}$ = **0.886** in
Bearing length;	$Y = \min(Y_1, Y_2)$ = **0.886** in
Tension force in bolts;	$T_u = q_{max} ´ Y - P_u$ = **12.4** kips
Max tension in single bolt;	$T_{rod} = T_u / (N_{bolt}/2)$ = **6.2** kips

Base plate yielding limit at bearing interface

Required plate thickness;	$t_{p,req} = \sqrt{((4 ´ f_{p,max} ´ Y ´ (l - Y/2))/(F_y / \Omega_b))}$ = **0.948** in

PASS - Thickness of plate exceeds required thickness

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines;	$x = abs(m - e_1)$ = **1.625** in
Plate thickness required;	$t_{p,req} = 2.58 ´ \sqrt{((T_u ´ x)/(B ´ F_y))}$ = **0.581** in

PASS - Thickness of plate exceeds required thickness

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Flange weld

Flange weld leg length; t_{wf} = **0.3750** in

Tension capacity of flange; $P_{tf} = b_f \times t \times F_{yCol}$ = **80.3** kips

Force in tension flange; $F_{tf} = M_u / (d - t) - P_u \times (b_f \times t) / A_{col}$ = **30.1** kips

Critical force in flange; $F_f = \min(P_{tf}, \max(F_{tf}, 0\text{kips}))$ = **30.1** kips

Flange weld force per in; $R_{wf} = F_f / b_f$ = **6.0** kips/in

Electrode classification number; F_{EXX} = **70.0** ksi

Design weld stress; $F_{nw}on\Omega = 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5}) / \Omega_v$ = **31.500**ksi

Design strength of weld per in; $R_{nf}on\Omega = F_{nw}on\Omega \times t_{wf} / \sqrt{(2)}$ = **8.4** kips/in

PASS - Available strength of flange weld exceeds force in flange weld

Tension weld for outset anchors

Tension flange weld leg length; t_{wf} = **0.3750** in

Effective flange weld width; $l_{Tweld,eff} = l_{Tweld,eff\_ud}$ = **2** in

Tensile load per inch; $R_{wf} = T_u / N_{bolty} / l_{Tweld,eff}$ = **3.1** kips/in

Electrode classification number; F_{EXX} = **70.0** ksi

Design weld stress; $F_{nw}on\Omega = 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5}) / \Omega_v$ = **31.500**ksi

Design strength of weld per in; $R_{nf}on\Omega = F_{nw}on\Omega \times t_{wf} / \sqrt{(2)}$ = **8.4** kips/in

PASS - Available strength of weld exceeds force in tension weld

Local stress on flange; $f_{T,local} = (T_u / N_{bolty}) / (l_{Tweld,eff} \times t)$ = **8.86** ksi

Column flange allowable stress; F_{yCol} / 1.67 = **27.54** ksi

PASS - Local column capacity exceeds local column stress

Shear weld

Shear web weld leg length; t_{ww} = **0.3750** in

Shear web weld force per in; $R_{wl} = F_v / (2 \times (d - 2 \times t))$ = **0.192** kips/in

Electrode classification number; F_{EXX} = **70.0** ksi

Design weld stress; $F_{nw}on\Omega = 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(0deg))^{1.5}) / \Omega_v$ = **21.000**ksi

Design strength of weld per in; $R_{nl}on\Omega = F_{nw}on\Omega \times t_{ww} / \sqrt{(2)}$ = **5.6** kips/in

PASS - Available strength of shear weld exceeds force in shear weld

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CUSTOM TRUSSES

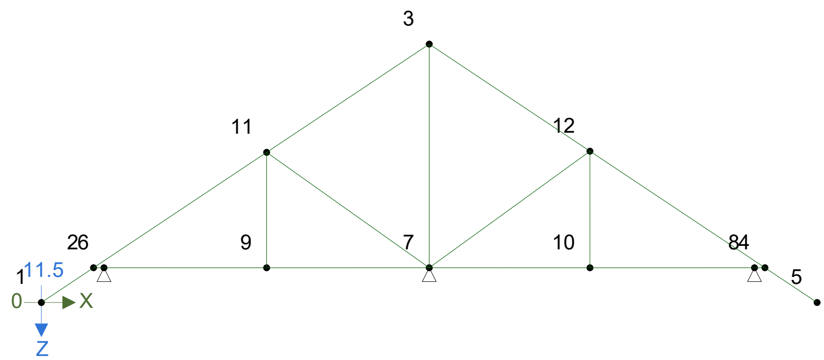
LOAD ANALYSIS

Tedds calculation version 1.0.37

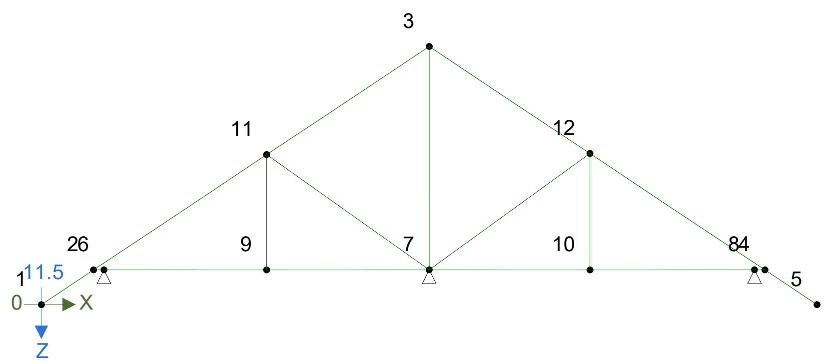
Results

Total deflection

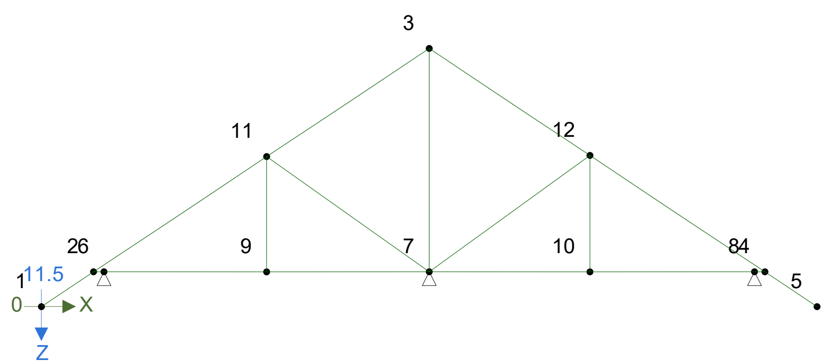
Self Weight - Total deflection



Dead - Total deflection



Roof Live - Total deflection



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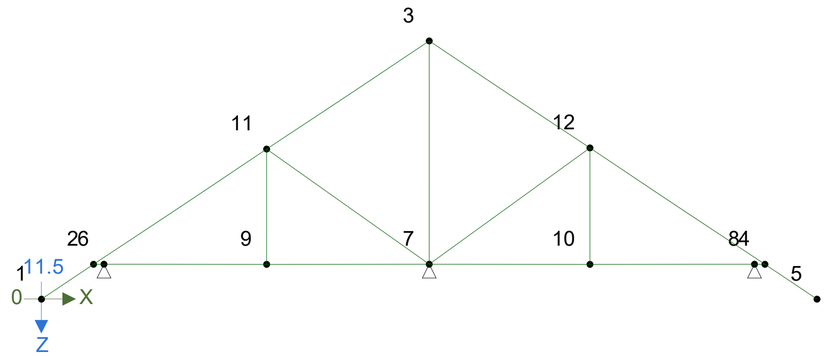
Wind - Total deflection



Node deflections

Load case: Self Weight

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
1	0	0	0	
2	0	0	0.00068	
3	0	0.001	-0.00006	
4	0	0	-0.00079	
5	0	0	0	
6	0	0	0.00084	
7	0	0	-0.00004	
8	0	0	-0.00094	
9	0	0.001	-0.00014	
10	0	0.001	0.00024	
11	0	0.001	0.00039	
12	0	0.001	-0.00028	

Load case: Dead

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
1	0.004	0.009	0	
2	0	0.003	-0.00328	
3	0	0.005	-0.00061	
4	0	0.003	0.00274	
5	-0.004	0.009	0	
6	0	0	-0.00464	
7	0	0	-0.00002	
8	0	0	0.00422	

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
9	0	0.005	0.00101	
10	0	0.005	-0.0009	
11	0	0.006	-0.00116	
12	0	0.006	0.00217	

Load case: Roof Live

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
1	0.005	0.01	0	
2	0	0.003	-0.00351	
3	0	0.005	-0.00065	
4	0	0.003	0.00293	
5	-0.004	0.01	0	
6	0	0	-0.00496	
7	0	0	-0.00002	
8	0	0	0.00452	
9	0	0.006	0.00108	
10	0	0.006	-0.00096	
11	0	0.006	-0.00124	
12	0	0.006	0.00232	

Load case: Wind

Node	Deflection X (in)	Deflection Z (in)	Rotation (°)	Co-ordinate system
1	0.006	0.012	0	
2	0	0.004	-0.00438	
3	0	0.006	-0.00082	
4	0	0.004	0.00365	
5	-0.005	0.012	0	
6	0	0	-0.00618	
7	0	0	-0.00003	
8	0	0	0.00563	
9	0	0.007	0.00135	
10	0	0.007	-0.00119	
11	0	0.007	-0.00155	
12	0	0.007	0.00289	

Total base reactions

Load case/combination	Force	
	FX (kips)	FZ (kips)
Self Weight	0	1.254
Dead	0	6.432
Roof Live	0	6.879
Wind	0	8.576

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
6	0.162	0.27	0
7	-0.002	0.714	0
8	-0.16	0.27	0

Load case: Dead

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
6	0.721	1.713	0
7	-0.009	3.007	0
8	-0.712	1.713	0

Load case: Roof Live

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
6	0.771	1.832	0
7	-0.009	3.216	0
8	-0.762	1.832	0

Load case: Wind

Node	Force		Moment
	Fx (kips)	Fz (kips)	My (kip_ft)
6	0.961	2.284	0
7	-0.011	4.009	0
8	-0.949	2.283	0

Element end forces

Load case: Self Weight

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	3	1	0	0	0

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
		2	-0.021	-0.032	-0.048
2	9.97	2	-0.229	-0.052	0.082
		11	0.158	-0.054	-0.092
3	9.36	11	0.016	-0.047	0.061
		3	-0.083	-0.053	-0.089
4	9.25	3	0.083	-0.053	0.089
		12	-0.017	-0.047	-0.061
5	10.08	12	-0.156	-0.055	0.093
		4	0.228	-0.053	-0.082
6	3	4	0.021	-0.032	0.048
		5	0	0	0
7	0.5	2	0.162	0.21	-0.033
		6	-0.162	-0.216	-0.073
8	7.79	6	0	-0.054	0.073
		9	0	-0.046	-0.042
9	7.79	9	0	-0.044	0.04
		7	0	-0.057	-0.092
10	7.7	7	0	-0.056	0.092
		10	0	-0.043	-0.04
11	7.88	10	0	-0.047	0.043
		8	0	-0.054	-0.072
12	0.5	8	0.16	-0.216	0.072
		4	-0.16	0.209	0.034
13	5.53	9	0.09	0	0.002
		11	-0.129	0	-0.001
14	10.72	7	-0.256	0	0
		3	0.18	0	0
15	5.59	10	0.09	0.001	-0.003
		12	-0.129	-0.001	-0.001
16	9.56	7	-0.264	-0.023	0.039
		11	0.232	-0.022	-0.032
17	9.52	7	-0.263	-0.023	0.039
		12	0.231	-0.021	-0.031

Load case: Dead

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	3	1	0	0	0
		2	-0.24	-0.36	-0.541

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
2	9.97	2	-1.281	-0.62	1.119
		11	0.484	-0.574	-0.891
3	9.36	11	-0.132	-0.563	0.891
		3	-0.615	-0.558	-0.866
4	9.25	3	0.612	-0.552	0.863
		12	0.127	-0.557	-0.883
5	10.08	12	-0.472	-0.58	0.897
		4	1.276	-0.627	-1.137
6	3	4	0.24	-0.36	0.541
		5	0	0	0
7	0.5	2	0.72	1.66	-0.579
		6	-0.72	-1.66	-0.252
8	7.79	6	0.001	-0.053	0.252
		9	-0.001	0.053	0.164
9	7.79	9	-0.001	0.04	-0.163
		7	0.001	-0.04	-0.147
10	7.7	7	-0.002	-0.04	0.147
		10	0.002	0.04	0.161
11	7.88	10	0.002	0.05	-0.157
		8	-0.002	-0.05	-0.236
12	0.5	8	0.712	-1.663	0.236
		4	-0.712	1.663	0.596
13	5.53	9	-0.093	-0.002	-0.001
		11	0.093	0.002	0.01
14	10.72	7	-1.605	0	0.002
		3	1.605	0	0.003
15	5.59	10	-0.09	0.004	-0.004
		12	0.09	-0.004	-0.019
16	9.56	7	-1.133	-0.002	0.012
		11	1.133	0.002	0.01
17	9.52	7	-1.129	-0.002	0.01
		12	1.129	0.002	0.005

Load case: Roof Live

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	3	1	0	0	0
		2	-0.257	-0.385	-0.578
2	9.97	2	-1.37	-0.663	1.197
		11	0.518	-0.614	-0.953

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
3	9.36	11	-0.141	-0.603	0.952
		3	-0.658	-0.597	-0.927
4	9.25	3	0.654	-0.591	0.923
		12	0.136	-0.595	-0.944
5	10.08	12	-0.504	-0.62	0.96
		4	1.365	-0.671	-1.216
6	3	4	0.257	-0.385	0.578
		5	0	0	0
7	0.5	2	0.77	1.775	-0.619
		6	-0.77	-1.775	-0.269
8	7.79	6	0.001	-0.057	0.269
		9	-0.001	0.057	0.175
9	7.79	9	-0.001	0.043	-0.174
		7	0.001	-0.043	-0.157
10	7.7	7	-0.002	-0.043	0.158
		10	0.002	0.043	0.172
11	7.88	10	0.002	0.053	-0.168
		8	-0.002	-0.053	-0.252
12	0.5	8	0.762	-1.779	0.252
		4	-0.762	1.779	0.637
13	5.53	9	-0.1	-0.002	-0.001
		11	0.1	0.002	0.011
14	10.72	7	-1.716	0	0.002
		3	1.716	0	0.003
15	5.59	10	-0.096	0.004	-0.004
		12	0.096	-0.004	-0.02
16	9.56	7	-1.212	-0.002	0.013
		11	1.212	0.002	0.01
17	9.52	7	-1.207	-0.002	0.011
		12	1.207	0.002	0.005

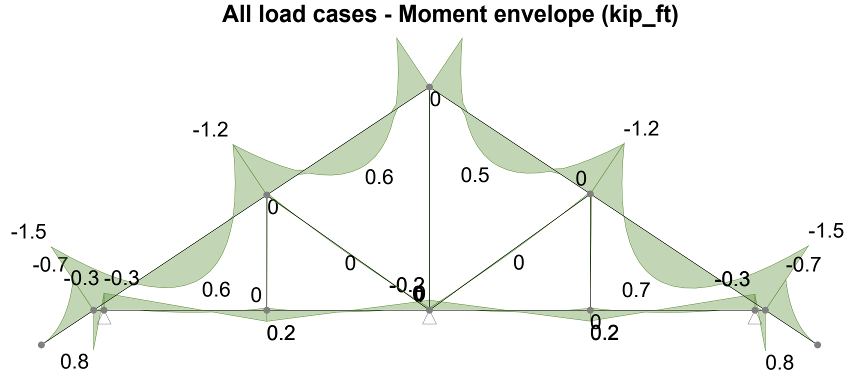
Load case: Wind

Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
1	3	1	0	0	0
		2	-0.32	-0.48	-0.721
2	9.97	2	-1.708	-0.827	1.493
		11	0.646	-0.766	-1.188
3	9.36	11	-0.176	-0.751	1.187
		3	-0.82	-0.744	-1.155

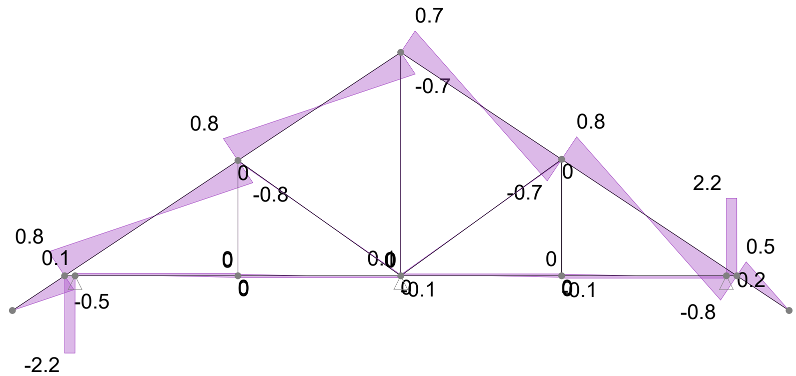
Element	Length (ft)	Nodes Start/End	Axial force (kips)	Shear force (kips)	Moment (kip_ft)
4	9.25	3	0.816	-0.736	1.151
		12	0.17	-0.742	-1.178
5	10.08	12	-0.629	-0.773	1.196
		4	1.702	-0.836	-1.516
6	3	4	0.32	-0.48	0.721
		5	0	0	0
7	0.5	2	0.96	2.214	-0.771
		6	-0.96	-2.214	-0.335
8	7.79	6	0.001	-0.071	0.335
		9	-0.001	0.071	0.219
9	7.79	9	-0.001	0.053	-0.217
		7	0.001	-0.053	-0.196
10	7.7	7	-0.003	-0.053	0.197
		10	0.003	0.053	0.215
11	7.88	10	0.003	0.067	-0.21
		8	-0.003	-0.067	-0.315
12	0.5	8	0.95	-2.218	0.315
		4	-0.95	2.218	0.795
13	5.53	9	-0.124	-0.002	-0.001
		11	0.124	0.002	0.013
14	10.72	7	-2.139	-0.001	0.002
		3	2.139	0.001	0.004
15	5.59	10	-0.12	0.005	-0.005
		12	0.12	-0.005	-0.025
16	9.56	7	-1.511	-0.003	0.016
		11	1.511	0.003	0.013
17	9.52	7	-1.505	-0.002	0.013
		12	1.505	0.002	0.007

Forces

All load cases - Moment envelope (kip_ft)



All load cases - Shear envelope (kips)



Element results

Envelope - All load cases

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
1	3	-0.48	0	0	3	-0.721
2	0	0.827	5.18	0.647	0	-1.493
3	0	0.751	4.7	0.579	0	-1.187
4	9.25	-0.742	4.61	0.546	9.25	-1.178
5	10.08	-0.836	4.84	0.674	10.08	-1.516 (min)
6	0	0.48	3	0	0	-0.721
7	0	-2.214	0	0.771	0.5	-0.335
8	0	0.071	7.79	0.219	0	-0.335
9	7.79	-0.057	0	0.217	7.79	-0.196
10	0	0.056	7.7	0.215	0	-0.197
11	0	-0.067	0	0.21	7.88	-0.315
12	0	2.218 (max abs)	0.5	0.795 (max)	0	-0.315
13	0	0.002	5.53	0.013	0	-0.002
14	0	0.001	10.72	0.004	0	-0.002
15	0	-0.005	0	0.005	5.59	-0.025
16	0	0.023	4.94	0.018	0	-0.039
17	0	0.023	4.92	0.018	0	-0.039

Envelope - All load cases

Element	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
1	0	0	3	-0.32
2	0	1.708	9.97	0.158
3	0	0.176	9.36	-0.82

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Element	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
4	9.25	0.17	0	-0.816
5	10.08	1.702	0	0.156
6	3	0	0	-0.32
7	0.5	-0.162	0	-0.96 (min)
8	0	0	0	-0.001
9	0	0.001	0	0
10	0	0.003	0	0
11	0	0	0	-0.003
12	0	-0.16	0	-0.95
13	0	0.124	5.53	-0.129
14	0	2.139 (max)	10.72	0.18
15	0	0.12	5.59	-0.129
16	0	1.511	9.56	0.232
17	0	1.505	9.52	0.231

Element results

Load case: Self Weight

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
1	3	-0.032	0	0	3	-0.048
2	9.97	-0.054	4.89	0.046	9.97	-0.092
3	9.36	-0.053	4.4	0.042	9.36	-0.089
4	0	0.053	4.91	0.04	0	-0.089
5	0	0.055	5.14	0.048 (max)	0	-0.093 (min)
6	0	0.032	3	0	0	-0.048
7	0.5	-0.216 (max abs)	0	0.033	0.5	-0.073
8	0	0.054	4.21	0.041	0	-0.073
9	7.79	-0.057	3.38	0.034	7.79	-0.092
10	0	0.056	4.37	0.031	0	-0.092
11	7.88	-0.054	3.65	0.043	7.88	-0.072
12	0	0.216	0.5	0.034	0	-0.072
13	0	0	5.53	-0.001	0	-0.002
14	0	0	10.72	0	0	0
15	0	-0.001	0	0.003	5.59	-0.001
16	0	0.023	4.94	0.018	0	-0.039
17	0	0.023	4.92	0.018	0	-0.039

Load case: Self Weight

Element	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
1	0	0	3	-0.021
2	0	0.229	9.97	0.158
3	0	-0.016	9.36	-0.083
4	9.25	-0.017	0	-0.083
5	10.08	0.228	0	0.156
6	3	0	0	-0.021
7	0.5	-0.162 (min)	0	-0.162 (min)
8	0	0	0	0
9	0	0	0	0
10	0	0	0	0
11	0	0	0	0
12	0	-0.16	0.5	-0.16
13	0	-0.09	5.53	-0.129
14	0	0.256	10.72	0.18
15	0	-0.09	5.59	-0.129
16	0	0.264 (max)	9.56	0.232
17	0	0.263	9.52	0.231

Element results

Load case: Dead

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
1	3	-0.36	0	0	3	-0.541
2	0	0.62	5.18	0.485	0	-1.119
3	0	0.563	4.7	0.434	0	-0.891
4	9.25	-0.557	4.61	0.409	9.25	-0.883
5	10.08	-0.627	4.84	0.506	10.08	-1.137 (min)
6	0	0.36	3	0	0	-0.541
7	0	-1.66	0	0.579	0.5	-0.252
8	0	0.053	7.79	0.164	0	-0.252
9	0	-0.04	0	0.163	7.79	-0.147
10	0	0.04	7.7	0.161	0	-0.147
11	0	-0.05	0	0.157	7.88	-0.236
12	0	1.663 (max abs)	0.5	0.596 (max)	0	-0.236
13	0	0.002	5.53	0.01	0	0.001
14	0	0	10.72	0.003	0	-0.002
15	0	-0.004	0	0.004	5.59	-0.019

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
16	0	0.002	9.56	0.01	0	-0.012
17	0	0.002	9.52	0.005	0	-0.01

Load case: Dead

Element	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
1	0	0	3	-0.24
2	0	1.281	9.97	0.484
3	0	0.132	9.36	-0.615
4	9.25	0.127	0	-0.612
5	10.08	1.276	0	0.472
6	3	0	0	-0.24
7	0	-0.72 (min)	0	-0.72 (min)
8	0	-0.001	0	-0.001
9	0	0.001	0	0.001
10	0	0.002	0	0.002
11	0	-0.002	0	-0.002
12	0	-0.712	0	-0.712
13	0	0.093	0	0.093
14	0	1.605 (max)	0	1.605 (max)
15	0	0.09	0	0.09
16	0	1.133	0	1.133
17	0	1.129	0	1.129

Element results

Load case: Roof Live

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
1	3	-0.385	0	0	3	-0.578
2	0	0.663	5.18	0.519	0	-1.197
3	0	0.603	4.7	0.464	0	-0.952
4	9.25	-0.595	4.61	0.438	9.25	-0.944
5	10.08	-0.671	4.84	0.541	10.08	-1.216 (min)
6	0	0.385	3	0	0	-0.578
7	0	-1.775	0	0.619	0.5	-0.269
8	0	0.057	7.79	0.175	0	-0.269
9	0	-0.043	0	0.174	7.79	-0.157
10	0	0.043	7.7	0.172	0	-0.158

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
11	0	-0.053	0	0.168	7.88	-0.252
12	0	1.779 (max abs)	0.5	0.637 (max)	0	-0.252
13	0	0.002	5.53	0.011	0	0.001
14	0	0	10.72	0.003	0	-0.002
15	0	-0.004	0	0.004	5.59	-0.02
16	0	0.002	9.56	0.01	0	-0.013
17	0	0.002	9.52	0.005	0	-0.011

Load case: Roof Live

Element	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
1	0	0	3	-0.257
2	0	1.37	9.97	0.518
3	0	0.141	9.36	-0.658
4	9.25	0.136	0	-0.654
5	10.08	1.365	0	0.504
6	3	0	0	-0.257
7	0	-0.77 (min)	0	-0.77 (min)
8	0	-0.001	0	-0.001
9	0	0.001	0	0.001
10	0	0.002	0	0.002
11	0	-0.002	0	-0.002
12	0	-0.762	0	-0.762
13	0	0.1	0	0.1
14	0	1.716 (max)	0	1.716 (max)
15	0	0.096	0	0.096
16	0	1.212	0	1.212
17	0	1.207	0	1.207

Element results

Load case: Wind

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
1	3	-0.48	0	0	3	-0.721
2	0	0.827	5.18	0.647	0	-1.493
3	0	0.751	4.7	0.579	0	-1.187
4	9.25	-0.742	4.61	0.546	9.25	-1.178
5	10.08	-0.836	4.84	0.674	10.08	-1.516 (min)

Element	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
6	0	0.48	3	0	0	-0.721
7	0	-2.214	0	0.771	0.5	-0.335
8	0	0.071	7.79	0.219	0	-0.335
9	0	-0.053	0	0.217	7.79	-0.196
10	0	0.053	7.7	0.215	0	-0.197
11	0	-0.067	0	0.21	7.88	-0.315
12	0	2.218 (max abs)	0.5	0.795 (max)	0	-0.315
13	0	0.002	5.53	0.013	0	0.001
14	0	0.001	10.72	0.004	0	-0.002
15	0	-0.005	0	0.005	5.59	-0.025
16	0	0.003	9.56	0.013	0	-0.016
17	0	0.002	9.52	0.007	0	-0.013

Load case: Wind

Element	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
1	0	0	3	-0.32
2	0	1.708	9.97	0.646
3	0	0.176	9.36	-0.82
4	9.25	0.17	0	-0.816
5	10.08	1.702	0	0.629
6	3	0	0	-0.32
7	0	-0.96 (min)	0	-0.96 (min)
8	0	-0.001	0	-0.001
9	0	0.001	0	0.001
10	0	0.003	0	0.003
11	0	-0.003	0	-0.003
12	0	-0.95	0	-0.95
13	0	0.124	0	0.124
14	0	2.139 (max)	0	2.139 (max)
15	0	0.12	0	0.12
16	0	1.511	0	1.511
17	0	1.505	0	1.505

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TOP CHORD DESIGN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation; 0.506

Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	725	354	0.488	PASS
Shear stress	lb/in^2	170	31	0.180	PASS
Bearing stress	lb/in^2	419	38	0.091	PASS
Compressive stress	lb/in^2	637	41	0.065	PASS
Bending and axial force				0.506	PASS

Design section 1

Member details

Service condition; Wet

Load duration - Table 2.3.2; Ten years

Sawn lumber section details

Number of sections in member; N = **1**

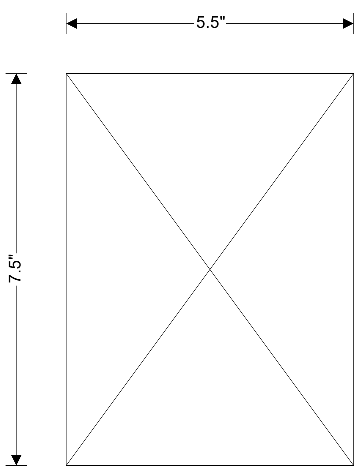
Nominal breadth of sections; b_{nom} = **6** in

Breadth of sections; b = **5.5** in

Nominal depth of sections; d_{nom} = **8** in

Depth of sections; d = **7.5** in

Material; **Douglas Fir-Larch North, Posts and timbers, No.2 grade**



6"x8" sawn lumber section
Cross-sectional area, A, 41.25 in^2
Section modulus, S_x, 51.6 in^3
Section modulus, S_y, 37.8 in^3
Second moment of area, I_x, 193.4 in^4
Second moment of area, I_y, 104 in^4
Radius of gyration, r_x, 2.165 in
Radius of gyration, r_y, 1.588 in
Douglas Fir-Larch North, Posts and timbers, No.2 grade
Bending, F_b, 725 psi
Shear parallel to grain, F_v, 170 psi
Compression parallel to grain, F_c, 700 psi
Compression perpendicular to grain, F_{c_perp}, 625 psi
Tension parallel to grain, F_t, 475 psi
Modulus of elasticity, E, 1300000 psi
Minimum modulus of elasticity, E_{min}, 470000 psi
Density, ρ, 35.104 lbm/ft^3
Specific gravity, G, 0.49

Span details

Unbraced length - Major axis; L_x = **10** ft

Effective bending length - Major axis; $L_{e,x}$ = 1.63 ′ L_x + 3 ′ b = **17.675** ft

Column buckling length - Major axis; $L_{b,x}$ = L_x = **10** ft

Unbraced length - Minor axis; L_y = **0** ft

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LEE'S SUMMIT, MO 64063
816.444.3144
WWW.LEOK.COM

Bearing length; L_b = **4** in

Analysis results

Design bending moment - Major axis; M_x = **1520** lb_ft

Design shear force - Major axis; V_x = **840** lb

Design perpendicular compression - Major axis; R_x = **840** lb

Design axial compression force; P = **1710** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2; C_D = **1**

Wet service factor for bending - Table 4D; C_{Mb} = **1**

Wet service factor for shear - Table 4D; C_{Mv} = **1**

Wet service factor for compression - Table 4D; C_{Mc} = **0.91**

Wet service factor for perp.comp. - Table 4D; C_{Mc_perp} = **0.67**

Wet service factor for mod.of elasticity - Table 4D; C_{ME} = **1**

Reference compression design value; $F_c^* = F_c$ ´ C_D ´ C_{Mc} = **637** lb/in^2

Adjusted modulus of elasticity; $E_{min}' = E_{min}$ = **470000** lb/in^2

Critical buckling design value; F_{cE} = 0.822 ´ E_{min}' = **386340** lb/in^2

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{(((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8)}$ = **1**

Compression members - General - cl.3.6

Design axial compression force; P = **1710** lb

Design compression parallel to grain - Table 4.3.1; $F_c' = F_c$ ´ C_D ´ C_{Mc} ´ C_P = **637** lb/in^2

Actual compression parallel to grain; $f_c = P / (b$ ´ $d)$ = **41** lb/in^2

f_c / F_c' = **0.065**

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **1520** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b$ ´ C_D = **725** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **354** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.488**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; V_x = **840** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v$ ´ C_D = **170** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x}$ = 3 ´ V_x / (2 ´ b ´ d) = **31** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.180**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **840** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ ´ C_{Mc_perp} = **419** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x / (b$ ´ $L_b)$ = **38** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.091**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis; $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2 =$ **1509** lb/in^2

Critical buckling design value in y-axis; $F_{cE2} = 0.822 \times E_{min}' =$ **386340** lb/in^2

Bending and compression check - eqs.3.9-3 and 3.9-4

$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2})) =$ **0.506**; < 1.0

PASS - Combined bending and compressive stresses are within permissible limits

BOTTOM CHORD DESIGN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation; 0.475

Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	725	78	0.108	PASS
Shear stress	lb/in^2	170	81	0.475	PASS
Bearing stress	lb/in^2	419	101	0.241	PASS
Tensile stress	lb/in^2	475	23	0.049	PASS
Bending and axial force				0.157	PASS

Design section 1

Member details

Service condition; Wet

Load duration - Table 2.3.2; Ten years

Sawn lumber section details

Number of sections in member; N = **1**

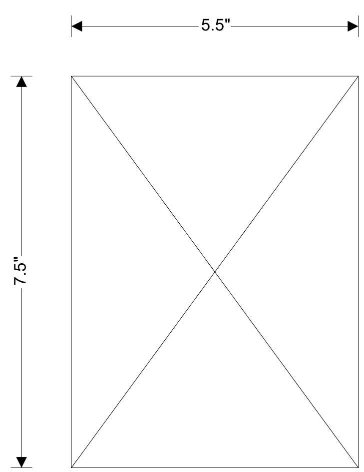
Nominal breadth of sections; b_{nom} = **6** in

Breadth of sections; b = **5.5** in

Nominal depth of sections; d_{nom} = **8** in

Depth of sections; d = **7.5** in

Material; **Douglas Fir-Larch North, Posts and timbers, No.2 grade**



6"x8" sawn lumber section
Cross-sectional area, A, 41.25 in^2
Section modulus, S_x, 51.6 in^3
Section modulus, S_y, 37.8 in^3
Second moment of area, I_x, 193.4 in^4
Second moment of area, I_y, 104 in^4
Radius of gyration, r_x, 2.165 in
Radius of gyration, r_y, 1.588 in
Douglas Fir-Larch North, Posts and timbers, No.2 grade
Bending, F_b, 725 psi
Shear parallel to grain, F_v, 170 psi
Compression parallel to grain, F_c, 700 psi
Compression perpendicular to grain, F_{c_perp}, 625 psi
Tension parallel to grain, F_t, 475 psi
Modulus of elasticity, E, 1300000 psi
Minimum modulus of elasticity, E_{min}, 470000 psi
Density, ρ, 35.104 lbm/ft^3
Specific gravity, G, 0.49

Span details

Unbraced length - Major axis; L_x = **8** ft

Effective bending length - Major axis; $L_{e,x} = 1.63 \times L_x + 3 \times b$ = **14.415** ft

Column buckling length - Major axis; $L_{b,x} = L_x$ = **8** ft

Unbraced length - Minor axis; L_y = **0** ft

Bearing length; L_b = **4** in

Analysis results

Design bending moment - Major axis; M_x = **335** lb_ft

Design shear force - Major axis; V_x = **2220** lb

Design perpendicular compression - Major axis; R_x = **2220** lb

Design axial tension force; P = **960** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2; C_D = **1**

Wet service factor for bending - Table 4D; C_{Mb} = **1**

Wet service factor for shear - Table 4D; C_{Mv} = **1**

Wet service factor for tension - Table 4D; C_{Mt} = **1**

Wet service factor for perp.comp. - Table 4D; C_{Mc_perp} = **0.67**

Tension members - cl.3.8

Design axial tension force; P = **960** lb

Design tension parallel to grain - Table 4.3.1; $F_t' = F_t$ ´ C_D = **475** lb/in^2

Actual tension parallel to grain; f_t = P / (b ´ d) = **23** lb/in^2

f_t / F_t' = **0.049**

PASS - Design tension stress exceeds actual tension stress

Bending members - Flexure - cl.3.3

Design bending moment; M_x = **335** lb_ft

Design bending stress - Table 4.3.1; $F_{b,x}' = F_b$ ´ C_D = **725** lb/in^2

Actual bending stress - eq.3.3-2; $f_{b,x} = M_x / S_x$ = **78** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.108**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force; V_x = **2220** lb

Design shear stress - Table 4.3.1; $F_{v,x}' = F_v$ ´ C_D = **170** lb/in^2

Actual shear stress - eq.3.4-2; $f_{v,x}$ = 3 ´ V_x / (2 ´ b ´ d) = **81** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.475**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression; R_x = **2220** lb

Design bearing stress - Table 4.3.1; $F_{c\_perp,x}' = F_{c\_perp}$ ´ C_{Mc_perp} = **419** lb/in^2

Actual bearing stress; $f_{c\_perp,x} = R_x$ / (b ´ L_b) = **101** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.241**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Combined bending and axial loading - cl.3.9

Reference bending design value without C_L; $F_{bt1}^* = F_b$ ´ C_D = **725** lb/in^2

Reference bending design value without C_V; $F_{bt1}^{**} = F_b$ ´ C_D = **725** lb/in^2

Bending and axial tension - eq.3.9-1 and 3.9-2; max($f_t / F_t' + f_{b,x} / F_{bt1}^*$, $(f_{b,x} - f_t) / F_{bt1}^{**}$) = **0.157**; < 1.0

PASS - Combined bending and tensile stresses are within permissible limits

LEOK
LEIGH + O'KANE
209 SE DOUGLAS
LEE'S SUMMIT, MO 64063
816.444.3144
WWW.LEOK.COM

VERTICAL MEMBER DESIGN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation; 0.157

Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	725	6	0.009	PASS
Shear stress	lb/in^2	170	0	0.003	PASS
Bearing stress	lb/in^2	419	0	0.001	PASS
Compressive stress	lb/in^2	451	71	0.157	PASS
Bending and axial force				0.035	PASS

Design section 1

Member details

Service condition; Wet

Load duration - Table 2.3.2; Ten years

Sawn lumber section details

Number of sections in member; N = **1**

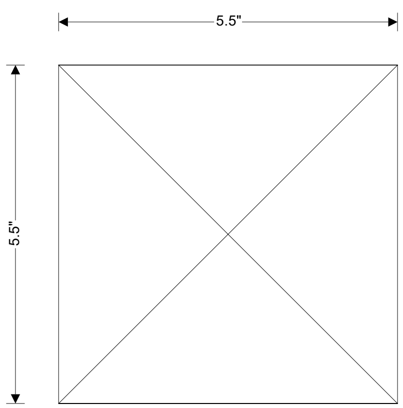
Nominal breadth of sections; b_{nom} = **6** in

Breadth of sections; b = **5.5** in

Nominal depth of sections; d_{nom} = **6** in

Depth of sections; d = **5.5** in

Material; **Douglas Fir-Larch North, Posts and timbers, No.2 grade**



6"x6" sawn lumber section
Cross-sectional area, A, 30.25 in^2
Section modulus, S_x, 27.7 in^3
Section modulus, S_y, 27.7 in^3
Second moment of area, I_x, 76.3 in^4
Second moment of area, I_y, 76.3 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 1.588 in
Douglas Fir-Larch North, Posts and timbers, No.2 grade
Bending, F_b, 725 psi
Shear parallel to grain, F_v, 170 psi
Compression parallel to grain, F_c, 700 psi
Compression perpendicular to grain, F_{c_perp}, 625 psi
Tension parallel to grain, F_t, 475 psi
Modulus of elasticity, E, 1300000 psi
Minimum modulus of elasticity, E_{min}, 470000 psi
Density, ρ, 35.104 lbm/ft^3
Specific gravity, G, 0.49

Span details

Unbraced length - Major axis; L_x = **11** ft

Effective bending length - Major axis; $L_{e,x}$ = 1.63 ´ L_x + 3 ´ b = **19.305** ft

Column buckling length - Major axis; $L_{b,x}$ = L_x = **11** ft

Unbraced length - Minor axis; L_y = **0** ft

Bearing length; L_b = **4** in

Analysis results

Design bending moment - Major axis;	M_x = **15** lb_ft
Design shear force - Major axis;	V_x = **10** lb
Design perpendicular compression - Major axis;	R_x = **10** lb
Design axial compression force;	P = **2140** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2;	C_D = **1**
Wet service factor for bending - Table 4D;	C_{Mb} = **1**
Wet service factor for shear - Table 4D;	C_{Mv} = **1**
Wet service factor for compression - Table 4D;	C_{Mc} = **0.91**
Wet service factor for perp.comp. - Table 4D;	C_{Mc_perp} = **0.67**
Wet service factor for mod.of elasticity - Table 4D;	C_{ME} = **1**
Reference compression design value;	$F_c^* = F_c ´ C_D ´ C_{Mc}$ = **637** lb/in^2
Adjusted modulus of elasticity;	$E_{min}' = E_{min}$ = **470000** lb/in^2
Critical buckling design value;	$F_{cE} = 0.822 ´ E_{min}' / (L_{b,x} / d)^2$ = **671** lb/in^2
Column stability factor - eq.3.7-1	

$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{(((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8)}$ = **0.709**

Compression members - General - cl.3.6

Design axial compression force;	P = **2140** lb
Design compression parallel to grain - Table 4.3.1;	$F_c' = F_c ´ C_D ´ C_{Mc} ´ C_P$ = **451** lb/in^2
Actual compression parallel to grain;	$f_c = P / (b ´ d)$ = **71** lb/in^2
	f_c / F_c' = **0.157**

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment;	M_x = **15** lb_ft
Design bending stress - Table 4.3.1;	$F_{b,x}' = F_b ´ C_D$ = **725** lb/in^2
Actual bending stress - eq.3.3-2;	$f_{b,x} = M_x / S_x$ = **6** lb/in^2
	$f_{b,x} / F_{b,x}'$ = **0.009**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;	V_x = **10** lb
Design shear stress - Table 4.3.1;	$F_{v,x}' = F_v ´ C_D$ = **170** lb/in^2
Actual shear stress - eq.3.4-2;	$f_{v,x} = 3 ´ V_x / (2 ´ b ´ d)$ = **0** lb/in^2
	$f_{v,x} / F_{v,x}'$ = **0.003**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;	R_x = **10** lb
Design bearing stress - Table 4.3.1;	$F_{c\_perp,x}' = F_{c\_perp} ´ C_{Mc\_perp}$ = **419** lb/in^2
Actual bearing stress;	$f_{c\_perp,x} = R_x / (b ´ L_b)$ = **0** lb/in^2
	$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.001**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis;	$F_{cE1} = 0.822 ´ E_{min}' / (L_{b,x} / d)^2$ = **671** lb/in^2

Critical buckling design value in y-axis; $F_{cE2} = 0.822 \times E_{min}' =$ **386340** lb/in^2

Bending and compression check - eqs.3.9-3 and 3.9-4

$max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2})) =$ **0.035**; < 1.0

PASS - Combined bending and compressive stresses are within permissible limits

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209 SE DOUGLAS
LEE'S SUMMIT, MO 64063
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DIAGONAL MEMBER DESIGN

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation; 0.129

Overall design status; PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	725	17	0.024	PASS
Shear stress	lb/in^2	170	1	0.007	PASS
Bearing stress	lb/in^2	419	1	0.003	PASS
Tensile stress	lb/in^2	475	50	0.105	PASS
Bending and axial force				0.129	PASS

Design section 1

Member details

Service condition; Wet

Load duration - Table 2.3.2; Ten years

Sawn lumber section details

Number of sections in member; N = **1**

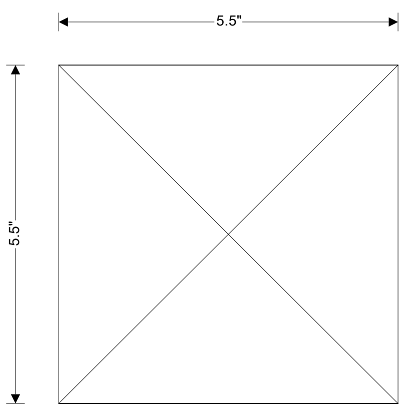
Nominal breadth of sections; b_{nom} = **6** in

Breadth of sections; b = **5.5** in

Nominal depth of sections; d_{nom} = **6** in

Depth of sections; d = **5.5** in

Material; **Douglas Fir-Larch North, Posts and timbers, No.2 grade**



6"x6" sawn lumber section
Cross-sectional area, A, 30.25 in^2
Section modulus, S_x, 27.7 in^3
Section modulus, S_y, 27.7 in^3
Second moment of area, I_x, 76.3 in^4
Second moment of area, I_y, 76.3 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 1.588 in
Douglas Fir-Larch North, Posts and timbers, No.2 grade
Bending, F_b, 725 psi
Shear parallel to grain, F_v, 170 psi
Compression parallel to grain, F_c, 700 psi
Compression perpendicular to grain, F_{c_perp}, 625 psi
Tension parallel to grain, F_t, 475 psi
Modulus of elasticity, E, 1300000 psi
Minimum modulus of elasticity, E_{min}, 470000 psi
Density, ρ, 35.104 lbm/ft^3
Specific gravity, G, 0.49

Span details

Unbraced length - Major axis; L_x = **9.5** ft

Effective bending length - Major axis; $L_{e,x}$ = 1.63 ´ L_x + 3 ´ b = **16.86** ft

Column buckling length - Major axis; $L_{b,x}$ = L_x = **9.5** ft

Unbraced length - Minor axis; L_y = **0** ft

Bearing length; L_b = **4** in

Analysis results

Design bending moment - Major axis;	M_x = **40** lb_ft
Design shear force - Major axis;	V_x = **25** lb
Design perpendicular compression - Major axis;	R_x = **25** lb
Design axial tension force;	P = **1510** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2;	C_D = **1**
Wet service factor for bending - Table 4D;	C_{Mb} = **1**
Wet service factor for shear - Table 4D;	C_{Mv} = **1**
Wet service factor for tension - Table 4D;	C_{Mt} = **1**
Wet service factor for perp.comp. - Table 4D;	C_{Mc_perp} = **0.67**

Tension members - cl.3.8

Design axial tension force;	P = **1510** lb
Design tension parallel to grain - Table 4.3.1;	F_t' = F_t ´ C_D = **475** lb/in^2
Actual tension parallel to grain;	f_t = P / (b ´ d) = **50** lb/in^2
	f_t / F_t' = **0.105**

PASS - Design tension stress exceeds actual tension stress

Bending members - Flexure - cl.3.3

Design bending moment;	M_x = **40** lb_ft
Design bending stress - Table 4.3.1;	$F_{b,x}'$ = F_b ´ C_D = **725** lb/in^2
Actual bending stress - eq.3.3-2;	$f_{b,x}$ = M_x / S_x = **17** lb/in^2
	$f_{b,x}$ / $F_{b,x}'$ = **0.024**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force;	V_x = **25** lb
Design shear stress - Table 4.3.1;	$F_{v,x}'$ = F_v ´ C_D = **170** lb/in^2
Actual shear stress - eq.3.4-2;	$f_{v,x}$ = 3 ´ V_x / (2 ´ b ´ d) = **1** lb/in^2
	$f_{v,x}$ / $F_{v,x}'$ = **0.007**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression;	R_x = **25** lb
Design bearing stress - Table 4.3.1;	$F_{c\_perp,x}'$ = F_{c_perp} ´ C_{Mc_perp} = **419** lb/in^2
Actual bearing stress;	$f_{c\_perp,x}$ = R_x / (b ´ L_b) = **1** lb/in^2
	$f_{c\_perp,x}$ / $F_{c\_perp,x}'$ = **0.003**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Combined bending and axial loading - cl.3.9

Reference bending design value without C_L;	F_{bt1}^* = F_b ´ C_D = **725** lb/in^2
Reference bending design value without C_V;	F_{bt1}^{**} = F_b ´ C_D = **725** lb/in^2
Bending and axial tension - eq.3.9-1 and 3.9-2;	max(f_t / F_t' + $f_{b,x}$ / F_{bt1}^*, ($f_{b,x}$ - f_t) / F_{bt1}^{**}) = **0.129**; < 1.0

PASS - Combined bending and tensile stresses are within permissible limits