STRUCTURAL ENGINEERS

Project WSS Lee's Summit, MO Project No. 23-283

Calc. By GL Checked By DN Date 8/1/2023

STRUCTURAL CALCULATIONS FOR :

WSS Lee's Summit, MO
Lee's Summit, MO

STATE OF MISSOURI
STEVEN N. BUSEY
NUMBER
E-25462
PROFESSIONAL ENGINEER
08.14.23

LOADS

Seismic

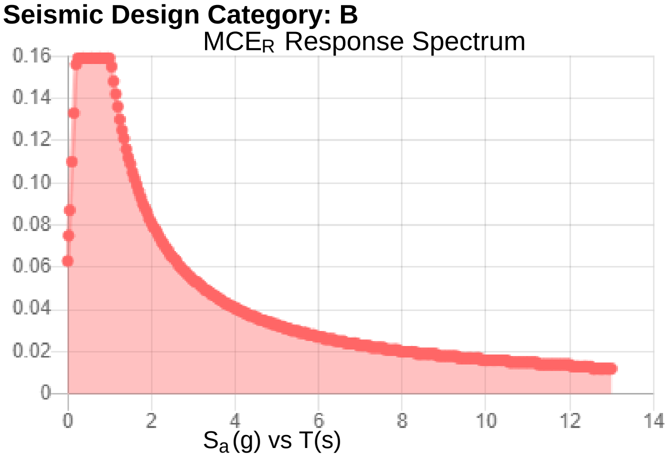
D - Stiff Soil

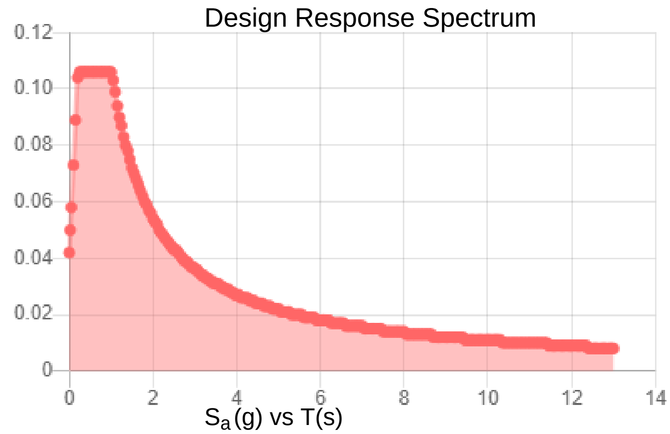
Site Soil Class:

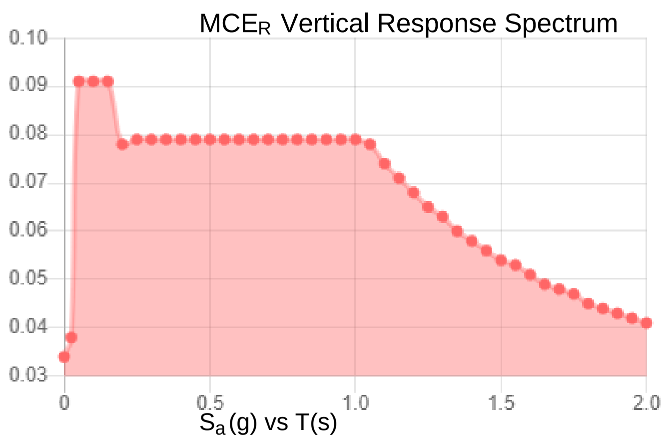
Results:

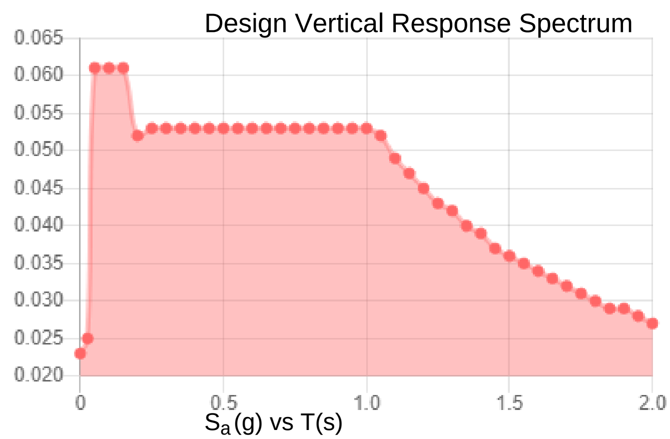
S_S :	0.099	S_{D1} :	0.109
S_1 :	0.068	T_L :	12
F_a :	1.6	PGA :	0.047
F_v :	2.4	PGA_M :	0.075
S_{MS} :	0.159	F_{PGA} :	1.6
S_{M1} :	0.163	I_e :	1
S_{DS} :	0.106	C_v :	0.7

Seismic Design Category: B









Data Accessed: **Mon Jun 26 2023**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

SEISMIC FORCES

In accordance with ASCE 7-16

Tedds calculation version 3.1.04

Site parameters

Site class	D
Mapped acceleration parameters (Section 11.4.2)	
at short period	S_S = **0.099**
at 1 sec period	S_1 = **0.068**
Site coefficientat short period (Table 11.4-1)	F_a = **1.600**
at 1 sec period (Table 11.4-2)	F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S =$ **0.158**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1 =$ **0.163**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS} =$ **0.106**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1} =$ **0.109**

Seismic design category

Occupancy category (Table 1-1)	II

Seismic design category based on short period response acceleration (Table 11.6-1)

A

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

B

Seismic design category	B

Approximate fundamental period

Height above base to highest level of building	h_n = **36.56** ft

From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**
Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x =$ **0.297** sec
Building fundamental period (Sect 12.8.2)	$T = T_a =$ **0.297** sec
Long-period transition period	T_L = **6** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems
	15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1)	R = **6.5**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2)	$C_{s\_calc} = S_{DS} / (R / I_e) =$ **0.0162**
Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e)) =$ **0.0563**
Minimum (Eq.12.8-5)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) =$ **0.0100**
Seismic response coefficient	C_s = **0.0162**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **2338.9** kips

Seismic response coefficient C_s = **0.0162**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **38.0** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12) $C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$

Lateral force induced at level i (Eq 12.8-11) $F_x = C_{vx} \times V$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x
1	9.6;	483.9;	1.00;	0.081;	3.1
2	19.0;	596.6;	1.00;	0.199;	7.6
3	28.5;	618.2;	1.00;	0.309;	11.8
4	36.6;	640.3;	1.00;	0.410;	15.6

Snow

Results:

Ground Snow Load, p_g :	20 lb/ft^2
Mapped Elevation:	956.7 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Mon Jun 26 2023

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported 'elevation' and 'mapped elevation' differ significantly from each other.

SNOW LOADING

In accordance with ASCE7-16

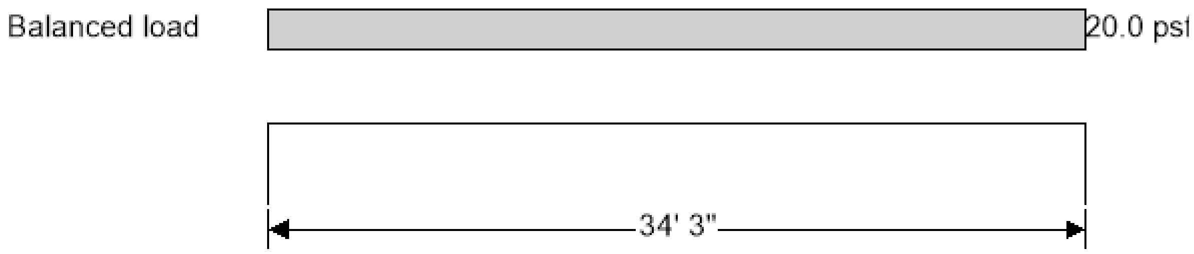
Tedds calculation version 1.0.11

Building details

Roof type	Flat
Width of roof	b = **34.25** ft

Ground snow load

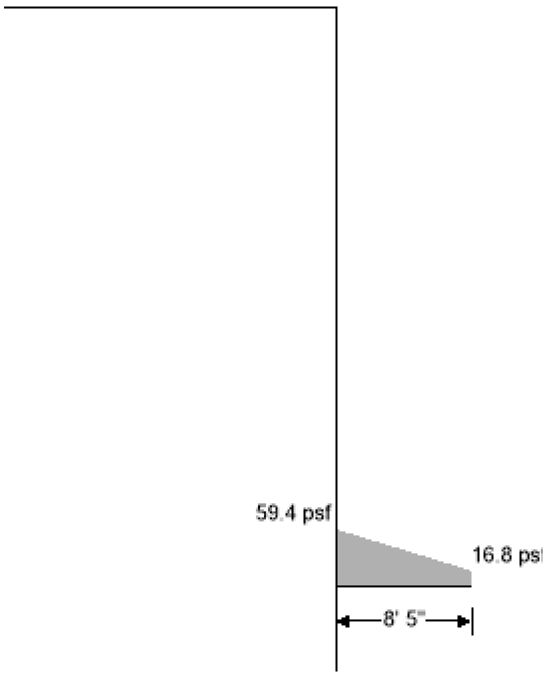
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	Unheated structures
Thermal factor (Table 7.3-2)	C_t = **1.20**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **16.80** lb/ft^2



Roof elevation

Drift calculations

Balanced snow load height	$h_b = p_f / \gamma$ = **1.01** ft
Length of upper roof	l_u = **65.90** ft
Length of lower roof	l_l = **8.42** ft
Height diff between uppper and lower roofs	h_{diff} = **35.60** ft
Height from balance load to top of upper roof	$h_c = h_{diff} - h_b$ = **34.59** ft
Drift height leeward drift	$h_{d\_l} = \min(\sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}), 0.6 \times l_l)$ = **2.56** ft
Drift height windward drift	$h_{d\_w} = \min(0.75 \times \sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}), \sqrt{(I_s \times p_g \times l_l / (4 \times \gamma))})$ = **0.92** ft
Maximum lw/ww drift height	$h_{d\_max} = \max(h_{d\_w}, h_{d\_l})$ = **2.56** ft
Drift height	$h_d = \min(h_{d\_max}, h_c)$ = **2.56** ft
Drift width	$W_d = \min(4 \times h_{d\_max}, 8 \times h_c)$ = **10.26** ft
Drift surcharge load	$p_d = h_d \times \gamma$ = **42.58** lb/ft^2



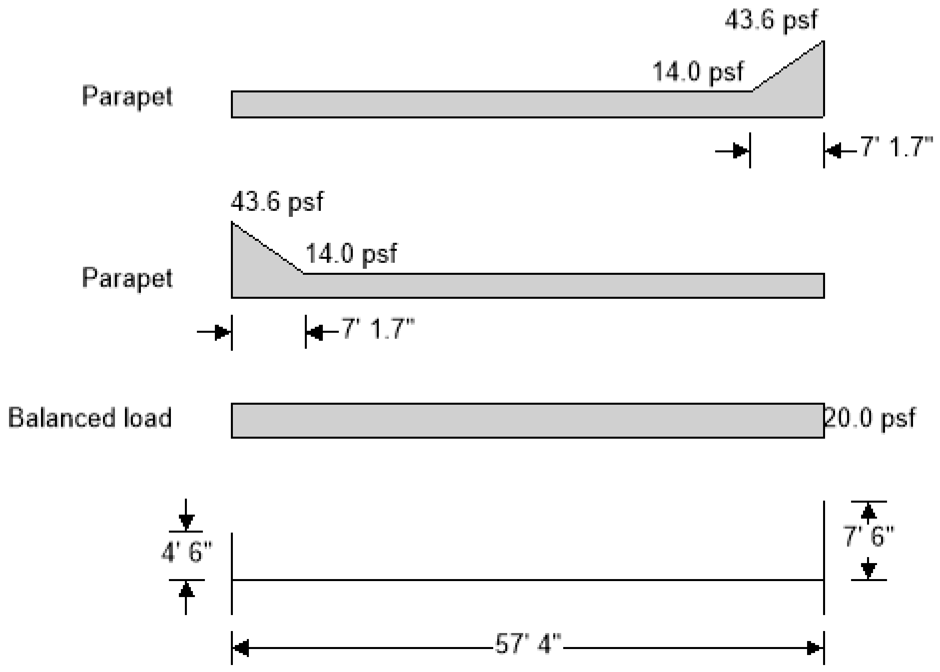
Elevation on snow drift

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.11

Building details	
Roof type	Flat
Width of roof	b = **57.33** ft
Ground snow load	
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	All
Thermal factor (Table 7.3-2)	C_t = **1.00**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **14.00** lb/ft^2
Left parapet	
Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height of left parapet	h_{pptL} = **4.50** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **3.66** ft
Length of roof - left parapet	$l_{u\_pptL} = b$ = **57.33** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.79** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.79** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **7.14** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **29.64** lb/ft^2
Right parapet	
Height of right parapet	h_{pptR} = **7.50** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **6.66** ft
Length of roof - right parapet	$l_{u\_pptR} = b$ = **57.33** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.79** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.79** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **7.14** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **29.64** lb/ft^2



Roof elevation

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.11

Building details

Roof type	Flat
Width of roof	b = **57.33** ft

Ground snow load

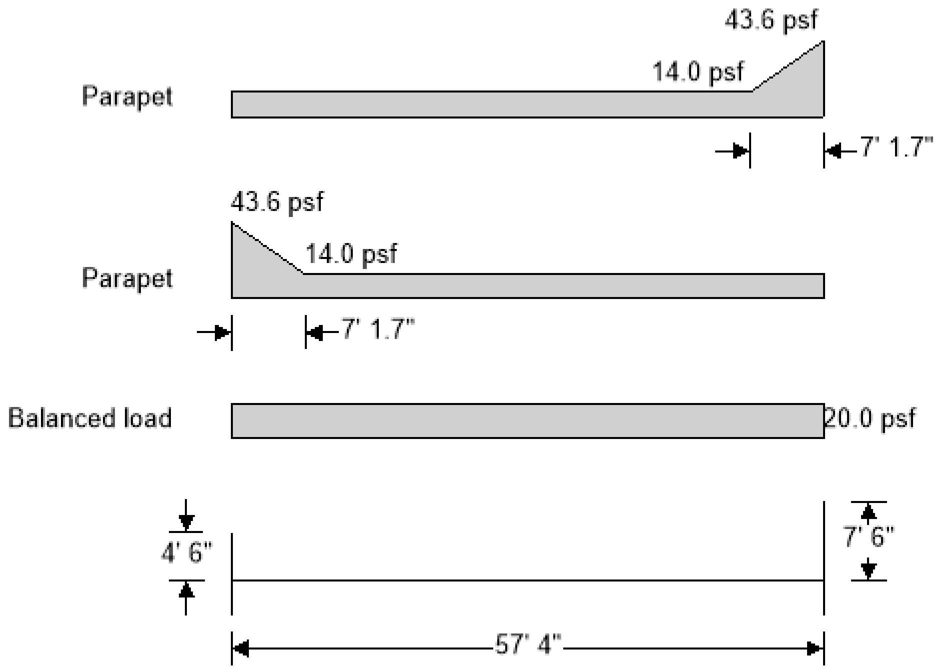
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	All
Thermal factor (Table 7.3-2)	C_t = **1.00**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **14.00** lb/ft^2

Left parapet

Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height of left parapet	h_{pptL} = **4.50** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **3.66** ft
Length of roof - left parapet	$l_{u\_pptL} = b$ = **57.33** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.79** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.79** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **7.14** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **29.64** lb/ft^2

Right parapet

Height of right parapet	h_{pptR} = **7.50** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **6.66** ft
Length of roof - right parapet	$l_{u\_pptR} = b$ = **57.33** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.79** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.79** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **7.14** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **29.64** lb/ft^2



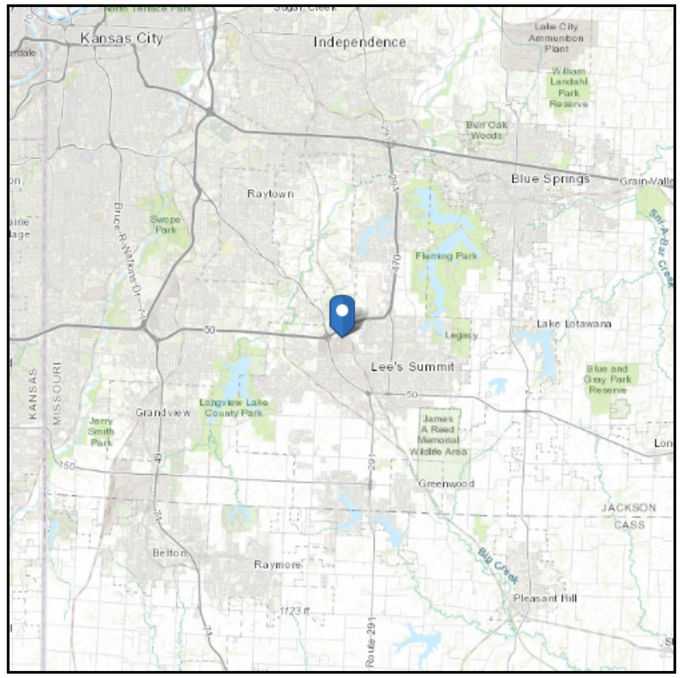
Roof elevation

ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 38.933407
Longitude: -94.396221
Elevation: 956.6529147073804 ft (NAVD 88)



Wind

Results:

Wind Speed	109 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	88 Vmph
100-year MRI	94 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Mon Jun 26 2023

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

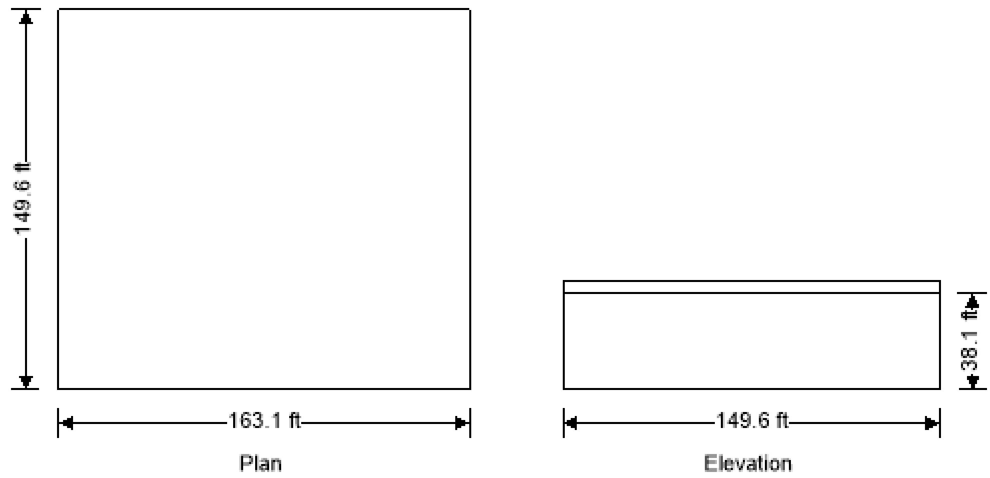
Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.13



Building data

Type of roof	Flat
Length of building	b = **163.13** ft
Width of building	d = **149.63** ft
Height to eaves	H = **38.08** ft
Height of parapet	h_p = **4.50** ft
Mean height	h = **38.08** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **14.96** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **957** ft
Ground elevation factor	K_e = exp(-0.0000362 × z_{gl}/1ft) = **0.97**
Exposure category (cl 26.7.3)	C
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**
Parapet internal pressure coef +ve (Table 26.11-1)	GC_{pi_pp} = **0.18**
Parapet internal pressure coef –ve (Table 26.11-1)	GC_{pi_np} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **1.03**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **25.7** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **1.05**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **26.3** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **25.68** psf

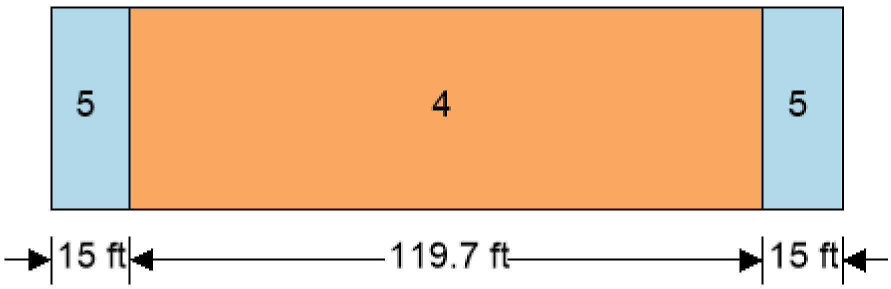
Equations used in tables

Net pressure $p = q_h \times [GC_p - GC_{pi}]$

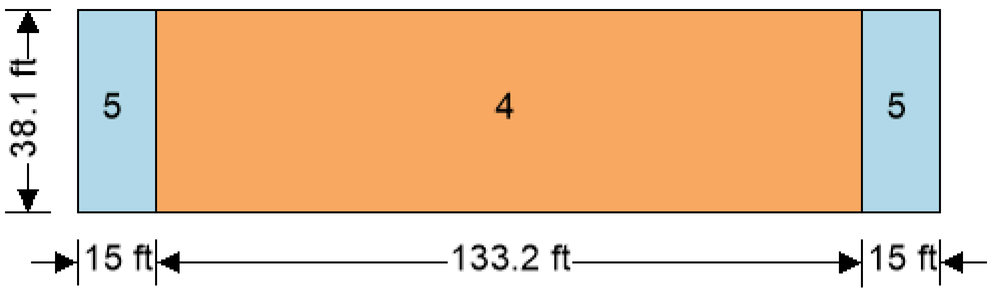
Parapet net pressure $p = q_p \times [GC_p - GC_{pi\_p}]$

Components and cladding pressures - Wall (Table 30.3-1 and (Figure 30.3-2A))

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	4	-	-	10.0	0.90	-0.99	27.7	-30.1
20sf	4	-	-	20.0	0.85	-0.94	26.5	-28.8
50sf	4	-	-	50.0	0.79	-0.88	24.9	-27.2
>100sf	4	-	-	100.0	0.74	-0.83	23.7	-26.0
10sf	5	-	-	10.0	0.90	-1.26	27.7	-37.0
20sf	5	-	-	20.0	0.85	-1.16	26.5	-34.5
50sf	5	-	-	50.0	0.79	-1.04	24.9	-31.3
>100sf	5	-	-	100.0	0.74	-0.94	23.7	-28.8
20 psf (W)	4p	-	-	20.0	0.85	-2.14	27.1	-61.0
20 psf (L)	4p	-	-	20.0	0.85	-0.94	27.1	-29.5
20 psf (W)	5p	-	-	20.0	0.85	-2.14	27.1	-61.0
20 psf (L)	5p	-	-	20.0	0.85	-1.16	27.1	-35.3



Elevation of gable wall



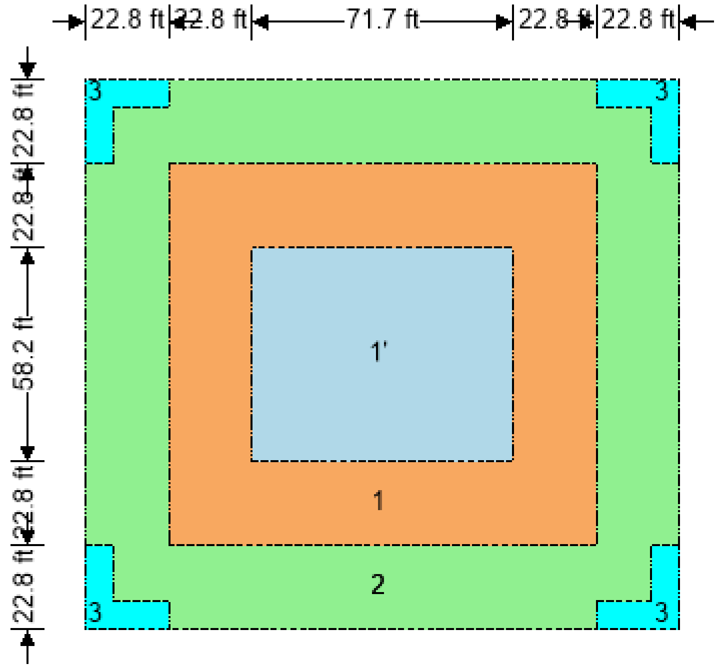
Elevation of side wall

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

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	1	-	-	10.0	0.30	-1.70	12.3 #	-48.3
20sf	1	-	-	20.0	0.27	-1.58	11.6 #	-45.1
50sf	1	-	-	50.0	0.23	-1.41	10.5 #	-40.9
>100sf	1	-	-	100.0	0.20	-1.29	9.8 #	-37.7
<10sf	2	-	-	10.0	0.90	-2.30	27.7	-63.7
20sf	2	-	-	20.0	0.85	-2.14	26.5	-59.6
50sf	2	-	-	50.0	0.79	-1.93	24.9	-54.2
>100sf	2	-	-	100.0	0.74	-1.77	23.7	-50.1
<10sf	3	-	-	10.0	0.90	-2.30	27.7	-63.7
20sf	3	-	-	20.0	0.85	-2.14	26.5	-59.6
50sf	3	-	-	50.0	0.79	-1.93	24.9	-54.2
>100sf	3	-	-	100.0	0.74	-1.77	23.7	-50.1

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



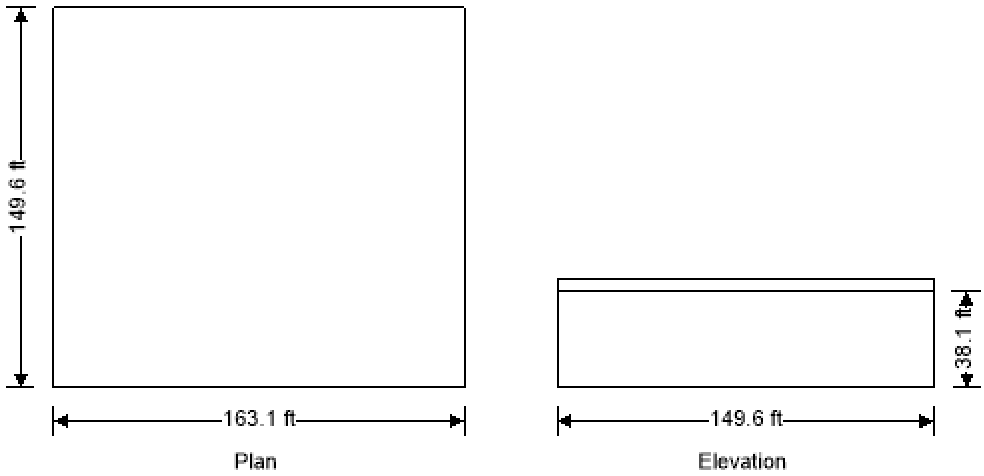
Plan on roof

WIND LOADING

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.13



Building data

Type of roof	Flat
Length of building	b = **163.13** ft
Width of building	d = **149.63** ft
Height to eaves	H = **38.08** ft
Height of parapet	h_p = **4.50** ft
Mean height	h = **38.08** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **14.96** ft
Plan length of Zone 2/2E when GC_{pf} negative	L_{Z2} = min(0.5 × d, 2.5 × H) = **74.81** ft
Plan length of Zone 3/3E encroachment on zone 2	L_{Z3} = max(0 ft,0.5 × d - L_{Z2}) = **0.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **1011** ft
Ground elevation factor	K_e = exp(-0.0000362 × z_{gl}/1ft) = **0.96**
Exposure category (cl 26.7.3)	C
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**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **1.03**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **25.6** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **1.05**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **26.2** psf

Parapet pressures and forces

Velocity pressure at top of parapet q_p = **26.24** psf

Combined net pressure coefficient, leeward GC_{pnl} = **-1.0**

Combined net parapet pressure, leeward $p_{pl} = q_p \times GC_{pnl}$ = **-26.24** psf

Combined net pressure coefficient, windward GC_{pnw} = **1.5**

Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw}$ = **39.36** psf

Wind direction 0 deg (|| to width):

Leeward parapet force $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-19.3** kips

Windward parapet force $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **28.9** kips

Wind direction 90 deg (|| to length):

Leeward parapet force $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-17.7** kips

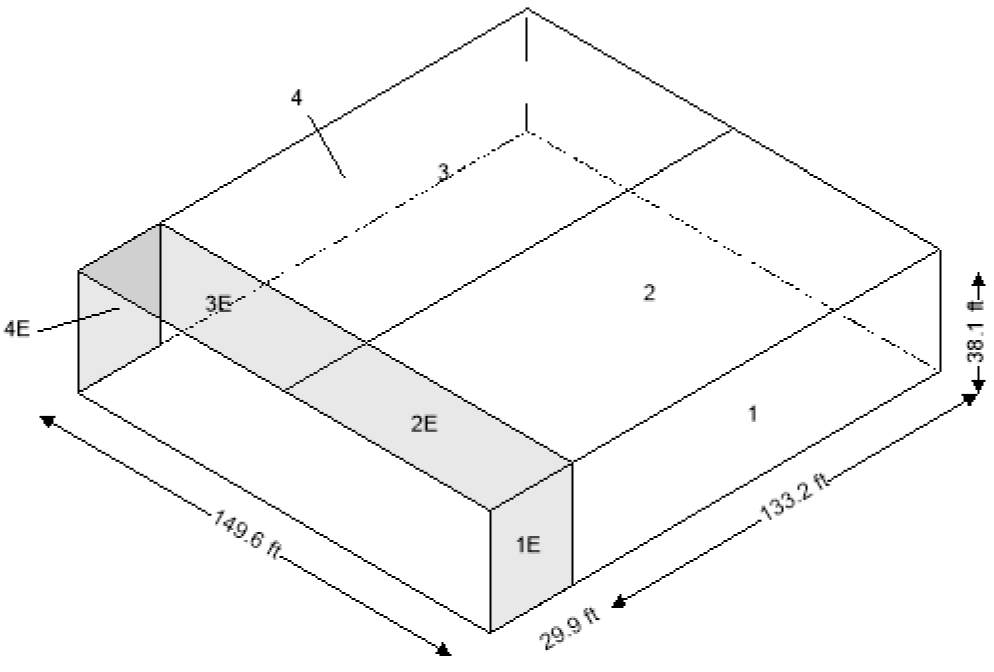
Windward parapet force $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **26.5** kips

Design wind pressures

Design wind pressure equation $p = q_h \times [(GC_{pf}) - (GC_{pi})]$

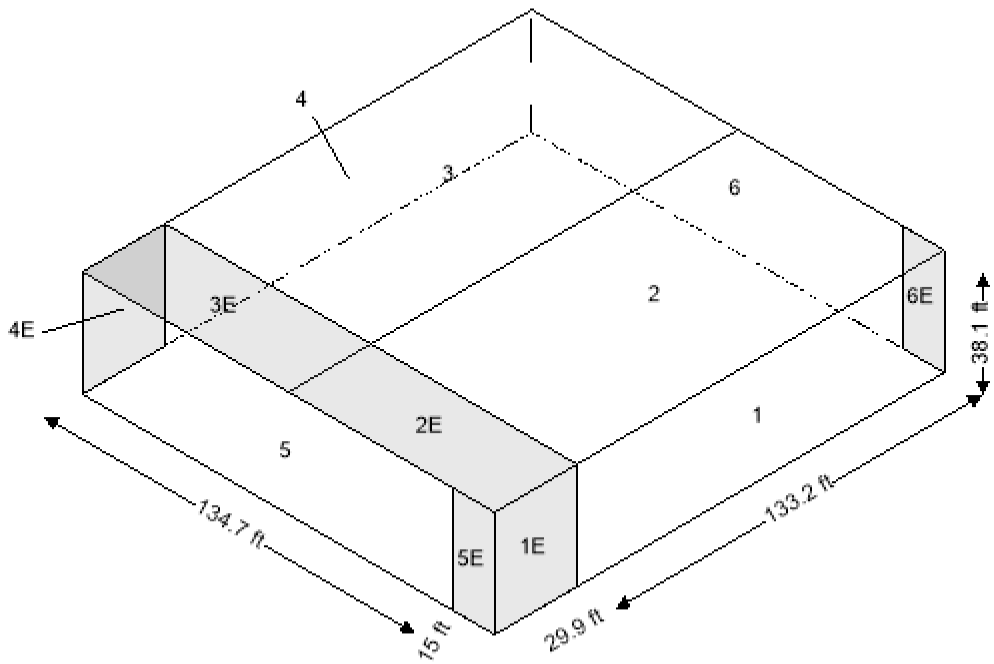
Design wind pressures – Loadcase A

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft²)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.6	14.9	5073	28.6	75.4
2	-0.69	-22.3	-13.1	9965	-222.2	-130.3
3	-0.37	-14.1	-4.9	9965	-140.5	-48.5
4	-0.29	-12.0	-2.8	5073	-61.1	-14.3
1E	0.61	11.0	20.3	1140	12.6	23.1
2E	-1.07	-32.0	-22.8	2239	-71.7	-51.1
3E	-0.53	-18.2	-9.0	2239	-40.7	-20.1
4E	-0.43	-15.6	-6.4	1140	-17.8	-7.3



Design wind pressures – Loadcase B

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-16.1	-6.9	5073	-81.9	-35.1
2	-0.69	-22.3	-13.1	9965	-222.2	-130.3
3	-0.37	-14.1	-4.9	9965	-140.5	-48.5
4	-0.45	-16.1	-6.9	5073	-81.9	-35.1
5	0.40	5.6	14.9	5128	28.9	76.2
6	-0.29	-12.0	-2.8	5128	-61.8	-14.5
1E	-0.48	-16.9	-7.7	1140	-19.3	-8.8
2E	-1.07	-32.0	-22.8	2239	-71.7	-51.1
3E	-0.53	-18.2	-9.0	2239	-40.7	-20.1
4E	-0.48	-16.9	-7.7	1140	-19.3	-8.8
5E	0.61	11.0	20.3	570	6.3	11.5
6E	-0.43	-15.6	-6.4	570	-8.9	-3.7

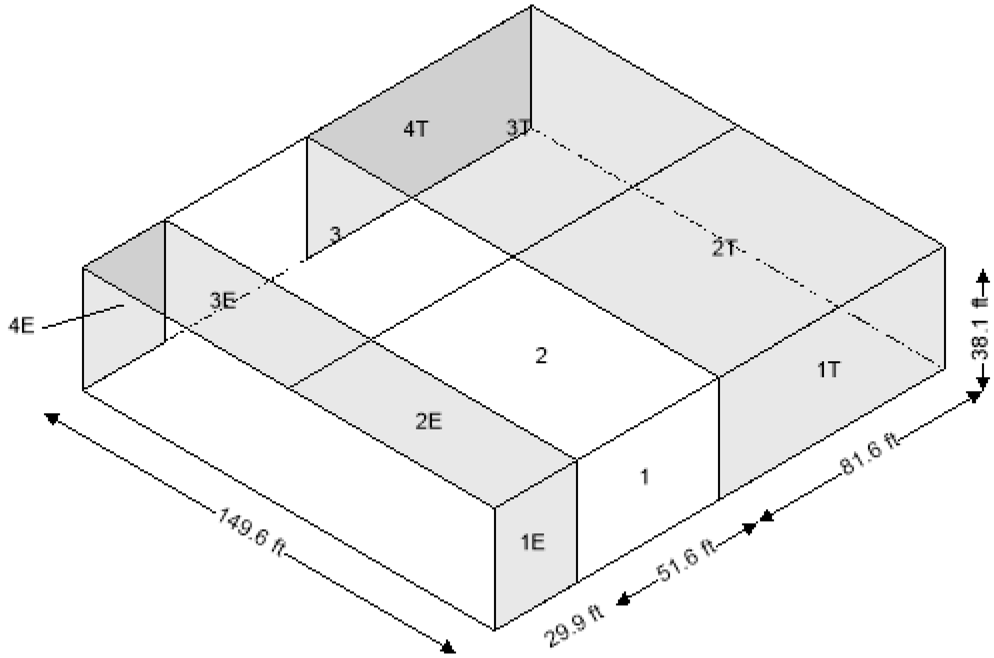


Loadcase B

Design wind pressures – Loadcase AT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.6	14.9	1967	11.1	29.2
2	-0.69	-22.3	-13.1	3863	-86.2	-50.5
3	-0.37	-14.1	-4.9	3863	-54.5	-18.8
4	-0.29	-12.0	-2.8	1967	-23.7	-5.5
1E	0.61	11.0	20.3	1140	12.6	23.1
2E	-1.07	-32.0	-22.8	2239	-71.7	-51.1

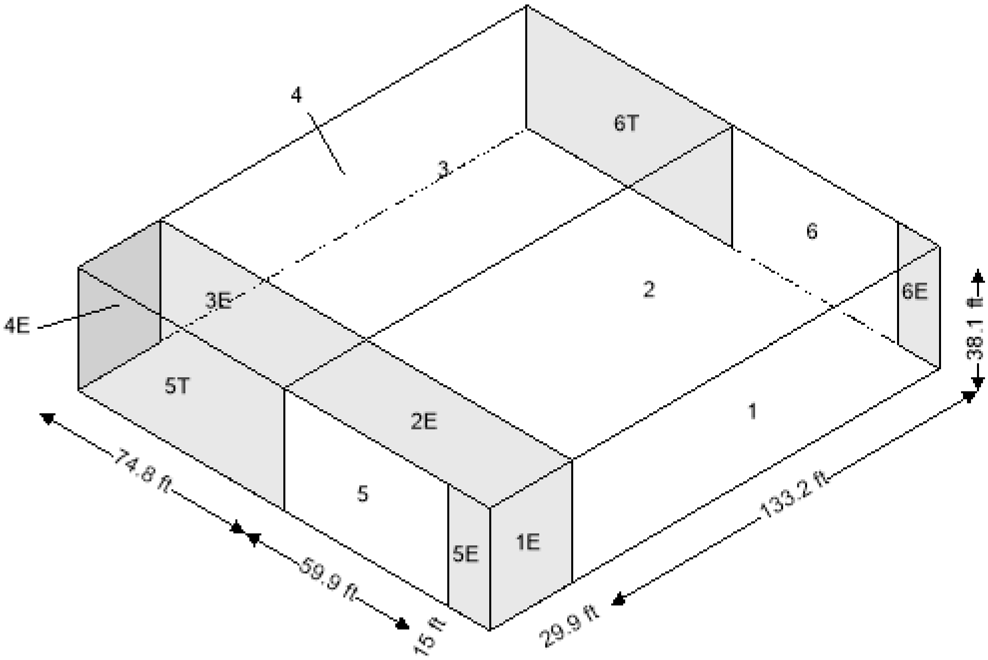
3E	-0.53	-18.2	-9.0	2239	-40.7	-20.1
4E	-0.43	-15.6	-6.4	1140	-17.8	-7.3
1T	-	1.4	3.7	3106	4.4	11.5
2T	-	-5.6	-3.3	6102	-34.0	-19.9
3T	-	-3.5	-1.2	6102	-21.5	-7.4
4T	-	-3.0	-0.7	3106	-9.4	-2.2



Loadcase AT

Design wind pressures – Loadcase BT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-16.1	-6.9	5073	-81.9	-35.1
2	-0.69	-22.3	-13.1	9965	-222.2	-130.3
3	-0.37	-14.1	-4.9	9965	-140.5	-48.5
4	-0.45	-16.1	-6.9	5073	-81.9	-35.1
5	0.40	5.6	14.9	2564	14.5	38.1
6	-0.29	-12.0	-2.8	2564	-30.9	-7.2
1E	-0.48	-16.9	-7.7	1140	-19.3	-8.8
2E	-1.07	-32.0	-22.8	2239	-71.7	-51.1
3E	-0.53	-18.2	-9.0	2239	-40.7	-20.1
4E	-0.48	-16.9	-7.7	1140	-19.3	-8.8
5E	0.61	11.0	20.3	285	3.1	5.8
6E	-0.43	-15.6	-6.4	570	-8.9	-3.7
5T	-	1.4	3.7	2849	4.0	10.6
6T	-	-3.0	-0.7	2849	-8.6	-2.0


4
3
6T
6
2
6E
3E
4E
5T
2E
1
5
5E
1E
74.8 ft
59.9 ft
15 ft
29.9 ft
133.2 ft
38.1 ft

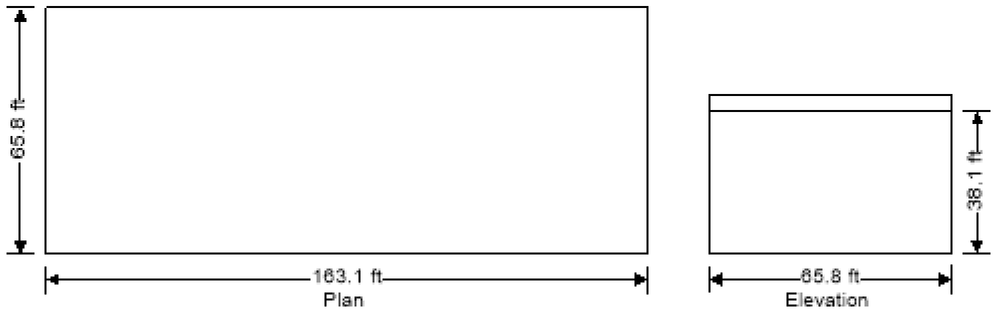
Loadcase BT

WIND LOADING

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.13



Building data	
Type of roof	Flat
Length of building	b = **163.13** ft
Width of building	d = **65.83** ft
Height to eaves	H = **38.08** ft
Height of parapet	h_p = **4.50** ft
Mean height	h = **38.08** ft
End zone width	$a = \max(\min(0.1\times\min(b, d), 0.4\times h), 0.04\times\min(b, d), 3\text{ft})$ = **6.58** ft
Plan length of Zone 2/2E when GC_{pf} negative	$L_{Z2} = \min(0.5 \times d, 2.5 \times H)$ = **32.92** ft
Plan length of Zone 3/3E encroachment on zone 2	$L_{Z3} = \max(0\text{ ft}, 0.5 \times d - L_{Z2})$ = **0.00** ft
General wind load requirements	
Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **1011** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **0.96**
Exposure category (cl 26.7.3)	C
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**
Topography	
Topography factor not significant	K_{zt} = 1.0
Velocity pressure	
Velocity pressure coefficient (Table 26.10-1)	K_z = **1.03**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **25.6** psf
Velocity pressure at parapet	
Velocity pressure coefficient (Table 26.10-1)	K_z = **1.05**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **26.2** psf
Parapet pressures and forces	
Velocity pressure at top of parapet	q_p = **26.24** psf
Combined net pressure coefficient, leeward	GC_{pnl} = **-1.0**
Combined net parapet pressure, leeward	$p_{pl} = q_p \times GC_{pnl}$ = **-26.24** psf
Combined net pressure coefficient, windward	GC_{pnw} = **1.5**

Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw} =$ **39.36** psf

Wind direction 0 deg (|| to width):

Leeward parapet force $F_{w,wpl\_0} = p_{pl} \times h_p \times b =$ **-19.3** kips

Windward parapet force $F_{w,wpw\_0} = p_{pw} \times h_p \times b =$ **28.9** kips

Wind direction 90 deg (|| to length):

Leeward parapet force $F_{w,wpl\_90} = p_{pl} \times h_p \times d =$ **-7.8** kips

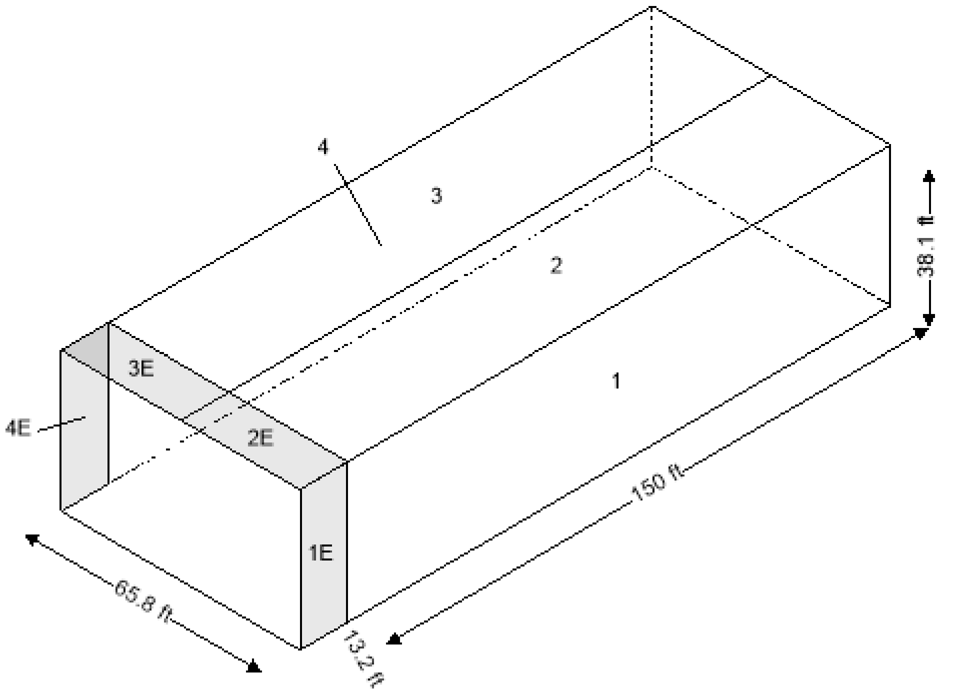
Windward parapet force $F_{w,wpw\_90} = p_{pw} \times h_p \times d =$ **11.7** kips

Design wind pressures

Design wind pressure equation $p = q_h \times [(GC_{pf}) - (GC_{pi})]$

Design wind pressures – Loadcase A

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.6	14.9	5711	32.2	84.9
2	-0.69	-22.3	-13.1	4936	-110.1	-64.5
3	-0.37	-14.1	-4.9	4936	-69.6	-24.0
4	-0.29	-12.0	-2.8	5711	-68.8	-16.1
1E	0.61	11.0	20.3	501	5.5	10.2
2E	-1.07	-32.0	-22.8	433	-13.9	-9.9
3E	-0.53	-18.2	-9.0	433	-7.9	-3.9
4E	-0.43	-15.6	-6.4	501	-7.8	-3.2

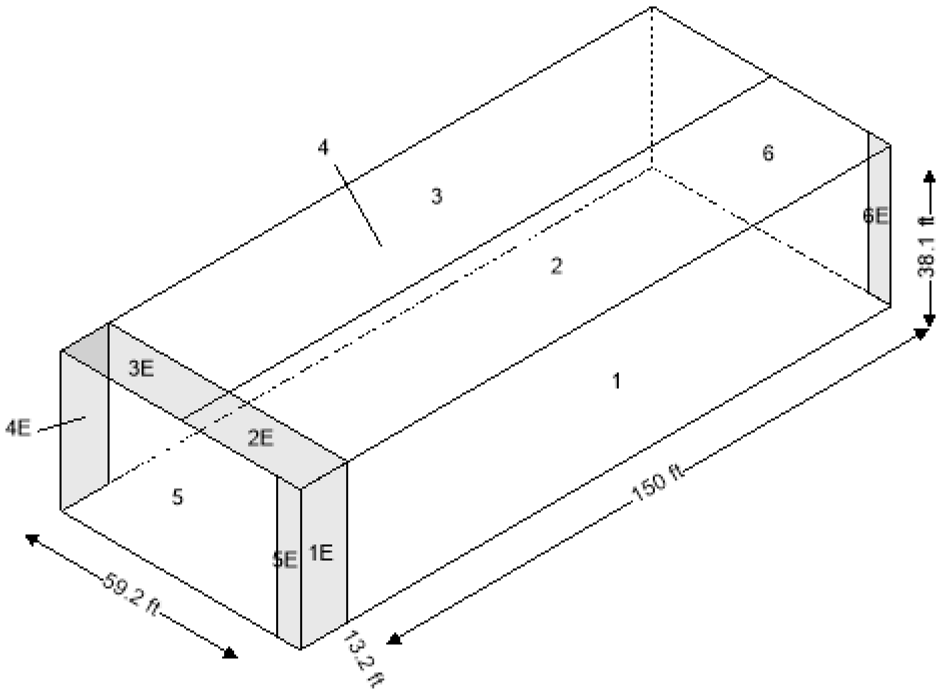


Loadcase A

Design wind pressures – Loadcase B

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-16.1	-6.9	5711	-92.2	-39.5
2	-0.69	-22.3	-13.1	4936	-110.1	-64.5

3	-0.37	-14.1	-4.9	4936	-69.6	-24.0
4	-0.45	-16.1	-6.9	5711	-92.2	-39.5
5	0.40	5.6	14.9	2256	12.7	33.5
6	-0.29	-12.0	-2.8	2256	-27.2	-6.4
1E	-0.48	-16.9	-7.7	501	-8.5	-3.9
2E	-1.07	-32.0	-22.8	433	-13.9	-9.9
3E	-0.53	-18.2	-9.0	433	-7.9	-3.9
4E	-0.48	-16.9	-7.7	501	-8.5	-3.9
5E	0.61	11.0	20.3	251	2.8	5.1
6E	-0.43	-15.6	-6.4	251	-3.9	-1.6

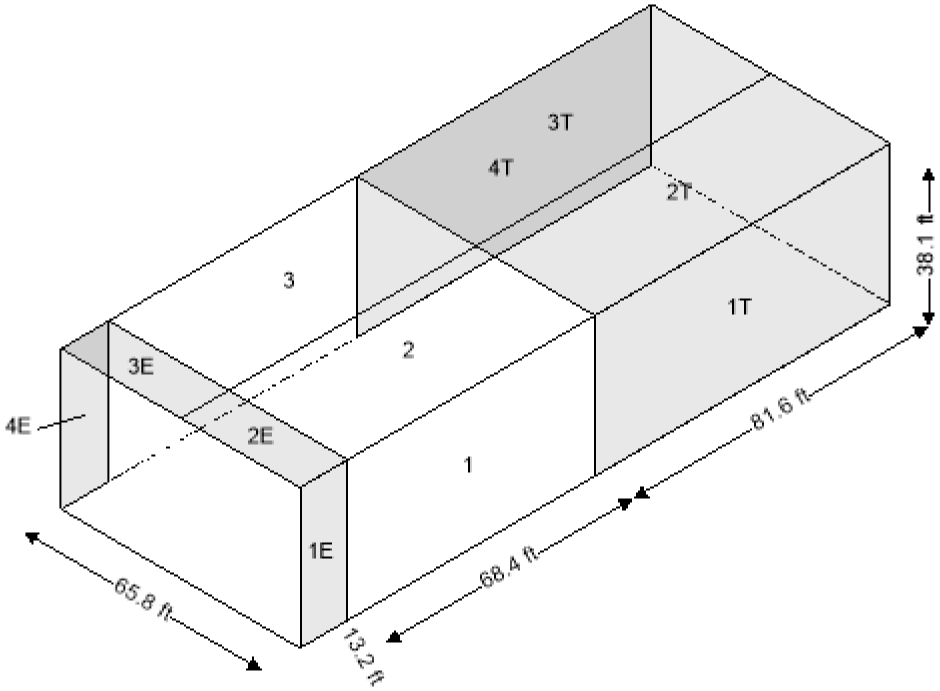


Loadcase B

Design wind pressures – Loadcase AT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.6	14.9	2605	14.7	38.7
2	-0.69	-22.3	-13.1	2251	-50.2	-29.4
3	-0.37	-14.1	-4.9	2251	-31.7	-11.0
4	-0.29	-12.0	-2.8	2605	-31.4	-7.3
1E	0.61	11.0	20.3	501	5.5	10.2
2E	-1.07	-32.0	-22.8	433	-13.9	-9.9
3E	-0.53	-18.2	-9.0	433	-7.9	-3.9
4E	-0.43	-15.6	-6.4	501	-7.8	-3.2
1T	-	1.4	3.7	3106	4.4	11.5
2T	-	-5.6	-3.3	2685	-15.0	-8.8

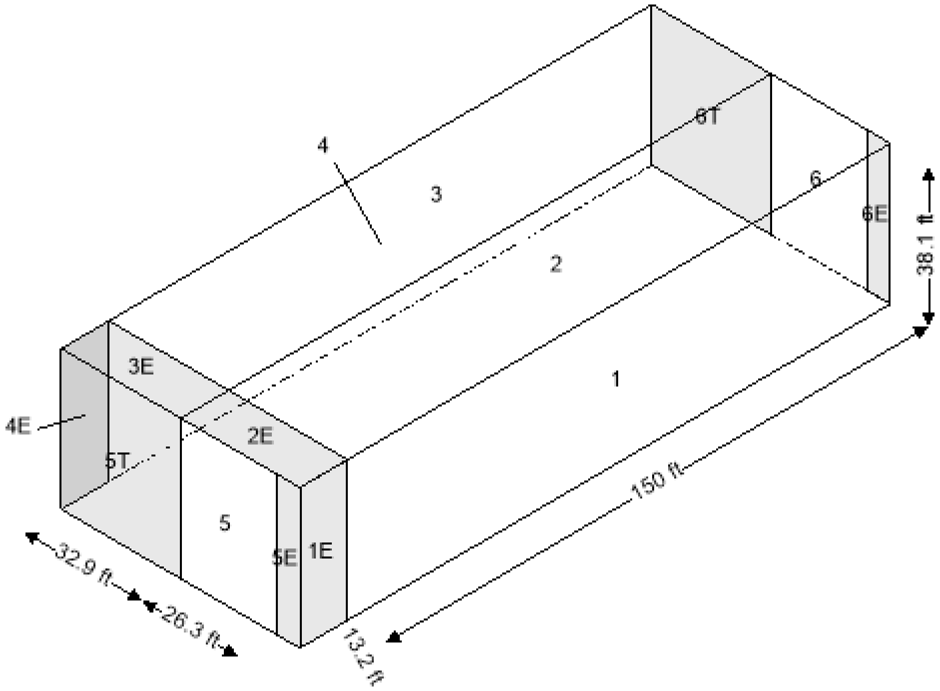
3T	-	-3.5	-1.2	2685	-9.5	-3.3
4T	-	-3.0	-0.7	3106	-9.4	-2.2



Loadcase AT

Design wind pressures – Loadcase BT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-16.1	-6.9	5711	-92.2	-39.5
2	-0.69	-22.3	-13.1	4936	-110.1	-64.5
3	-0.37	-14.1	-4.9	4936	-69.6	-24.0
4	-0.45	-16.1	-6.9	5711	-92.2	-39.5
5	0.40	5.6	14.9	1128	6.4	16.8
6	-0.29	-12.0	-2.8	1128	-13.6	-3.2
1E	-0.48	-16.9	-7.7	501	-8.5	-3.9
2E	-1.07	-32.0	-22.8	433	-13.9	-9.9
3E	-0.53	-18.2	-9.0	433	-7.9	-3.9
4E	-0.48	-16.9	-7.7	501	-8.5	-3.9
5E	0.61	11.0	20.3	125	1.4	2.5
6E	-0.43	-15.6	-6.4	251	-3.9	-1.6
5T	-	1.4	3.7	1254	1.8	4.7
6T	-	-3.0	-0.7	1254	-3.8	-0.9



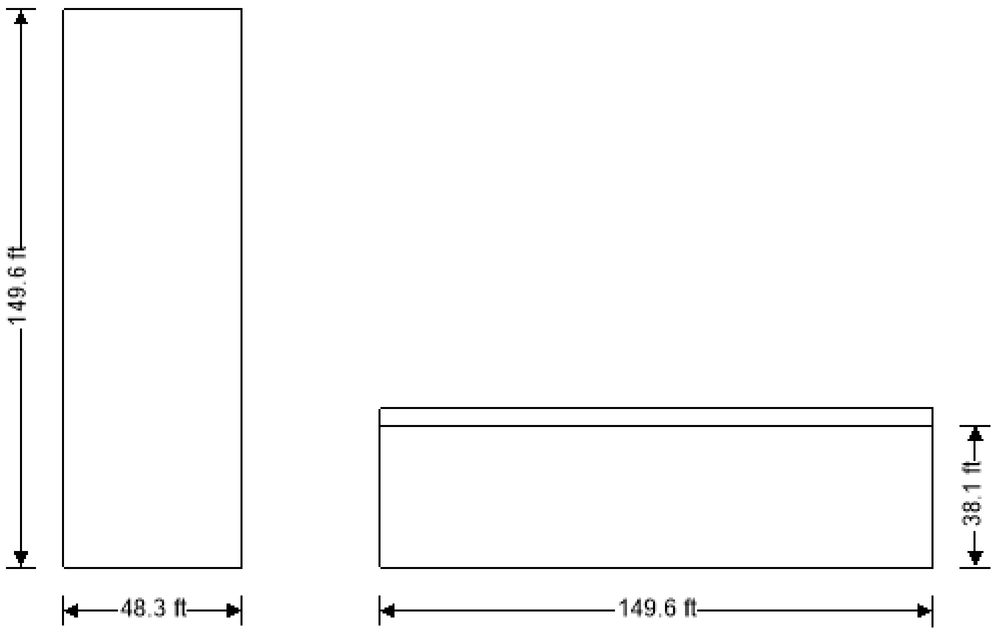
Loadcase BT

WIND LOADING

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.13



Plan

Elevation

Building data

Type of roof	Flat
Length of building	b = **48.33** ft
Width of building	d = **149.63** ft
Height to eaves	H = **38.08** ft
Height of parapet	h_p = **4.50** ft
Mean height	h = **38.08** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **4.83** ft
Plan length of Zone 2/2E when GC_{pf} negative	L_{Z2} = min(0.5 × d, 2.5 × H) = **74.81** ft
Plan length of Zone 3/3E encroachment on zone 2	L_{Z3} = max(0 ft,0.5 × d - L_{Z2}) = **0.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **1011** ft
Ground elevation factor	K_e = exp(-0.0000362 × z_{gl}/1ft) = **0.96**
Exposure category (cl 26.7.3)	C
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**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **1.03**

Velocity pressure | $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **25.6** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1) | K_z = **1.05**

Velocity pressure | $q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **26.2** psf

Parapet pressures and forces

Velocity pressure at top of parapet | q_p = **26.24** psf

Combined net pressure coefficient, leeward | GC_{pnl} = **-1.0**

Combined net parapet pressure, leeward | $p_{pl} = q_p \times GC_{pnl}$ = **-26.24** psf

Combined net pressure coefficient, windward | GC_{pnw} = **1.5**

Combined net parapet pressure, windward | $p_{pw} = q_p \times GC_{pnw}$ = **39.36** psf

Wind direction 0 deg (|| to width):

Leeward parapet force | $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-5.7** kips

Windward parapet force | $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **8.6** kips

Wind direction 90 deg (|| to length):

Leeward parapet force | $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-17.7** kips

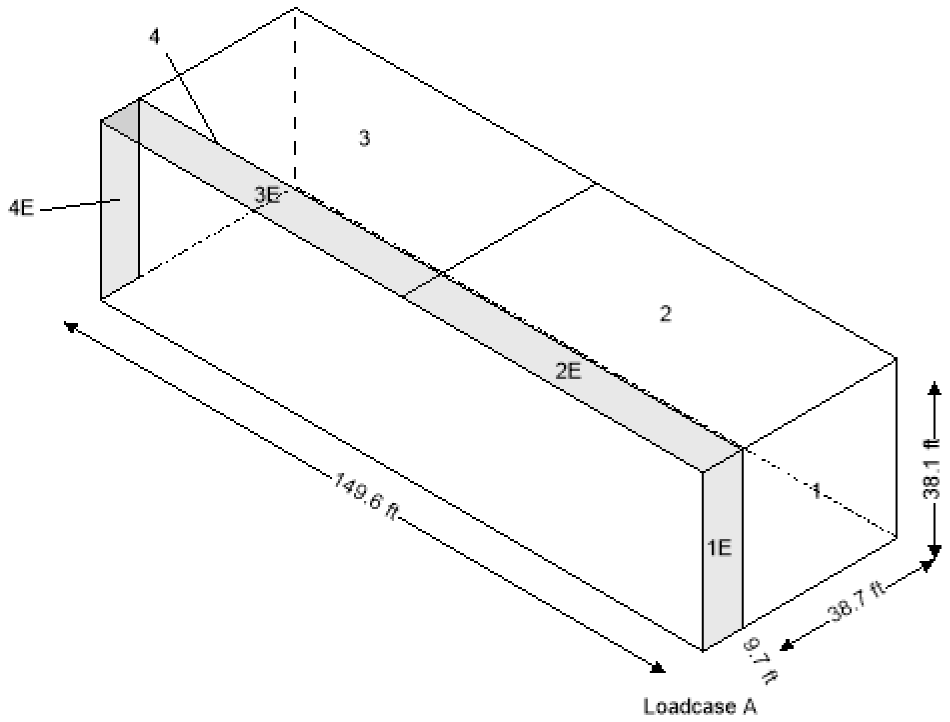
Windward parapet force | $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **26.5** kips

Design wind pressures

Design wind pressure equation | $p = q_h \times [(GC_{pf}) - (GC_{pi})]$

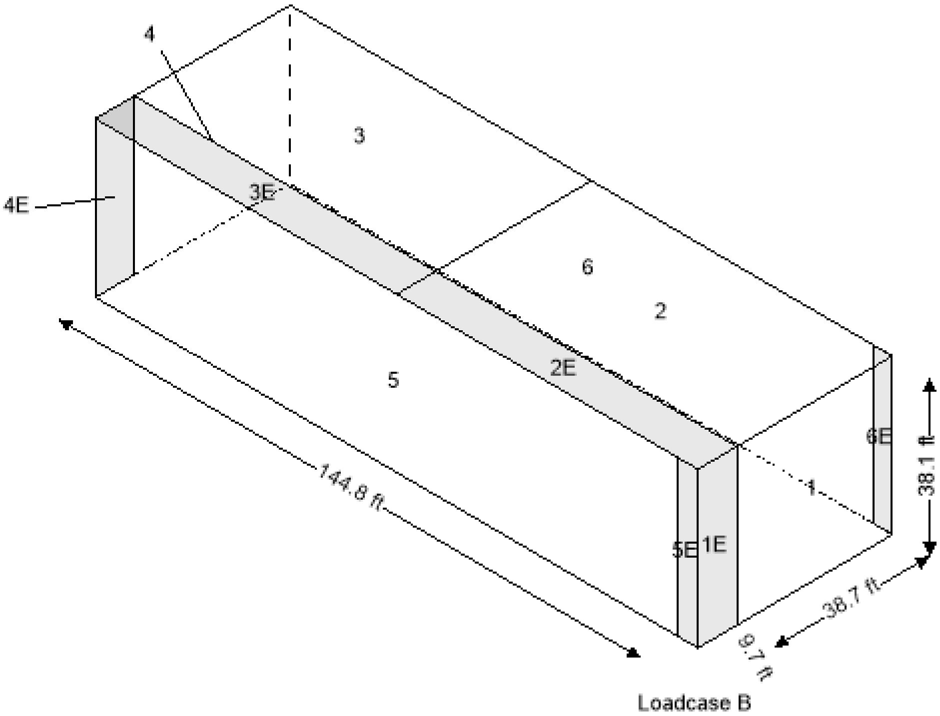
Design wind pressures – Loadcase A

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft²)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.6	14.9	1473	8.3	21.9
2	-0.69	-22.3	-13.1	2893	-64.5	-37.8
3	-0.37	-14.1	-4.9	2893	-40.8	-14.1
4	-0.29	-12.0	-2.8	1473	-17.7	-4.2
1E	0.61	11.0	20.3	368	4.1	7.5
2E	-1.07	-32.0	-22.8	723	-23.2	-16.5
3E	-0.53	-18.2	-9.0	723	-13.2	-6.5
4E	-0.43	-15.6	-6.4	368	-5.8	-2.4



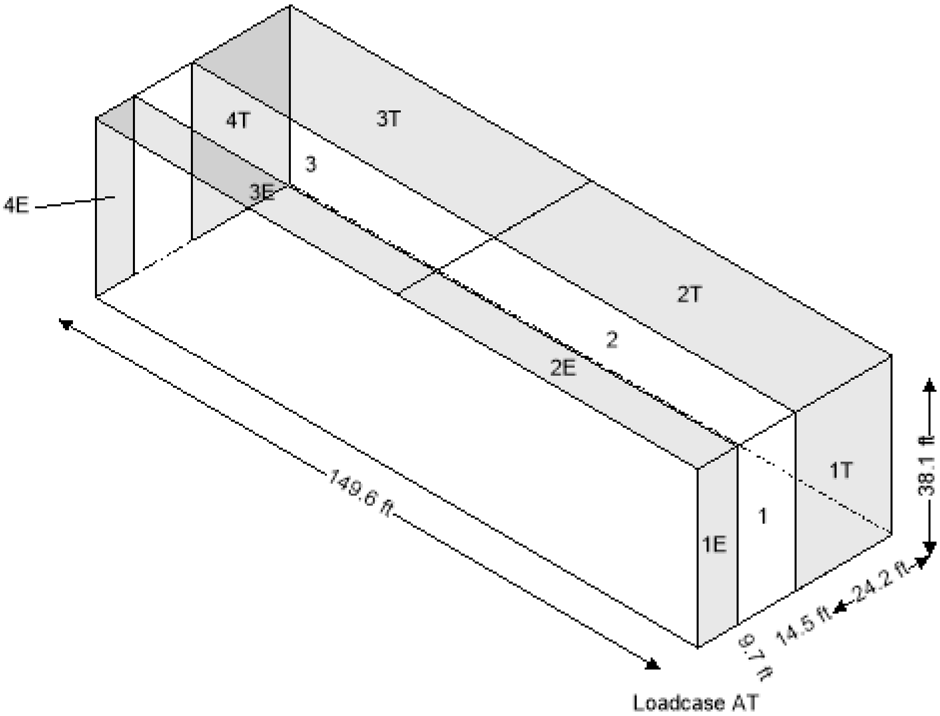
Design wind pressures – Loadcase B

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-16.1	-6.9	1473	-23.8	-10.2
2	-0.69	-22.3	-13.1	2893	-64.5	-37.8
3	-0.37	-14.1	-4.9	2893	-40.8	-14.1
4	-0.45	-16.1	-6.9	1473	-23.8	-10.2
5	0.40	5.6	14.9	5514	31.1	82.0
6	-0.29	-12.0	-2.8	5514	-66.4	-15.5
1E	-0.48	-16.9	-7.7	368	-6.2	-2.8
2E	-1.07	-32.0	-22.8	723	-23.2	-16.5
3E	-0.53	-18.2	-9.0	723	-13.2	-6.5
4E	-0.48	-16.9	-7.7	368	-6.2	-2.8
5E	0.61	11.0	20.3	184	2.0	3.7
6E	-0.43	-15.6	-6.4	184	-2.9	-1.2



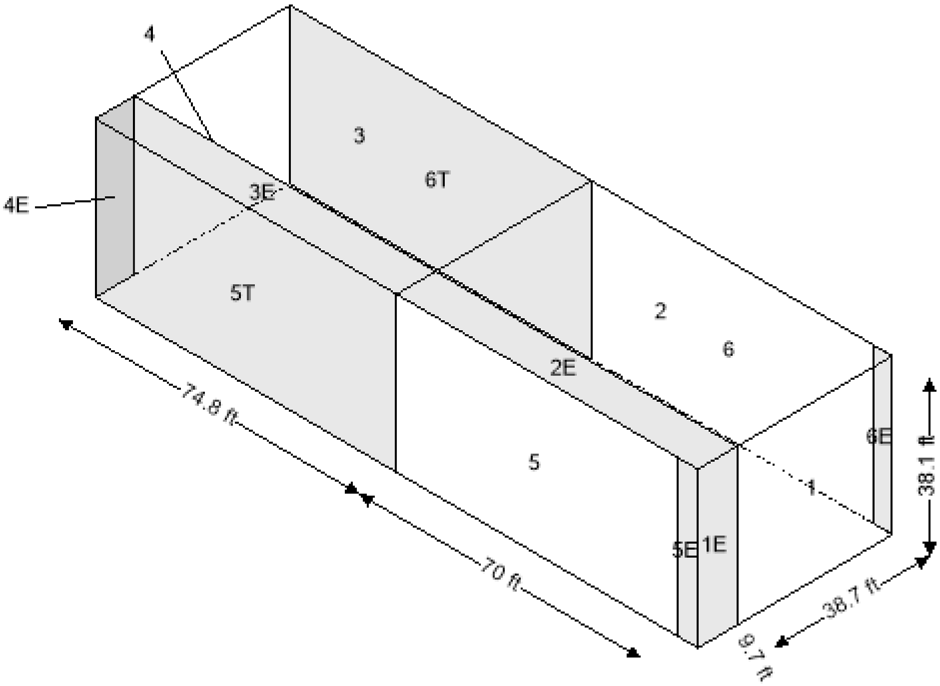
Design wind pressures – Loadcase AT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.6	14.9	552	3.1	8.2
2	-0.69	-22.3	-13.1	1085	-24.2	-14.2
3	-0.37	-14.1	-4.9	1085	-15.3	-5.3
4	-0.29	-12.0	-2.8	552	-6.7	-1.6
1E	0.61	11.0	20.3	368	4.1	7.5
2E	-1.07	-32.0	-22.8	723	-23.2	-16.5
3E	-0.53	-18.2	-9.0	723	-13.2	-6.5
4E	-0.43	-15.6	-6.4	368	-5.8	-2.4
1T	-	1.4	3.7	920	1.3	3.4
2T	-	-5.6	-3.3	1808	-10.1	-5.9
3T	-	-3.5	-1.2	1808	-6.4	-2.2
4T	-	-3.0	-0.7	920	-2.8	-0.6



Design wind pressures – Loadcase BT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-16.1	-6.9	1473	-23.8	-10.2
2	-0.69	-22.3	-13.1	2893	-64.5	-37.8
3	-0.37	-14.1	-4.9	2893	-40.8	-14.1
4	-0.45	-16.1	-6.9	1473	-23.8	-10.2
5	0.40	5.6	14.9	2757	15.5	41.0
6	-0.29	-12.0	-2.8	2757	-33.2	-7.8
1E	-0.48	-16.9	-7.7	368	-6.2	-2.8
2E	-1.07	-32.0	-22.8	723	-23.2	-16.5
3E	-0.53	-18.2	-9.0	723	-13.2	-6.5
4E	-0.48	-16.9	-7.7	368	-6.2	-2.8
5E	0.61	11.0	20.3	92	1.0	1.9
6E	-0.43	-15.6	-6.4	184	-2.9	-1.2
5T	-	1.4	3.7	2849	4.0	10.6
6T	-	-3.0	-0.7	2849	-8.6	-2.0



Loadcase BT

Local Building Code if applicable (CBC, FBC, etc)

(IBC 2018)	(IBC 2015)	(IBC 2012)	
ASCE 7-16	ASCE 7-10	ASCE 7-10	S_{DS} = **0.106**
ACI 318-14	ACI 318-14	ACI 318-11	
AISC 360-16	AISC 360-10	AISC 360-10	
ANSI/AWC NDS-18	ANSI/AWC NDS-15	AWC/AF&PA NDS-12	
TMS 402/602-16	ACI 530-13	ACI 530-13	

Dead Loads

Roof

Top Chord	15 psf
Bottom Chord	10 psf
Total	**25 psf**

Floor

3/4" Gypcrete	7 psf
3/4" plywood	2.5 psf
Floor Trusses	3 psf
MEP	4 psf
(2) 5/8" Gyp. ceiling	5.5 psf
Flooring	1 psf
Insulation	1 psf
Misc.	1 psf
Total	**25 psf**

Ext. Stud Walls (2x6)

gypsum, insulated, 3/8-in. siding, Stucco, and Stone	25 psf
Total	**25 psf**

Int. Stud Walls (2x4)

Wood or steel studs, 5/8" gypsum board each side	11 psf
Total	**11 psf**

Live Loads

Roof

Top Chord	20 psf	
Bottom Chord	10 psf	(Non-concurrent)
Total	**20 psf**	

Floor

Private Rooms/Corridors	40 psf
Public Rooms/Corridors	100 psf

Ext. Stud Walls W/ Veneer (2x6)

Stucco	12 psf
	23 psf

1 1/2" stone veneer (2x6)

Veneer	15 psf
	26 psf

Ext. Stud Walls W/ Brick (2x6)

Brick	30 psf
	41 psf

Masonry Elevator

8" CMU	60 psf
	60 psf

Material Thickness			Snow Loading	
3/4" plywood	0.75 "			**20 psf**
Floor Trusses	16.00 "			
Top PL	3.00 "		***Interior Wind Pressure***	
Sill PL	1.50 "			**5 psf**
Total	21.25 "			
Roof Truss Depth	24.00 "			

Actual Stud Height				
Elevations		*Wall*	*FLR to FLR Height (ft)*	Stud Height
136.56 '	138.56 '	Roof Int. LB 2x4 (Corridor)	2.00 '	1.75 '
128.47 '	136.56 '	4th FLR Int. LB 2x4 (Ext. & Corridor)	8.09 '	7.72 '
118.98 '	128.47 '	3rd FLR Int. LB 2x4 (Ext. & Corridor)	9.49 '	7.72 '
109.49 '	118.98 '	2nd FLR Int. LB 2x4 (Ext. & Corridor)	9.49 '	7.72 '
100.00 '	109.49 '	1st FLR Int. LB 2x4 (Ext. & Corridor)	9.49 '	7.72 '
Total Wall Height			38.56 '	

FRAMING

BSE STRUCTURAL ENGINEERS

Project WSS LEE'S SUMMIT Project No. 23-283

Calc. By GL Checked By DN Date 7/28/2023

Stud Wall Loads (Load Bearing)

Short Exterior Grid		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow	Wind Pres.	Stud	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor	(psf)	(psf)	Spac. (in)	DL	DL wall	LL	SL/LLr	DL	LL	SL/LLr	DL	LL	SL/LLr	Wind	Seismic
Elevations		Roof Ext. LB	2.00	Public	0.00	25	25	-	20	39.36	16	254.43	50.00	-	203.54	50.00	-	203.54	66.67	0.00	271.39	104.96	7.07
136.56	138.56	2x6		Private	10.18	-	25	-	20														
				Drift	0.00	-	-	-	0														
128.47	136.56	4th FLR Ext. LB	8.09	Public	0.00	25	25	100	0	28.8	16	254.43	202.34	407.08	-	506.77	0.00	-	675.69	0.00	271.39	314.44	28.93
		2x6		Private	10.18	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	254.43	237.24	407.08	-	998.43	407.08	-	1331.24	542.77	271.39	432.25	39.77
		2x6		Private	10.18	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	254.43	237.24	407.08	-	1490.10	814.16	-	1986.80	1085.55	271.39	432.25	39.77
		2x6		Private	10.18	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	-	237.24	0.00	-	1981.76	1221.24	-	2642.35	1628.32	271.39	432.25	39.77
		2x6		Private	10.18	-	25	40	0														
				Drift	0.00	-	-	-	0														

Short Interior Grid W-QQ, II-NN		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow	Wind Pres.	Stud	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor	(psf)	(psf)	Spac. (in)	DL (plf)	DL wall (plf)	LL (plf)	SL (plf)	DL (plf)	LL (plf)	SL (plf)	DL (lb)	LL (lb)	SL (lb)	Wind	Seismic
Elevations		Roof Int. LB	0.00	Public	2.67	11	25	-	20	5	16	368.25	0.00	-	294.60	0.00	-	294.60	0.00	0.00	392.80	0.00	0.00
		2x4		Private	12.06	-	25	-	20														
		(Corridor)																					
128.47	136.56	4th FLR Int. LB	8.09	Public	2.67	11	25	100	0	5	16	368.25	89.03	749.40	-	457.28	0.00	-	609.71	0.00	392.80	54.59	12.73
		2x4		Private	12.06	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Int. LB	9.49	Public	2.67	11	25	100	0	5	16	368.25	104.39	749.40	-	929.92	749.40	-	1239.89	999.20	392.80	75.04	17.50
		2x4		Private	12.06	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Int. LB	9.49	Public	2.67	11	25	100	0	5	16	368.25	104.39	749.40	-	1402.55	1498.80	-	1870.07	1998.40	392.80	75.04	17.50
		2x4		Private	12.06	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Int. LB	9.49	Public	2.67	11	25	100	0	5	16	-	104.39	-	-	1875.19	2248.20	-	2500.25	2997.60	392.80	75.04	17.50
		2x4		Private	12.06	-	25	40	0														
				Drift	0.00	-	-	-	0														

Medium Exterior Grid TT-RR, 6-8, 10-11, 13-15, & 17-19		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor				DL (plf)	DL_{wall} (plf)	LL (plf)	SL (plf)	DL (plf)	LL (plf)	SL (plf)	DL (lb)	LL (lb)	SL (lb)	Wind	Seismic
Elevations		Roof Ext. LB	2.00	Public	0.00	25	25	-	20	39.36	16	304.43	50.00	-	243.54	50.00	-	243.54	66.67	0.00	324.72	104.96	7.07
136.56	138.56	2x6		Private	12.18	-	25	-	20														
				Drift	0.00	-	-	-	0														
0	0																						
128.47	136.56	4th FLR Ext. LB	8.09	Public	0.00	25	25	100	0	28.8	16	304.43	202.34	487.08	-	556.77	0.00	-	742.36	0.00	324.72	314.44	28.93
		2x6		Private	12.18	-	25	40	0														
				Drift	0.00	-	-	-	0														
0	0																						
118.98	128.47	3rd FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	304.43	237.24	487.08	-	1098.43	487.08	-	1464.58	649.44	324.72	432.25	39.77
		2x6		Private	12.18	-	25	40	0														
				Drift	0.00	-	-	-	0														
0	0																						
109.49	118.98	2nd FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	304.43	237.24	487.08	-	1640.10	974.16	-	2186.80	1298.88	324.72	432.25	39.77
		2x6		Private	12.18	-	25	40	0														
				Drift	0.00	-	-	-	0														
0	0																						
100	109.489583	1st FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	-	237.24	-	-	2181.76	1461.24	-	2909.02	1948.32	324.72	432.25	39.77
		2x6		Private	12.18	-	25	40	0														
		(Corridor)		Drift	0.00	-	-	-	0														
Medium Interior Grid 2-4, 6-8, 10-11, 13-15, & 17-19		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor				DL (plf)	DL_{wall} (plf)	LL (plf)	SL (plf)	DL (plf)	LL (plf)	SL (plf)	DL (lb)	LL (lb)	SL (lb)	Wind	Seismic
Elevations		Roof Int. LB	0.00	Public	2.81	11	25	-	20	5	16	374.74	0.00	-	299.79	0.00	-	299.79	0.00	0.00	399.72	0.00	0.00
		2x4		Private	12.18	-	25	-	20														
128.47	136.56	4th FLR Int. LB	8.09	Public	2.67	11	25	100	0	5	16	372.25	89.03	755.80	-	463.77	0.00	-	618.36	0.00	399.72	54.59	12.73
		2x4		Private	12.22	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Int. LB	9.49	Public	2.67	11	25	100	0	5	16	372.25	104.39	755.80	-	940.40	755.80	-	1253.87	1007.73	399.72	75.04	17.50
		2x4		Private	12.22	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Int. LB	9.49	Public	2.67	11	25	100	0	5	16	372.25	104.39	755.80	-	1417.04	1511.60	-	1889.39	2015.47	399.72	75.04	17.50
		2x4		Private	12.22	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Int. LB	9.49	Public	2.67	11	25	100	0	5	16	-	104.39	-	-	1893.68	2267.40	-	2524.90	3023.20	399.72	75.04	17.50
		2x4		Private	12.22	-	25	40	0														
				Drift	0.00	-	-	-	0														

Long Exterior Grid 4-6, 8-10, 11-13, & 15-17		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow	Wind Pres.	Stud	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor	(psf)	(psf)	Spac. (in)	DL (plf)	DL_{wall} (plf)	LL (plf)	SL (plf)	DL (plf)	LL (plf)	SL (plf)	DL (lb)	LL (lb)	SL (lb)	Wind	Seismic
Elevations		Roof Ext. LB	2.00	Public	0.00	25	25	-	20	39.36	16	323.00	50.00	-	258.40	50.00	-	258.40	66.67	0.00	344.53	104.96	7.07
136.56	138.56	2x6		Private	12.92	-	25	-	20														
				Drift	0.00	-	-	-	0														
128.47	136.56	4^{th} FLR Ext. LB	8.09	Public	0.00	25	25	100	0	28.8	16	323.00	202.34	516.80	-	575.34	0.00	-	767.13	0.00	344.53	314.44	28.93
		2x6		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3^{rd} FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	323.00	237.24	516.80	-	1135.58	516.80	-	1514.11	689.07	344.53	432.25	39.77
		2x6		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2^{nd} FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	323.00	237.24	516.80	-	1695.82	1033.60	-	2261.10	1378.13	344.53	432.25	39.77
		2x6		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1^{st} FLR Ext. LB	9.49	Public	0.00	25	25	100	0	28.8	16	-	237.24	-	-	2256.06	1550.40	-	3008.08	2067.20	344.53	432.25	39.77
		2x6		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														

Long Interior Grid 4-6, 8-10, 11-13, & 15-17		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow	Wind Pres.	Stud	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor	(psf)	(psf)	Spac. (in)	DL (plf)	DL_{wall} (plf)	LL (plf)	SL (plf)	DL (plf)	LL (plf)	SL (plf)	DL (lb)	LL (lb)	SL (lb)	Wind	Seismic
Elevations		Roof Int. LB	0.00	Public	2.81	11	25	-	20	5	16	393.31	0.00	-	314.65	0.00	-	314.65	0.00	0.00	419.53	0.00	0.00
		2x4		Private	12.92	-	25	-	20														
128.47	136.56	4^{th} FLR Int. LB	8.09	Public	2.81	11	25	100	0	5	16	393.31	89.03	798.05	-	482.34	0.00	-	643.13	0.00	419.53	54.59	12.73
		2x4		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3^{rd} FLR Int. LB	9.49	Public	2.81	11	25	100	0	5	16	393.31	104.39	798.05	-	980.04	798.05	-	1306.72	1064.07	419.53	75.04	17.50
		2x4		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2^{nd} FLR Int. LB	9.49	Public	2.81	11	25	100	0	5	16	393.31	104.39	798.05	-	1477.74	1596.10	-	1970.32	2128.13	419.53	75.04	17.50
		2x4		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1^{st} FLR Int. LB	9.49	Public	2.81	11	25	100	0	5	16	-	104.39	-	-	1975.44	2394.15	-	2633.92	3192.20	419.53	75.04	17.50
		2x4		Private	12.92	-	25	40	0														
				Drift	0.00	-	-	-	0														

Applicable Load Combinations (2018 IBC)

1.) D
2.) D + L
3.) D + (Lr or S)
4.) D + 0.75L + 0.75(Lr or S)
5.) D + 0.6W
6.) D + 0.75L + 0.45W + 0.75(Lr or S)
7.) 0.6D + 0.6W
8.) D + 0.7E
9.) D + 0.525E + 0.75L + 0.75S
10.) 0.6D + 0.7E

Wall Stud Loads @ Floor Level

	Wall	Combo 2		Combo 3		Combo 5		Combo 6		Combo 8		Combo 9*		Max	
		Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.
Short Exterior Grid	Roof Ext. LB	67	0	338	0	67	63	270	47	67	5	270	4	338	63
	4^{th} FLR Ext. LB	676	0	947	0	676	189	879	141	676	20	879	15	947	189
	3^{rd} FLR Ext. LB	1874	0	1603	0	1331	259	1942	195	1331	28	1942	21	1942	259
	2^{nd} FLR Ext. LB	3072	0	2258	0	1987	259	3004	195	1987	28	3004	21	3072	259
	1^{st} FLR Ext. LB	4271	0	2914	0	2642	259	4067	195	2642	28	4067	21	4271	259
Short Interior Grid W-QQ, II-NN	Roof Int. LB	0	0	393	0	0	0	295	0	0	0	295	0	393	0
	4^{th} FLR Int. LB	610	0	1003	0	610	33	904	25	610	9	904	7	1003	33
	3^{rd} FLR Int. LB	2239	0	1633	0	1240	45	2284	34	1240	12	2284	9	2284	45
	2^{nd} FLR Int. LB	3868	0	2263	0	1870	45	3663	34	1870	12	3663	9	3868	45
	1^{st} FLR Int. LB	5498	0	2893	0	2500	45	5043	34	2500	12	5043	9	5498	45
Medium Exterior Grid TT-RR, 6-8, 10-11, 13-15, & 17-19	Roof Ext. LB	67	0	391	0	67	63	310	47	67	5	310	4	391	63
	4^{th} FLR Ext. LB	742	0	1067	0	742	189	986	141	742	20	986	15	1067	189
	3^{rd} FLR Ext. LB	2114	0	1789	0	1465	259	2195	195	1465	28	2195	21	2195	259
	2^{nd} FLR Ext. LB	3486	0	2512	0	2187	259	3404	195	2187	28	3404	21	3486	259
	1^{st} FLR Ext. LB	4857	0	3234	0	2909	259	4614	195	2909	28	4614	21	4857	259
Medium Interior Grid 2-4, 6-8, 10-11, 13-15, & 17-19	Roof Int. LB	0	0	400	0	0	0	300	0	0	0	300	0	400	0
	4^{th} FLR Int. LB	618	0	1018	0	618	33	918	25	618	9	918	7	1018	33
	3^{rd} FLR Int. LB	2262	0	1654	0	1254	45	2309	34	1254	12	2309	9	2309	45
	2^{nd} FLR Int. LB	3905	0	2289	0	1889	45	3701	34	1889	12	3701	9	3905	45
	1^{st} FLR Int. LB	5548	0	2925	0	2525	45	5092	34	2525	12	5092	9	5548	45
Long Exterior Grid 4-6, 8-10, 11-13, & 15-17	Roof Ext. LB	67	0	411	0	67	63	325	47	67	5	325	4	411	63
	4^{th} FLR Ext. LB	767	0	1112	0	767	189	1026	141	767	20	1026	15	1112	189
	3^{rd} FLR Ext. LB	2203	0	1859	0	1514	259	2289	195	1514	28	2289	21	2289	259
	2^{nd} FLR Ext. LB	3639	0	2606	0	2261	259	3553	195	2261	28	3553	21	3639	259
	1^{st} FLR Ext. LB	5075	0	3353	0	3008	259	4817	195	3008	28	4817	21	5075	259
Long Interior Grid 4-6, 8-10, 11-13, & 15-17	Roof Int. LB	0	0	420	0	0	0	315	0	0	0	315	0	420	0
	4^{th} FLR Int. LB	643	0	1063	0	643	33	958	25	643	9	958	7	1063	33
	3^{rd} FLR Int. LB	1307	0	1726	0	1307	45	2419	34	1307	12	2419	9	2419	45
	2^{nd} FLR Int. LB	4098	0	2390	0	1970	45	3881	34	1970	12	3881	9	4098	45
	1^{st} FLR Int. LB	5826	0	3053	0	2634	45	5343	34	2634	12	5343	9	5826	45

BSE STRUCTURAL ENGINEERS

Project WSS LEE'S SUMMIT Project No. 23-283
Calc. By GL Checked By DN Date 7/28/2023

Stud Wall Loads (Non-Load Bearing)

Exterior Grid 2 & 19 Elevations		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf) Wall	DL (psf) Floor	LL (psf) Floor	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf) DL	Individual - w (plf) DL_{wall}	Individual - w (plf) LL	Individual - w (plf) SL/LL_r	Cumulative - w (plf) DL	Cumulative - w (plf) LL	Cumulative - w (plf) SL/LL_r	Cumulative - Axial (lb) DL	Cumulative - Axial (lb) LL	Cumulative - Axial (lb) SL/LL_r	Moment (lb-ft) Wind	Moment (lb-ft) Seismic
		Roof Ext. LB	2.00	Public	0.00	12	25	-	20	39.36	16	25.00	24.00	-	20.00	24.00	-	20.00	32.00	0.00	26.67	104.96	3.39
136.56	138.56	2x6		Private	1.00	-	25	-	20														
				Drift	0.00	-	-	-	0														
128.47	136.56	4th FLR Ext. LB	8.09	Public	0.00	12	25	100	0	28.8	16	25.00	97.13	40.00	-	146.13	0.00	-	194.83	0.00	26.67	314.44	13.89
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Ext. LB	9.49	Public	0.00	12	25	100	0	28.8	16	25.00	113.88	40.00	-	285.00	40.00	-	380.00	53.33	26.67	432.25	19.09
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Ext. LB	9.49	Public	0.00	12	25	100	0	28.8	16	25.00	113.88	40.00	-	423.88	80.00	-	565.17	106.67	26.67	432.25	19.09
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Ext. LB	9.49	Public	0.00	12	25	100	0	28.8	16	-	113.88	-	-	562.75	120.00	-	750.33	160.00	26.67	432.25	19.09
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														

Interior Grid 3-18 Elevations		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf) Wall	DL (psf) Floor	LL (psf) Floor	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf) DL	Individual - w (plf) DL_{wall}	Individual - w (plf) LL	Individual - w (plf) SL/LL_r	Cumulative - w (plf) DL	Cumulative - w (plf) LL	Cumulative - w (plf) SL/LL_r	Cumulative - Axial (lb) DL	Cumulative - Axial (lb) LL	Cumulative - Axial (lb) SL/LL_r	Moment (lb-ft) Wind	Moment (lb-ft) Seismic
		Roof Ext. LB	0.00	Public	0.00	11	25	-	20	5	16	50.00	0.00	-	40.00	0.00	-	40.00	0.00	0.00	53.33	0.00	0.00
		2x6		Private	2.00	-	25	-	20														
128.47	136.56	4th FLR Ext. LB	8.09	Public	0.00	11	25	100	0	5	16	50.00	89.03	80.00	-	139.03	0.00	-	185.38	0.00	53.33	54.59	12.73
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Ext. LB	9.49	Public	0.00	11	25	100	0	5	16	50.00	104.39	80.00	-	293.42	80.00	-	391.22	106.67	53.33	75.04	17.50
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Ext. LB	9.49	Public	0.00	11	25	100	0	5	16	50.00	104.39	80.00	-	447.80	160.00	-	597.07	213.33	53.33	75.04	17.50
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Ext. LB	9.49	Public	0.00	11	25	100	0	5	16	-	104.39	-	-	602.19	240.00	-	802.92	320.00	53.33	75.04	17.50
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														

Stair Interior Grid 2 & 19 Elevations		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor				DL	DL_{wall}	LL	SL/LL_r	DL	LL	SL/LL_r	DL	LL	SL/LL_r	Wind	Seismic
		Roof Int. LB	0.00	Public	1.00	11	25	-	20	5	16	50.00	0.00	-	40.00	0.00	-	40.00	0.00	0.00	53.33	0.00	0.00
		2x6		Private	1.00	-	25	-	20														
128.47	136.56	4th FLR Int. LB	8.09	Public	1.00	11	25	100	0	5	16	50.00	89.03	140.00	-	139.03	0.00	-	185.38	0.00	53.33	54.59	12.73
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Int. LB	9.49	Public	1.00	11	25	100	0	5	16	50.00	104.39	140.00	-	293.42	140.00	-	391.22	186.67	53.33	75.04	17.50
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Int. LB	9.49	Public	1.00	11	25	100	0	5	16	50.00	104.39	140.00	-	447.80	280.00	-	597.07	373.33	53.33	75.04	17.50
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Int. LB	9.49	Public	1.00	11	25	100	0	5	16	-	104.39	-	-	602.19	420.00	-	802.92	560.00	53.33	75.04	17.50
		2x6		Private	1.00	-	25	40	0														
				Drift	0.00	-	-	-	0														

Mid Interior Elevations		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor				DL	DL_{wall}	LL	SL/LL_r	DL	LL	SL/LL_r	DL	LL	SL/LL_r	Wind	Seismic
		Roof Int. LB	0.00	Public	0.00	11	25	-	20	5	16	50.00	0.00	-	40.00	0.00	-	40.00	0.00	0.00	53.33	0.00	0.00
		2x6		Private	2.00	-	25	-	20														
128.46875	136.5625	4th FLR Int. LB	8.09	Public	0.00	11	25	100	0	5	16	50.00	89.03	80.00	-	139.03	0.00	-	185.38	0.00	53.33	54.59	12.73
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.97917	128.46875	3rd FLR Int. LB	9.49	Public	0.00	11	25	100	0	5	16	50.00	104.39	80.00	-	293.42	80.00	-	391.22	106.67	53.33	75.04	17.50
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.48958	118.97917	2nd FLR Int. LB	9.49	Public	0.00	11	25	100	0	5	16	50.00	104.39	80.00	-	447.80	160.00	-	597.07	213.33	53.33	75.04	17.50
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
100	109.48958	1st FLR Int. LB	9.49	Public	0.00	11	25	100	0	5	16	-	104.39	-	-	602.19	240.00	-	802.92	320.00	53.33	75.04	17.50
		2x6		Private	2.00	-	25	40	0														
				Drift	0.00	-	-	-	0														

Stair Exterior Grid 1 & 20 Elevations		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor				DL	DL_{wall}	LL	SL/LL_r	DL	LL	SL/LL_r	DL	LL	SL/LL_r	Wind	Seismic
		Roof Int. LB	2.00	Public	1.00	12	25	-	20	39.36	16	25.00	24.00	-	20.00	24.00	-	20.00	32.00	0.00	26.67	104.96	3.39
136.56	138.56	2x6		Private	0.00	-	25	-	20														
				Drift	0.00	-	-	-	0														
128.47	136.56	4th FLR Int. LB	8.09	Public	1.00	12	25	100	0	28.8	16	25.00	97.13	100.00	-	146.13	0.00	-	194.83	0.00	26.67	314.44	13.89
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Int. LB	9.49	Public	1.00	12	25	100	0	28.8	16	25.00	113.88	100.00	-	285.00	100.00	-	380.00	133.33	26.67	432.25	19.09
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Int. LB	9.49	Public	1.00	12	25	100	0	28.8	16	25.00	113.88	100.00	-	423.88	200.00	-	565.17	266.67	26.67	432.25	19.09
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Int. LB	9.49	Public	1.00	12	25	100	0	28.8	16	-	113.88	-	-	562.75	300.00	-	750.33	400.00	26.67	432.25	19.09
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														

Placeholder Elevations		Wall	Wall Height (ft)	Trib. Width (ft)		DL (psf)		LL (psf)	Snow (psf)	Wind Pres. (psf)	Stud Spac. (in)	Individual - w (plf)				Cumulative - w (plf)			Cumulative - Axial (lb)			Moment (lb-ft)	
						Wall	Floor	Floor				DL	DL_{wall}	LL	SL/LL_r	DL	LL	SL/LL_r	DL	LL	SL/LL_r	Wind	Seismic
		Roof Ext. LB	2.00	Public	1.00	12	25	-	20	39.36	16	25.00	24.00	-	20.00	24.00	-	20.00	32.00	0.00	26.67	104.96	3.39
136.56	138.56	2x6		Private	0.00	-	25	-	20														
				Drift	0.00	-	-	-	0														
128.47	136.56	4th FLR Ext. LB	8.09	Public	1.00	12	25	100	0	28.8	16	25.00	97.13	100.00	-	146.13	0.00	-	194.83	0.00	26.67	314.44	13.89
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
118.98	128.47	3rd FLR Ext. LB	9.49	Public	1.00	12	25	100	0	28.8	16	25.00	113.88	100.00	-	285.00	100.00	-	380.00	133.33	26.67	432.25	19.09
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
109.49	118.98	2nd FLR Ext. LB	9.49	Public	1.00	12	25	100	0	28.8	16	25.00	113.88	100.00	-	423.88	200.00	-	565.17	266.67	26.67	432.25	19.09
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														
100.00	109.49	1st FLR Ext. LB	9.49	Public	1.00	12	25	100	0	28.8	16	-	113.88	-	-	562.75	300.00	-	750.33	400.00	26.67	432.25	19.09
		2x6		Private	0.00	-	25	40	0														
				Drift	0.00	-	-	-	0														

Applicable Load Combinations (2018 IBC)

1.) D	6.) D + 0.75L + 0.45W + 0.75(Lr or S)
2.) D + L	7.) 0.6D + 0.6W
3.) D + (Lr or S)	8.) D + 0.7E
4.) D + 0.75L + 0.75(Lr or S)	9.) D + 0.525E + 0.75L + 0.75S
5.) D + 0.6W	10.) 0.6D + 0.7E

Wall Stud Loads @ Floor Level

	Wall	Combo 2		Combo 3		Combo 5		Combo 6		Combo 8		Combo 9*		Max	
		Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.	Axial	Mom.
Exterior Grid 2 & 19	Roof Ext. NLB	32	0	59	0	32	63	52	47	32	2	52	2	59	63
	4^{th} FLR Ext. NLB	195	0	222	0	195	189	215	141	195	10	215	7	222	189
	3^{rd} FLR Ext. NLB	433	0	407	0	380	259	440	195	380	13	440	10	440	259
	2^{nd} FLR Ext. NLB	672	0	592	0	565	259	665	195	565	13	665	10	672	259
	1^{st} FLR Ext. NLB	910	0	777	0	750	259	890	195	750	13	890	10	910	259
Interior Grid 3-18	Roof Ext. NLB	0	0	53	0	0	0	40	0	0	0	40	0	53	0
	4^{th} FLR Ext. NLB	185	0	239	0	185	33	225	25	185	9	225	7	239	33
	3^{rd} FLR Ext. NLB	498	0	445	0	391	45	511	34	391	12	511	9	511	45
	2^{nd} FLR Ext. NLB	810	0	650	0	597	45	797	34	597	12	797	9	810	45
	1^{st} FLR Ext. NLB	1123	0	856	0	803	45	1083	34	803	12	1083	9	1123	45
Stair Interior Grid 2 & 19	Roof Int. LB	0	0	53	0	0	0	40	0	0	0	40	0	53	0
	4^{th} FLR Int. LB	185	0	239	0	185	33	225	25	185	9	225	7	239	33
	3^{rd} FLR Int. LB	578	0	445	0	391	45	571	34	391	12	571	9	578	45
	2^{nd} FLR Int. LB	970	0	650	0	597	45	917	34	597	12	917	9	970	45
	1^{st} FLR Int. LB	1363	0	856	0	803	45	1263	34	803	12	1263	9	1363	45
Med. Int.	Roof Int. NLB	0	0	53	0	0	0	40	0	0	0	40	0	53	0
	4^{th} FLR Int. NLB	185	0	239	0	185	33	225	25	185	9	225	7	239	33
	3^{rd} FLR Int. NLB	498	0	445	0	391	45	511	34	391	12	511	9	511	45
	2^{nd} FLR Int. NLB	810	0	650	0	597	45	797	34	597	12	797	9	810	45
	1^{st} FLR Int. NLB	1123	0	856	0	803	45	1083	34	803	12	1083	9	1123	45
Stair Exterior Grid 1 & 20	Roof Int. NLB	32	0	59	0	32	63	52	47	32	2	52	2	59	63
	4^{th} FLR Int. NLB	195	0	222	0	195	189	215	141	195	10	215	7	222	189
	3^{rd} FLR Int. NLB	513	0	407	0	380	259	500	195	380	13	500	10	513	259
	2^{nd} FLR Int. NLB	832	0	592	0	565	259	785	195	565	13	785	10	832	259
	1^{st} FLR Int. NLB	1150	0	777	0	750	259	1070	195	750	13	1070	10	1150	259
Placeholder	Roof Ext. NLB	32	0	59	0	32	63	52	47	32	2	52	2		
	4^{th} FLR Ext. NLB	195	0	222	0	195	189	215	141	195	10	215	7		
	3^{rd} FLR Ext. NLB	380	0	407	0	380	259	500	195	380	13	500	10		
	2^{nd} FLR Ext. NLB	832	0	592	0	565	259	785	195	565	13	785	10		
	1^{st} FLR Ext. NLB	1150	0	777	0	750	259	1070	195	750	13	1070	10		

Exterior Headers

Short - Grid 3-8 & 10-12 & 15-20

	H (ft)	Ext. Wall Weight (plf)	Window L (ft)	BRNG Stud PNT Load (#)	FLR Loads DL (plf)	FLR Loads LL(LLr/S) (plf)	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	0.00 '	50.00	4		254.43	203.54	304.43	203.54
4th FLR	7.72 '	202.34	4	404.69	254.43	407.08	456.77	407.08
3rd FLR	7.72 '	237.24	4	474.48	254.43	407.08	491.66	407.08
2nd FLR	7.72 '	237.24	4	474.48	254.43	407.08	491.66	407.08
1st FLR	7.72 '							

King Stud Loads

	Header	Wind Pres. (psf)	Window L (ft)	Stud Spac. (in)	Trib. Width (ft)	Wind (plf)	H (ft)	Wind (#-ft)
Roof	4th FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
4th FLR	3rd FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
3rd FLR	2nd FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
2nd FLR	1st FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
1st FLR								

Header Loads Entered In TEDDS			
	Header	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	4th FLR	304.43	203.54
4th FLR	3rd FLR	456.77	407.08
3rd FLR	2nd FLR	491.66	407.08
2nd FLR	1st FLR	491.66	407.08
1st FLR			

Bearing Stud Loads	
DL (#)	LL (#)
608.85	0.00
913.54	814.16
983.33	814.16
983.33	814.16

King Stud + Cumulative Bearing Stud Loads		
DL (#)	LL (#)	Wind (#-ft)
608.85	0.00	781.68
1522.39	814.16	781.68
2505.72	1628.32	781.68
3489.05	2442.48	781.68

Med - Grid (Placeholder)

	H (ft)	Ext. Wall Weight (plf)	Window L (ft)	BRNG Stud PNT Load (#)	FLR Loads DL (plf)	FLR Loads LL(LLr/S) (plf)	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	0.00 '	50.00	4		304.43	243.54	354.43	243.54
4th FLR	7.72 '	202.34	4	404.69	304.43	487.08	506.77	487.08
3rd FLR	7.72 '	237.24	4	474.48	304.43	487.08	541.66	487.08
2nd FLR	7.72 '	237.24	4	474.48	304.43	487.08	541.66	487.08
1st FLR	7.72 '							

King Stud Loads

	Header	Wind Pres. (psf)	Window L (ft)	Stud Spac. (in)	Trib. Width (ft)	Wind (plf)	H (ft)	Wind (#-ft)
Roof	4th FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
4th FLR	3rd FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
3rd FLR	2nd FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
2nd FLR	1st FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
1st FLR								

Header Loads Entered In TEDDS			
	Header	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	4th FLR	354.43	243.54
4th FLR	3rd FLR	506.77	487.08
3rd FLR	2nd FLR	541.66	487.08
2nd FLR	1st FLR	541.66	487.08
1st FLR			

Bearing Stud Loads	
DL (#)	LL (#)
708.85	0.00
1013.54	974.16
1083.33	974.16
1083.33	974.16

King Stud + Cumulative Bearing Stud Loads		
DL (#)	LL (#)	Wind (#-ft)
708.85	0.00	781.68
1722.39	974.16	781.68
2805.72	1948.32	781.68
3889.05	2922.48	781.68

Long - Grid 4-6, 8-10, & 17-19

	H (ft)	Ext. Wall Weight (plf)	Window L (ft)	BRNG Stud PNT Load (#)	FLR Loads DL (plf)	FLR Loads LL(LLr/S) (plf)	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	0.00 '	50.00	4		323.00	258.40	373.00	258.40
4th FLR	7.72 '	202.34	4	404.69	323.00	516.80	525.34	516.80
3rd FLR	7.72 '	237.24	4	474.48	323.00	516.80	560.24	516.80
2nd FLR	7.72 '	237.24	4	474.48	323.00	516.80	560.24	516.80
1st FLR	7.72 '							

King Stud Loads

	Header	Wind Pres. (psf)	Window L (ft)	Stud Spac. (in)	Trib. Width (ft)	Wind (plf)	H (ft)	Wind (#-ft)
Roof	4th FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
4th FLR	3rd FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
3rd FLR	2nd FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
2nd FLR	1st FLR	39.36	4	16	2.67	104.96	7.72 '	781.68
1st FLR								

Header Loads Entered In TEDDS			
	Header	Total Loads DL (plf)	Total Loads LL(LLr/S) (#)
Roof	4th FLR	373.00	258.40
4th FLR	3rd FLR	525.34	516.80
3rd FLR	2nd FLR	560.24	516.80
2nd FLR	1st FLR	560.24	516.80
1st FLR			

Bearing Stud Loads	
DL (#)	LL (#)
746.00	0.00
1050.69	1033.60
1120.48	1033.60
1120.48	1033.60

King Stud + Cumulative Bearing Stud Loads		
DL (#)	LL (#)	Wind (#-ft)
746.00	0.00	781.68
1796.69	1033.60	781.68
2917.17	2067.20	781.68
4037.65	3100.80	781.68

Applicable Load Combinations (2015 IBC)

1.) D + L
2.) D + 0.6W
3.) D + 0.75L + 0.75(0.6W) + 0.75S

Worst Case					
	Bearing Stud Loads		King Stud Loads		
	DL (#)	LL (#)	DL (#)	LL (#)	Wind (#-ft)
4^{th} FLR	746.00	0.00	746.00	0.00	781.68
3^{rd} FLR	1050.69	1033.60	1796.69	1033.60	781.68
2^{nd} FLR	1120.48	1033.60	2917.17	2067.20	781.68
1^{st} FLR	1120.48	1033.60	4037.65	3100.80	781.68

Bearing Studs	King Studs				
1	1	2		3	
(#)	(#)	(#)	(#-ft)	(#)	(#-ft)
746.00	746.00	746.00	469.01	746.00	351.76
2084.29	2830.29	1796.69	469.01	2571.89	351.76
2154.08	4984.37	2917.17	469.01	4467.57	351.76
2154.08	7138.45	4037.65	469.01	6363.25	351.76

STRUCTURAL ENGINEERS

Project WSS LEE'S SUMMIT Project No. 23-283
Calc. By GL Checked By DN Date 7/28/2023

Interior Headers

Short - Grid 1-2 & 19-20

	H (ft)	Int. Wall Weight (plf)	Door L (ft)	BRNG Stud PNT Load (#)	FLR Loads DL (plf)	FLR Loads LL(LLr/S) (plf)	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	0.00 '	0.00	3		368.25	294.60	368.25	294.60
4th FLR	7.72 '	89.03	3	133.55	368.25	749.40	457.28	749.40
3rd FLR	7.72 '	104.39	3	156.58	368.25	749.40	472.64	749.40
2nd FLR	7.72 '	104.39	3	156.58	368.25	749.40	472.64	749.40
1st FLR	7.72 '							

King Stud Loads

	Header	Wind Pres. (psf)	Door L (ft)	Stud Spac. (in)	Trib. Width (ft)	Wind (plf)	H (ft)	Wind (#-ft)
Roof	4th FLR	5	3	16	2.17	10.83	7.72	80.68
4th FLR	3rd FLR	5	3	16	2.17	10.83	7.72	80.68
3rd FLR	2nd FLR	5	3	16	2.17	10.83	7.72	80.68
2nd FLR	1st FLR	5	3	16	2.17	10.83	7.72	80.68
1st FLR								

Header Loads Entered In TEDDS			
	Header	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	4th FLR	368.25	294.60
4th FLR	3rd FLR	457.28	749.40
3rd FLR	2nd FLR	472.64	749.40
2nd FLR	1st FLR	472.64	749.40
1st FLR			

Bearing Stud Loads	
DL (#)	LL (#)
552.38	0.00
685.92	1124.10
708.95	1124.10
708.95	1124.10

King Stud + Cumulative Bearing Stud Loads		
DL (#)	LL (#)	Wind (#-ft)
552.38	0.00	80.68
1238.30	1124.10	80.68
1947.25	2248.20	80.68
2656.20	3372.30	80.68

Medium - Grid 2-4, 6-8, 10-11, 13-15, & 17-19

	H (ft)	Int. Wall Weight (plf)	Door L (ft)	BRNG Stud PNT Load (#)	FLR Loads DL (plf)	FLR Loads LL(LLr/S) (plf)	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	0.00 '	0.00	3		374.74	299.79	374.74	299.79
4^{th} FLR	7.72 '	89.03	3	133.55	372.25	755.80	461.28	755.80
3^{rd} FLR	7.72 '	104.39	3	156.58	372.25	755.80	476.64	755.80
2^{nd} FLR	7.72 '	104.39	3	156.58	372.25	755.80	476.64	755.80
1^{st} FLR	7.72 '							

King Stud Loads

	Header	Wind Pres. (psf)	Door L (ft)	Stud Spac. (in)	Trib. Width (ft)	Wind (plf)	H (ft)	Wind (#-ft)
Roof	4^{th} FLR	5	3	16	2.17	10.83	7.72	80.68
4^{th} FLR	3^{rd} FLR	5	3	16	2.17	10.83	7.72	80.68
3^{rd} FLR	2^{nd} FLR	5	3	16	2.17	10.83	7.72	80.68
2^{nd} FLR	1^{st} FLR	5	3	16	2.17	10.83	7.72	80.68
1^{st} FLR								

Header Loads Entered In TEDDS			
	Header	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	4^{th} FLR	374.74	299.79
4^{th} FLR	3^{rd} FLR	461.28	755.80
3^{rd} FLR	2^{nd} FLR	476.64	755.80
2^{nd} FLR	1^{st} FLR	476.64	755.80
1^{st} FLR			

Bearing Stud Loads	
DL (#)	LL (#)
562.11	0.00
691.92	1133.70
714.95	1133.70
714.95	1133.70

King Stud + Cumulative Bearing Stud Loads		
DL (#)	LL (#)	Wind (#-ft)
562.11	0.00	80.68
1254.03	1133.70	80.68
1968.98	2267.40	80.68
2683.93	3401.10	80.68

Long - Grid 4-6, 8-10, 11-13, & 15-17

	H (ft)	Int. Wall Weight (plf)	Door L (ft)	BRNG Stud PNT Load (#)	FLR Loads DL (plf)	FLR Loads LL(LLr/S) (plf)	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	0.00 '	0.00	3		393.31	314.65	393.31	314.65
4th FLR	7.72 '	89.03	3	133.55	393.31	798.05	482.34	798.05
3rd FLR	7.72 '	104.39	3	156.58	393.31	798.05	497.70	798.05
2nd FLR	7.72 '	104.39	3	156.58	393.31	798.05	497.70	798.05
1st FLR	7.72 '							

King Stud Loads

	Header	Wind Pres. (psf)	Door L (ft)	Stud Spac. (in)	Trib. Width (ft)	Wind (plf)	H (ft)	Wind (#-ft)
Roof	4th FLR	5	3	16	2.17	10.83	7.72	80.68
4th FLR	3rd FLR	5	3	16	2.17	10.83	7.72	80.68
3rd FLR	2nd FLR	5	3	16	2.17	10.83	7.72	80.68
2nd FLR	1st FLR	5	3	16	2.17	10.83	7.72	80.68
1st FLR								

Header Loads Entered In TEDDS			
	Header	Total Loads DL (plf)	Total Loads LL(LLr/S) (plf)
Roof	4th FLR	393.31	314.65
4th FLR	3rd FLR	482.34	798.05
3rd FLR	2nd FLR	497.70	798.05
2nd FLR	1st FLR	497.70	798.05
1st FLR			

Bearing Stud Loads	
DL (#)	LL (#)
589.97	0.00
723.52	1197.08
746.55	1197.08
746.55	1197.08

King Stud + Cumulative Bearing Stud Loads		
DL (#)	LL (#)	Wind (#-ft)
589.97	0.00	80.68
1313.48	1197.08	80.68
2060.03	2394.15	80.68
2806.58	3591.23	80.68

Applicable Load Combinations (2015 IBC)

1.) D + L
2.) D + 0.6W
3.) D + 0.75L + 0.75(0.6W) + 0.75S

Worst Case					
	Bearing Stud Loads		King Stud Loads		
	DL (#)	LL (#)	DL (#)	LL (#)	Wind (#-ft)
4th FLR	589.97	0.00	589.97	0.00	80.68
3rd FLR	723.52	1197.08	1313.48	1197.08	80.68
2nd FLR	746.55	1197.08	2060.03	2394.15	80.68
1st FLR	746.55	1197.08	2806.58	3591.23	80.68

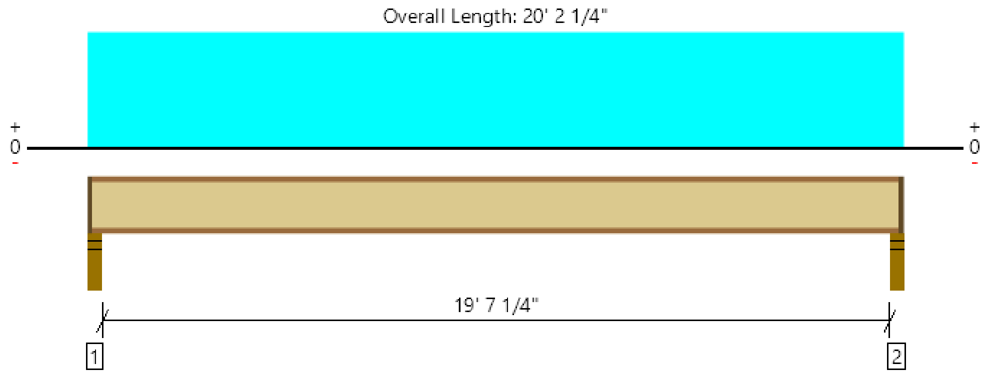
Bearing Studs	King Studs				
1	1	2		3	
(#)	(#)	(#)	(#-ft)	(#)	(#-ft)
589.97	589.97	589.97	48.41	589.97	36.31
1920.59	2510.56	1313.48	48.41	2211.29	36.31
1943.62	4454.18	2060.03	48.41	3855.64	36.31
1943.62	6397.80	2806.58	48.41	5500.00	36.31

Joists_Imported

Level			
Member Name	Results	Current Solution	Comments
Private Room - Short 20'-4 1/4"	Passed	1 piece(s) 16" TJI® 360 @ 16" OC	
Private Room - Med 24'-4 1/4"	Passed	1 piece(s) 16" TJI® 360 @ 16" OC	
Private Room - Long 25'-10 1/4"	Passed	1 piece(s) 16" TJI® 360 @ 16" OC	
4 Elevator Lobby - Short 16'-4.5"	Passed	1 piece(s) 16" TJI® 360 @ 16" OC	
4 Elevator Utility - Long 27'	Passed	1 piece(s) 16" TJI® 560 @ 10" OC	
2&3 Elevator Lobby - Short	Passed	1 piece(s) 16" TJI® 560 @ 16" OC	
2&3 Elevator Utility - Long	Passed	1 piece(s) 16" TJI® 560 @ 12" OC	
Corridor - 5'-7 1/2"	Passed	1 piece(s) 2 x 8 DF No.2 @ 16" OC	
Ladder Framing 2x	Passed	1 piece(s) 2 x 8 DF No.2 @ 24" OC	
Ladder Framing TJI	Passed	2 piece(s) 16" TJI® 560 @ 16" OC	

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Private Room - Short 20'-4 1/4"
1 piece(s) 16" TJI® 210 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	866 @ 19' 11 3/4"	1134 (2.25")	Passed (76%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	850 @ 3 1/2"	2190	Passed (39%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4235 @ 10' 1 1/8"	5140	Passed (82%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.276 @ 10' 1 1/8"	0.659	Passed (L/859)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.449 @ 10' 1 1/8"	0.989	Passed (L/529)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	46	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.50"	1.75"	336	538	875	1" Rim Board
2 - Stud wall - DF	3.50"	2.25"	1.75"	336	538	875	1 1/4" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 1" o/c	
Bottom Edge (Lu)	20' o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 20' 2 1/4"	16"	25.0	40.0	Default Load

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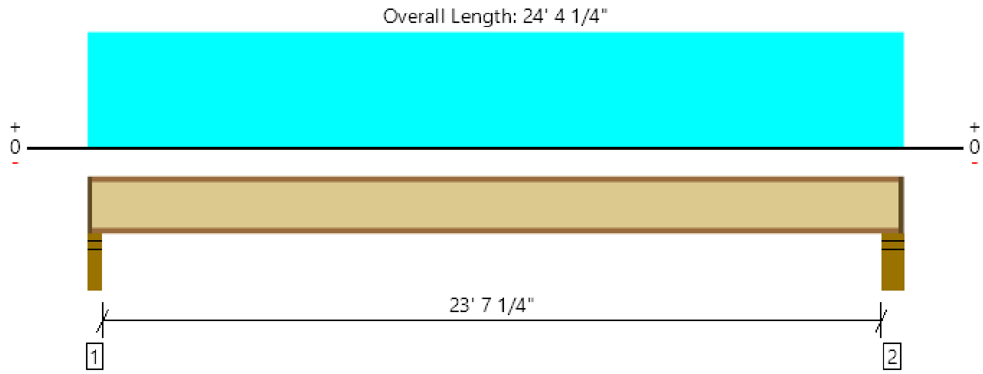
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Private Room - Med 24'-4 1/4"

1 piece(s) 16" TJI® 360 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1041 @ 2 1/2"	1263 (2.50")	Passed (82%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1023 @ 3 1/2"	2190	Passed (47%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	6121 @ 12' 1 1/8"	8405	Passed (73%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.445 @ 12' 1 1/8"	0.792	Passed (L/640)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.724 @ 12' 1 1/8"	1.189	Passed (L/394)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	36	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.50"	1.75"	403	645	1048	1" Rim Board
2 - Stud wall - DF	5.50"	4.25"	1.75"	409	654	1063	1 1/4" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 4" o/c	
Bottom Edge (Lu)	24' 2" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 24' 4 1/4"	16"	25.0	40.0	Default Load

Weyerhaeuser Notes

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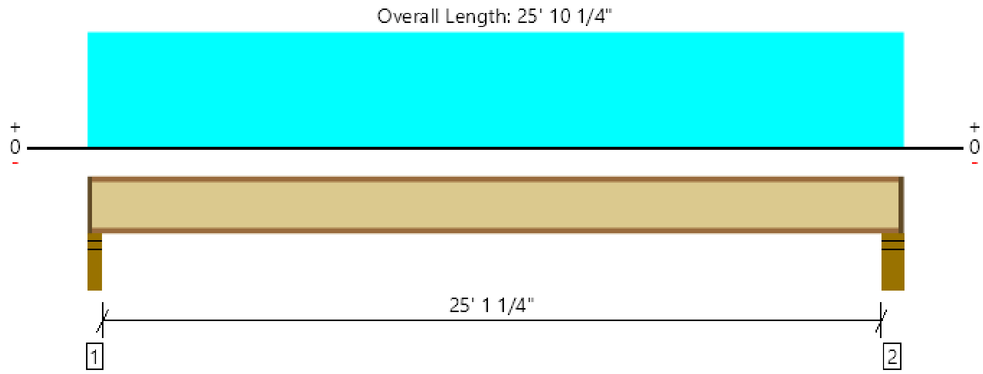
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Private Room - Long 25'-10 1/4"
1 piece(s) 16" TJI® 360 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1106 @ 2 1/2"	1263 (2.50")	Passed (88%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1088 @ 3 1/2"	2190	Passed (50%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	6918 @ 12' 10 1/8"	8405	Passed (82%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.562 @ 12' 10 1/8"	0.842	Passed (L/540)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.913 @ 12' 10 1/8"	1.264	Passed (L/332)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	29	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.50"	1.86"	428	685	1113	1" Rim Board
2 - Stud wall - DF	5.50"	4.25"	1.91"	434	694	1128	1 1/4" Rim Board

• Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	
Bottom Edge (Lu)	25' 8" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 25' 10 1/4"	16"	25.0	40.0	Default Load

Weyerhaeuser Notes

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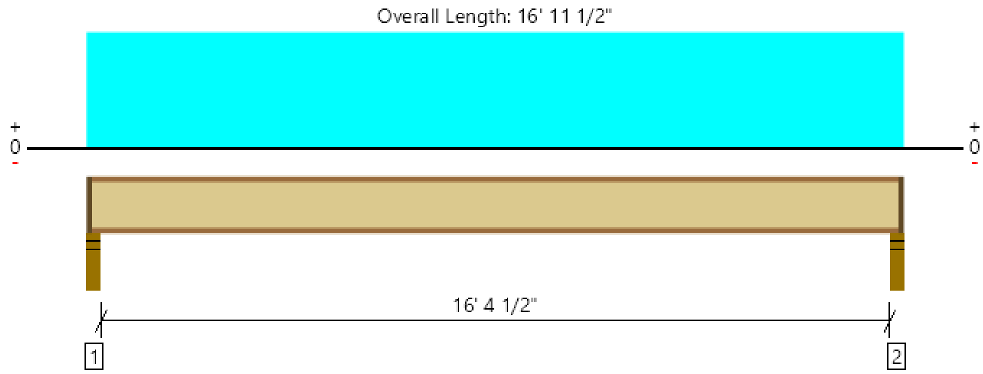
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, 4 Elevator Lobby - Short 16'-4.5"

1 piece(s) 16" TJI® 360 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	503 @ 2 1/2"	1502 (2.25")	Passed (33%)	1.25	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	491 @ 3 1/2"	2738	Passed (18%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	2052 @ 8' 5 3/4"	10506	Passed (20%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.058 @ 8' 5 3/4"	0.551	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.132 @ 8' 5 3/4"	0.827	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
TJ-Pro™ Rating	57	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.75"	283	226	509	1 1/4" Rim Board
2 - Stud wall - DF	3.50"	2.25"	1.75"	283	226	509	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 11" o/c	
Bottom Edge (Lu)	16' 9" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
1 - Uniform (PSF)	0 to 16' 11 1/2"	16"	25.0	20.0	Default Load

Weyerhaeuser Notes

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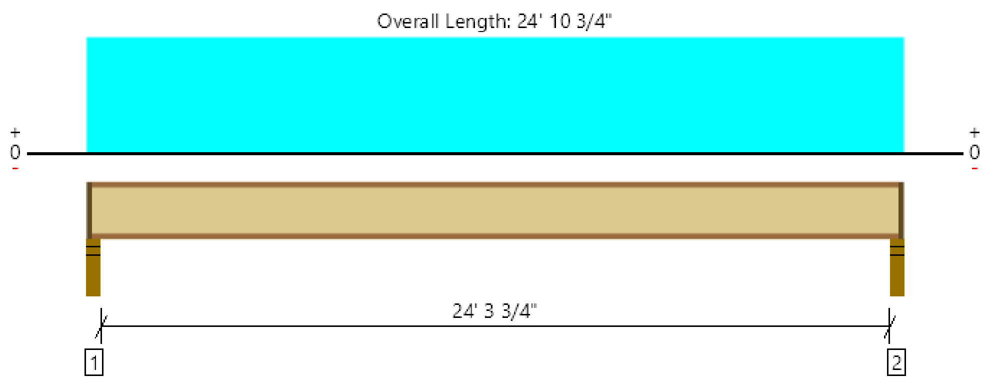
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Weyerhaeuser

Level, 4 Elevator Utility - Long 27'
1 piece(s) 16" TJI® 560 @ 10" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1286 @ 2 1/2"	1396 (2.25")	Passed (92%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1266 @ 3 1/2"	2710	Passed (47%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	7802 @ 12' 5 3/8"	12925	Passed (60%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.568 @ 12' 5 3/8"	0.816	Passed (L/517)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.711 @ 12' 5 3/8"	1.224	Passed (L/413)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	48	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.83"	259	1037	1297	1 1/4" Rim Board
2 - Stud wall - DF	3.50"	2.25"	1.83"	259	1037	1297	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 4" o/c	
Bottom Edge (Lu)	24' 8" o/c	

•TJI joists are only analyzed using Maximum Allowable bracing solutions.
•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 24' 10 3/4"	10"	25.0	100.0	Default Load

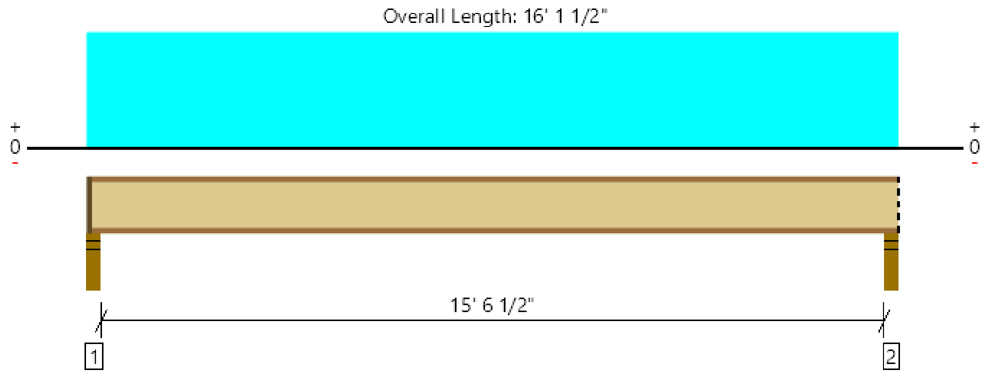
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

MEMBER REPORT

PASSED

Level, 2&3 Elevator Lobby - Short
1 piece(s) 16" TJI® 560 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1326 @ 2 1/2"	1396 (2.25")	Passed (95%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1295 @ 3 1/2"	2710	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	5141 @ 8' 3/4"	12925	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.177 @ 8' 3/4"	0.524	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.221 @ 8' 3/4"	0.785	Passed (L/851)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	62	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.98"	269	1075	1344	1 1/4" Rim Board
2 - Stud wall - DF	3.50"	3.50"	2.05"	269	1075	1344	Blocking

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 3" o/c	
Bottom Edge (Lu)	16' o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 16' 1 1/2"	16"	25.0	100.0	Default Load

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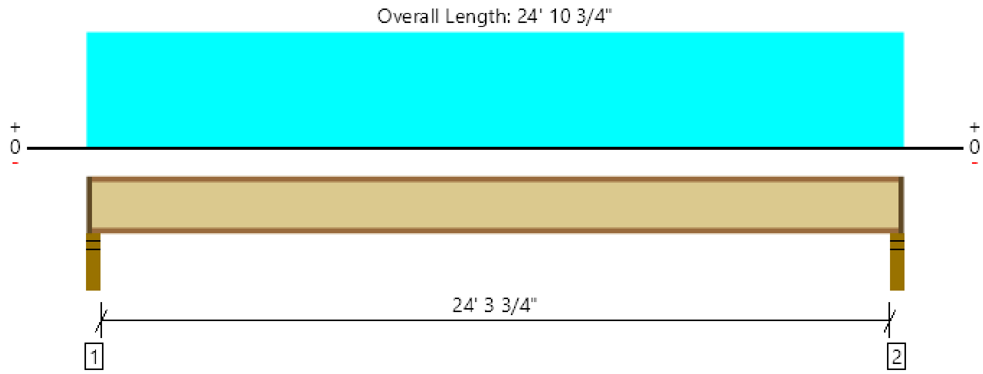
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, 2&3 Elevator Utility - Long
1 piece(s) 16" TJI® 560 @ 12" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1111 @ 2 1/2"	1396 (2.25")	Passed (80%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1094 @ 3 1/2"	2710	Passed (40%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	6741 @ 12' 5 3/8"	12925	Passed (52%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.329 @ 12' 5 3/8"	0.816	Passed (L/893)	--	1.0 D + 0.75 L + 0.75 Lr (All Spans)
Total Load Defl. (in)	0.666 @ 12' 5 3/8"	1.224	Passed (L/441)	--	1.0 D + 0.75 L + 0.75 Lr (All Spans)
TJ-Pro™ Rating	45	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.75"	622	498	311	1229	1 1/4" Rim Board
2 - Stud wall - DF	3.50"	2.25"	1.75"	622	498	311	1229	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 8" o/c	
Bottom Edge (Lu)	24' 8" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

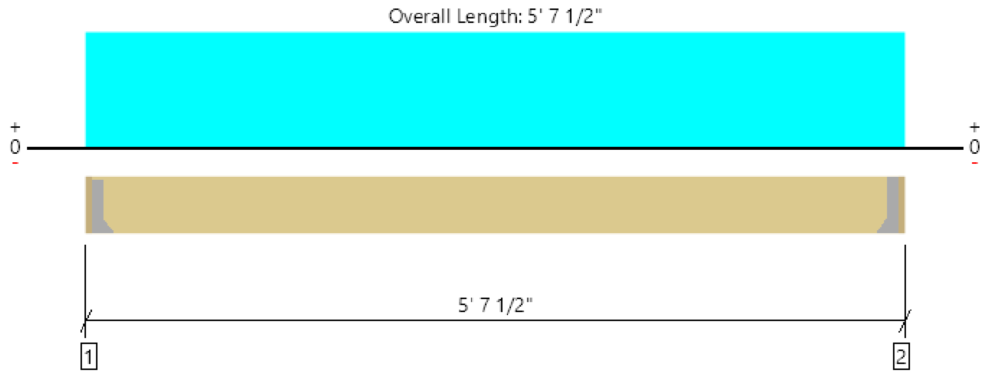
Vertical Loads	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
1 - Uniform (PSF)	0 to 24' 10 3/4"	12"	25.0	40.0	-	Default Load
2 - Uniform (PSF)	0 to 24' 10 3/4"	12"	25.0	-	25.0	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Corridor - 5'-7 1/2"

1 piece(s) 2 x 8 DF No.2 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	448 @ 1 1/2"	1406 (1.50")	Passed (32%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	347 @ 8 3/4"	1305	Passed (27%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	602 @ 2' 9 3/4"	1360	Passed (44%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.033 @ 2' 9 3/4"	0.179	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.041 @ 2' 9 3/4"	0.269	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 7 1/4" DF beam	1.50"	Hanger[1]	1.50"	94	375	469	See note [1]
2 - Hanger on 7 1/4" DF beam	1.50"	Hanger[1]	1.50"	94	375	469	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 5" o/c	
Bottom Edge (Lu)	5' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 5' 7 1/2"	16"	25.0	100.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

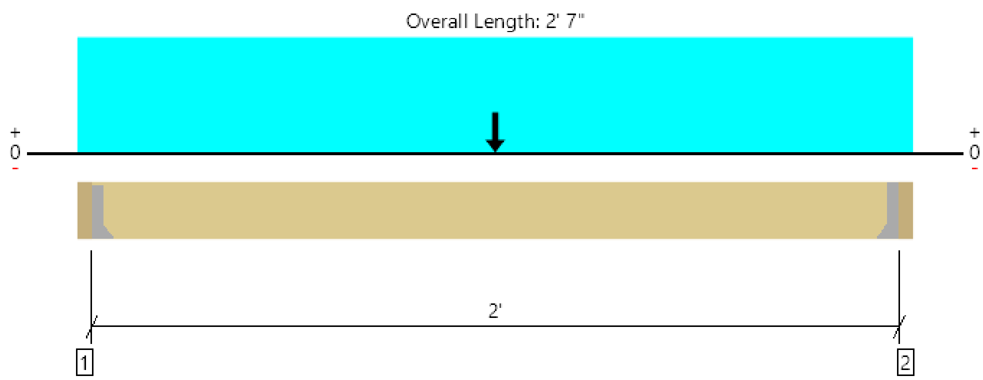
FORTEWEB

MEMBER REPORT

PASSED

Level, Ladder Framing 2x

1 piece(s) 2 x 8 DF No.2 @ 24" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	465 @ 3 1/2"	1406 (1.50")	Passed (33%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	355 @ 10 3/4"	1175	Passed (30%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	360 @ 1' 3 1/2"	1224	Passed (29%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.000 @ 1' 3 1/2"	0.067	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.003 @ 1' 3 1/2"	0.100	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Hanger on 7 1/4" DF beam	3.50"	Hanger[1]	1.50"	400	103	503	See note [1]
2 - Hanger on 7 1/4" DF beam	3.50"	Hanger[1]	1.50"	400	103	503	See note [1]

- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	2' o/c	
Bottom Edge (Lu)	2' o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
1 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Point (lb)	1' 3 1/2"	N/A	670	-	Default Commercial Load
2 - Uniform (PLF)	0 to 2' 7"	N/A	50.0	80.0	

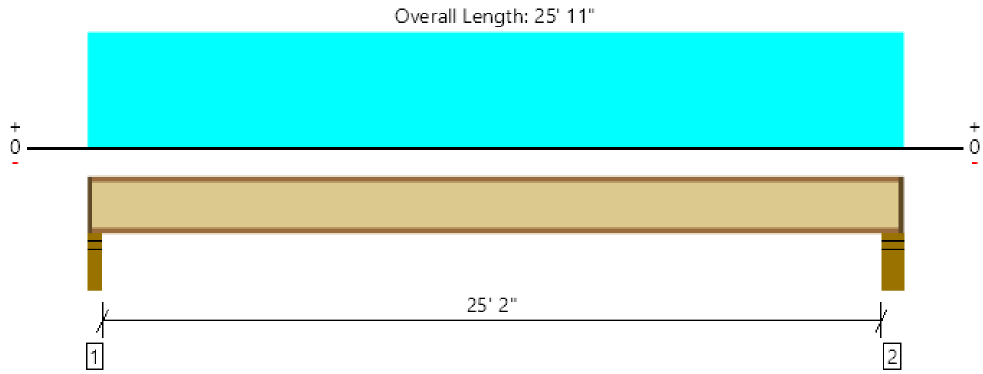
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Weyerhaeuser

Level, Ladder Framing TJI

2 piece(s) 16" TJI® 560 @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2897 @ 2 1/2"	2924 (2.50")	Passed (99%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2850 @ 3 1/2"	5420	Passed (53%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	18170 @ 12' 10 1/2"	25850	Passed (70%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.209 @ 12' 10 1/2"	0.844	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.889 @ 12' 10 1/2"	1.267	Passed (L/342)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	48	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.50"	2.45"	2230	687	2916	1" Rim Board
2 - Stud wall - DF	5.50"	4.25"	2.51"	2258	696	2954	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 9" o/c	
Bottom Edge (Lu)	25' 9" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PLF)	0 to 25' 11"	N/A	142.5	-	
2 - Uniform (PSF)	0 to 25' 11"	16"	23.0	40.0	Default Commercial Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

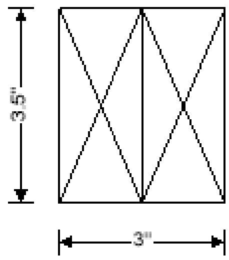
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression — P = **5826** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b =$ **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **10.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **6.12** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **5.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **10.72** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2 — C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Column stability coefficient – cl.15.3.2 K_f = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

c = **0.80**

Column stability factor - eq.15.3-1

$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.39**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2

Applied compressive stress $f_c = P / A$ = **555** lb/in^2

f_c / F_c' = **0.919**

PASS - Design compressive stress exceeds applied compressive stress

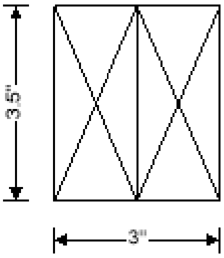
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **45** lb_ft
Design axial compression P = **2634** lb



Sawn lumber section details

Nominal breadth of sections b_{nom} = **2** in
Dressed breadth of sections b = **1.5** in
Nominal depth of sections d_{nom} = **4** in
Dressed depth of sections d = **3.5** in
Number of sections in member N = **2**
Overall breadth of member $b_b = N \times b$ = **3** in
Species, grade and size classification Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain F_b = **900** lb/in^2
Tension parallel to grain F_t = **575** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Compression perpendicular to grain F_{c_perp} = **625** lb/in^2
Shear parallel to grain F_v = **180** lb/in^2
Modulus of elasticity E = **1600000** lb/in^2
Modulus of elasticity, stability calculations E_{min} = **580000** lb/in^2
Mean shear modulus G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition **Dry**
Load duration **Ten years**
Unbraced length in x-axis L_x = **7.72** ft
Effective length factor in x-axis K_x = **1**
Effective length in x-axis $L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis L_y = **1** ft
Effective length factor in y-axis K_y = **1**
Effective length in y-axis $L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member $A = N \times b \times d$ = **10.50** in^2
Section modulus $S_x = N \times b \times d^2 / 6$ = **6.12** in^3
$S_y = d \times (N \times b)^2 / 6$ = **5.25** in^3
Second moment of area $I_x = N \times b \times d^3 / 12$ = **10.72** in^4
$I_y = d \times (N \times b)^3 / 12$ = **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2 — C_D = **1.00**

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

Size factor for bending - Table 4A — C_{Fb} = **1.50**

Size factor for tension - Table 4A — C_{Ft} = **1.50**

Size factor for compression - Table 4A — C_{Fc} = **1.15**

Flat use factor - Table 4A — C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 — C_r = **1.00**

Bearing area factor - cl.3.10.4 — C_b = **1.00**

Column stability coefficient – cl.15.3.2 — K_f = **1.00**

Adjusted modulus of elasticity for column stability — $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value — $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression — $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

c = **0.80**

Column stability factor - eq.15.3-1

$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.39**

Depth-to-breadth ratio — $d_{nom} / (N \times b_{nom})$ = **1.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress — $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1350** lb/in^2

Actual bending stress — $f_b = M_x / S_x$ = **88** lb/in^2

f_b / F_b' = **0.065**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress — $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2

Applied compressive stress — $f_c = P / A$ = **251** lb/in^2

f_c / F_c' = **0.416**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis — $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

Bending and compression check - eq.3.9-3 — $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.276 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

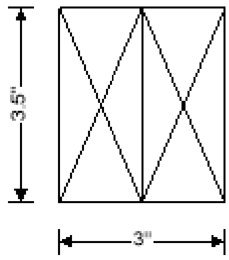
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **34** lb_ft
Design axial compression	P = **5343** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b =$ **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **10.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **6.12** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **5.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **10.72** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.50**
Size factor for tension - Table 4A	C_{Ft} = **1.50**
Size factor for compression - Table 4A	C_{Fc} = **1.15**
Flat use factor - Table 4A	C_{fu} = **1.10**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.00**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Column stability coefficient – cl.15.3.2	K_f = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in²
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in²
	c = **0.80**

Column stability factor - eq.15.3-1

$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.39**

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **1.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Strength in bending - cl.3.3.1

Design bending stress	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1350** lb/in²
Actual bending stress	$f_b = M_x / S_x$ = **67** lb/in²
	f_b / F_b' = **0.049**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in²
Applied compressive stress	$f_c = P / A$ = **509** lb/in²
	f_c / F_c' = **0.843**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis	$F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in²
Bending and compression check - eq.3.9-3	$[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.906** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

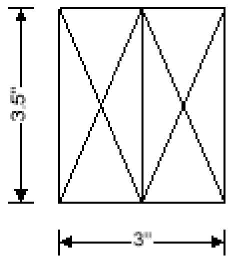
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression	P = **4098** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b =$ **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **10.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **6.12** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **5.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **10.72** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**

Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for bending - Table 4A C_{Fb} = **1.50**
Size factor for tension - Table 4A C_{Ft} = **1.50**
Size factor for compression - Table 4A C_{Fc} = **1.15**
Flat use factor - Table 4A C_{fu} = **1.10**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8
C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9 C_r = **1.00**
Bearing area factor - cl.3.10.4 C_b = **1.00**
Column stability coefficient – cl.15.3.2 K_f = **1.00**
Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in²
Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in²
c = **0.80**
Column stability factor - eq.15.3-1
$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.39**
Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.00**
- Beam is fully restrained
Beam stability factor - cl.3.3.3 C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3
Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in²
Applied compressive stress $f_c = P / A$ = **390** lb/in²
f_c / F_c' = **0.646**
PASS - Design compressive stress exceeds applied compressive stress

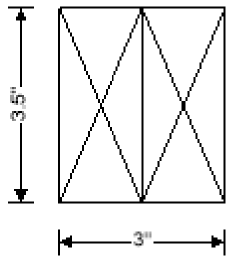
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **45** lb_ft
Design axial compression	P = **1970** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b$ = **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **10.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **6.12** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **5.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **10.72** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**
Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for bending - Table 4A C_{Fb} = **1.50**
Size factor for tension - Table 4A C_{Ft} = **1.50**
Size factor for compression - Table 4A C_{Fc} = **1.15**
Flat use factor - Table 4A C_{fu} = **1.10**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8
C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9 C_r = **1.00**
Bearing area factor - cl.3.10.4 C_b = **1.00**
Column stability coefficient – cl.15.3.2 K_f = **1.00**
Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in²
Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in²
c = **0.80**
Column stability factor - eq.15.3-1
$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.39**
Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.00**
- Beam is fully restrained
Beam stability factor - cl.3.3.3 C_L = **1.00**

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1350** lb/in²
Actual bending stress $f_b = M_x / S_x$ = **88** lb/in²
f_b / F_b' = **0.065**
PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in²
Applied compressive stress $f_c = P / A$ = **188** lb/in²
f_c / F_c' = **0.311**
PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in²
Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.187 < 1**
PASS - Combined compressive and bending stresses are within permissible limits

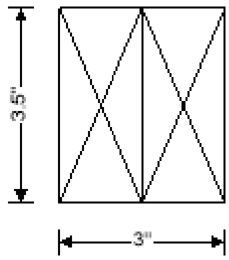
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **34** lb_ft
Design axial compression	P = **3881** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b =$ **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **10.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **6.12** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **5.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **10.72** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**
Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for bending - Table 4A C_{Fb} = **1.50**
Size factor for tension - Table 4A C_{Ft} = **1.50**
Size factor for compression - Table 4A C_{Fc} = **1.15**
Flat use factor - Table 4A C_{fu} = **1.10**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8
C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9 C_r = **1.00**
Bearing area factor - cl.3.10.4 C_b = **1.00**
Column stability coefficient – cl.15.3.2 K_f = **1.00**
Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2
Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2
c = **0.80**
Column stability factor - eq.15.3-1
$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.39**
Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.00**
- Beam is fully restrained
Beam stability factor - cl.3.3.3 C_L = **1.00**

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1350** lb/in^2
Actual bending stress $f_b = M_x / S_x$ = **67** lb/in^2
f_b / F_b' = **0.049**
PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2
Applied compressive stress $f_c = P / A$ = **370** lb/in^2
f_c / F_c' = **0.612**
PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2
Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.483 < 1**
PASS - Combined compressive and bending stresses are within permissible limits

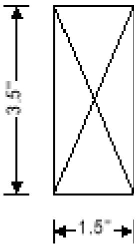
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression $P = \mathbf{1726}$ lb



Sawn lumber section details

Nominal breadth of sections	$b_{nom} = \mathbf{2}$ in
Dressed breadth of sections	$b = \mathbf{1.5}$ in
Nominal depth of sections	$d_{nom} = \mathbf{4}$ in
Dressed depth of sections	$d = \mathbf{3.5}$ in
Number of sections in member	$N = \mathbf{1}$
Overall breadth of member	$b_b = N \times b = \mathbf{1.5}$ in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	$F_b = \mathbf{900}$ lb/in^2
Tension parallel to grain	$F_t = \mathbf{575}$ lb/in^2
Compression parallel to grain	$F_c = \mathbf{1350}$ lb/in^2
Compression perpendicular to grain	$F_{c\_perp} = \mathbf{625}$ lb/in^2
Shear parallel to grain	$F_v = \mathbf{180}$ lb/in^2
Modulus of elasticity	$E = \mathbf{1600000}$ lb/in^2
Modulus of elasticity, stability calculations	$E_{min} = \mathbf{580000}$ lb/in^2
Mean shear modulus	$G_{def} = E / 16 = \mathbf{100000}$ lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	$L_x = \mathbf{7.72}$ ft
Effective length factor in x-axis	$K_x = \mathbf{1}$
Effective length in x-axis	$L_{ex} = L_x \times K_x = \mathbf{7.72}$ ft
Unbraced length in y-axis	$L_y = \mathbf{1}$ ft
Effective length factor in y-axis	$K_y = \mathbf{1}$
Effective length in y-axis	$L_{ey} = L_y \times K_y = \mathbf{1}$ ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d = \mathbf{5.25}$ in^2
Section modulus	$S_x = N \times b \times d^2 / 6 = \mathbf{3.06}$ in^3
	$S_y = d \times (N \times b)^2 / 6 = \mathbf{1.31}$ in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 = \mathbf{5.36}$ in^4
	$I_y = d \times (N \times b)^3 / 12 = \mathbf{0.98}$ in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.39**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2

Applied compressive stress $f_c = P / A$ = **329** lb/in^2

f_c / F_c' = **0.545**

PASS - Design compressive stress exceeds applied compressive stress

STRUCTURAL WOOD MEMBER DESIGN (NDS)

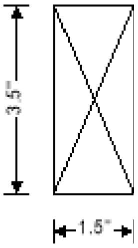
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **45** lb_ft

Design axial compression P = **1307** lb



Sawn lumber section details

Nominal breadth of sections b_{nom} = **2** in

Dressed breadth of sections b = **1.5** in

Nominal depth of sections d_{nom} = **4** in

Dressed depth of sections d = **3.5** in

Number of sections in member N = **1**

Overall breadth of member $b_b = N \times b$ = **1.5** in

Species, grade and size classification Douglas Fir-Larch, No.2 grade, 2" & wider

Bending parallel to grain F_b = **900** lb/in^2

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Compression perpendicular to grain F_{c_perp} = **625** lb/in^2

Shear parallel to grain F_v = **180** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Modulus of elasticity, stability calculations E_{min} = **580000** lb/in^2

Mean shear modulus $G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition **Dry**

Load duration **Ten years**

Unbraced length in x-axis L_x = **7.72** ft

Effective length factor in x-axis K_x = **1**

Effective length in x-axis $L_{ex} = L_x \times K_x$ = **7.72** ft

Unbraced length in y-axis L_y = **1** ft

Effective length factor in y-axis K_y = **1**

Effective length in y-axis $L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

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

Section modulus $S_x = N \times b \times d^2 / 6$ = **3.06** in^3

$S_y = d \times (N \times b)^2 / 6$ = **1.31** in^3

Second moment of area $I_x = N \times b \times d^3 / 12$ = **5.36** in^4

$I_y = d \times (N \times b)^3 / 12$ = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.50**
Size factor for tension - Table 4A	C_{Ft} = **1.50**
Size factor for compression - Table 4A	C_{Fc} = **1.15**
Flat use factor - Table 4A	C_{fu} = **1.10**
Incising factor for modulus of elasticity - Table 4.3.8	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.15**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2
	c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.39**

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **2.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Strength in bending - cl.3.3.1

Design bending stress	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1552** lb/in^2
Actual bending stress	$f_b = M_x / S_x$ = **176** lb/in^2
	f_b / F_b' = **0.114**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2
Applied compressive stress	$f_c = P / A$ = **249** lb/in^2
	f_c / F_c' = **0.412**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis	$F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2
Bending and compression check - eq.3.9-3	$[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.349** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

STRUCTURAL WOOD MEMBER DESIGN (NDS)

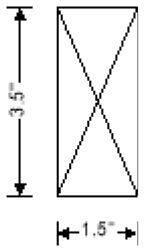
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis — M_x = **34** lb_ft

Design axial compression — P = **2419** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b =$ **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **5.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **3.06** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **1.31** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **5.36** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in²

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in²

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.39**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1552** lb/in²

Actual bending stress $f_b = M_x / S_x$ = **133** lb/in²

f_b / F_b' = **0.086**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in²

Applied compressive stress $f_c = P / A$ = **461** lb/in²

f_c / F_c' = **0.763**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in²

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.848 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

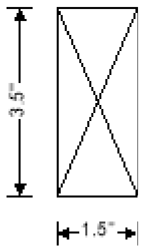
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression — P = **1063** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	L_{ex} = L_x × K_x = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	L_{ey} = L_y × K_y = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	A = N × b × d = **5.25** in^2
Section modulus	S_x = N × b × d^2 / 6 = **3.06** in^3
	S_y = d × (N × b)^2 / 6 = **1.31** in^3
Second moment of area	I_x = N × b × d^3 / 12 = **5.36** in^4
	I_y = d × (N × b)^3 / 12 = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.39**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2

Applied compressive stress $f_c = P / A$ = **202** lb/in^2

f_c / F_c' = **0.335**

PASS - Design compressive stress exceeds applied compressive stress

STRUCTURAL WOOD MEMBER DESIGN (NDS)

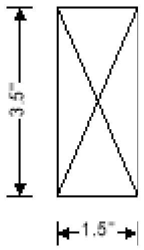
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **33** lb_ft

Design axial compression P = **643** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **5.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **3.06** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **1.31** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **5.36** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.39**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1552** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **129** lb/in^2

f_b / F_b' = **0.083**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2

Applied compressive stress $f_c = P / A$ = **122** lb/in^2

f_c / F_c' = **0.203**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.143 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

STRUCTURAL WOOD MEMBER DESIGN (NDS)

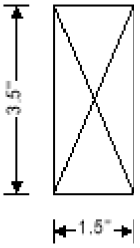
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **25** lb_ft

Design axial compression P = **958** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **5.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **3.06** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **1.31** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **5.36** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.39**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1552** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **98** lb/in^2

f_b / F_b' = **0.063**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **604** lb/in^2

Applied compressive stress $f_c = P / A$ = **182** lb/in^2

f_c / F_c' = **0.302**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **681** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.178 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

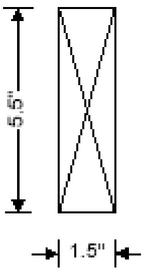
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression P = **5075** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b =$ **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2

Applied compressive stress $f_c = P / A$ = **615** lb/in^2

f_c / F_c' = **0.566**

PASS - Design compressive stress exceeds applied compressive stress

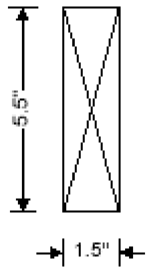
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **259** lb_ft
Design axial compression	P = **3008** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten minutes**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.60**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **2376** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.56**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **2153** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **411** lb/in^2

f_b / F_b' = **0.191**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1337** lb/in^2

Applied compressive stress $f_c = P / A$ = **365** lb/in^2

f_c / F_c' = **0.273**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.318** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

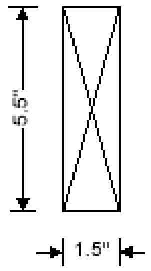
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **195** lb_ft
Design axial compression	P = **4817** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten minutes**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.60**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.30**
Size factor for tension - Table 4A	C_{Ft} = **1.30**
Size factor for compression - Table 4A	C_{Fc} = **1.10**
Flat use factor - Table 4A	C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.15**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **2376** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2
	c = **0.80**

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.56}$$

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **3.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Strength in bending - cl.3.3.1

Design bending stress	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **2153** lb/in^2
Actual bending stress	$f_b = M_x / S_x$ = **309** lb/in^2
	f_b / F_b' = **0.144**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1337** lb/in^2
Applied compressive stress	$f_c = P / A$ = **584** lb/in^2
	f_c / F_c' = **0.437**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis	$F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2
Bending and compression check - eq.3.9-3	$[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.411** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

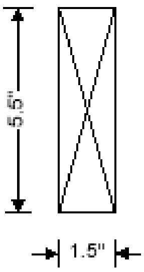
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression — P = **3639** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**
Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for bending - Table 4A C_{Fb} = **1.30**
Size factor for tension - Table 4A C_{Ft} = **1.30**
Size factor for compression - Table 4A C_{Fc} = **1.10**
Flat use factor - Table 4A C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8
C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9 C_r = **1.15**
Bearing area factor - cl.3.10.4 C_b = **1.00**
Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2
c = **0.80**
Column stability factor - eq.3.7-1
$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**
Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**
 - Beam is fully restrained
Beam stability factor - cl.3.3.3 C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2
Applied compressive stress $f_c = P / A$ = **441** lb/in^2
f_c / F_c' = **0.406**

PASS - Design compressive stress exceeds applied compressive stress

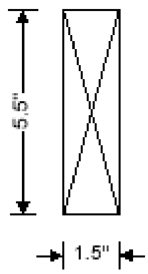
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **259** lb_ft
Design axial compression	P = **2261** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.30**
Size factor for tension - Table 4A	C_{Ft} = **1.30**
Size factor for compression - Table 4A	C_{Fc} = **1.10**
Flat use factor - Table 4A	C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.15**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2
	c = **0.80**

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.73}$$

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1346** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **411** lb/in^2

f_b / F_b' = **0.305**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2

Applied compressive stress $f_c = P / A$ = **274** lb/in^2

f_c / F_c' = **0.252**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.429** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

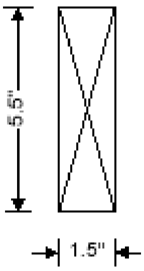
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **195** lb_ft
Design axial compression	P = **3553** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in²

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in²

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1346** lb/in²

Actual bending stress $f_b = M_x / S_x$ = **309** lb/in²

f_b / F_b' = **0.230**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in²

Applied compressive stress $f_c = P / A$ = **431** lb/in²

f_c / F_c' = **0.396**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in²

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.466** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

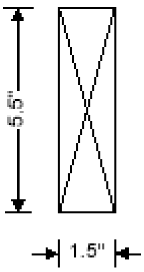
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression	P = **2203** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b =$ **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in²

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in²

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in²

Applied compressive stress $f_c = P / A$ = **267** lb/in²

f_c / F_c' = **0.246**

PASS - Design compressive stress exceeds applied compressive stress

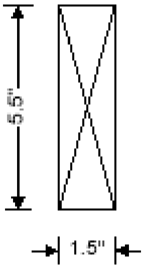
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **259** lb_ft
Design axial compression	P = **1514** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1346** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **411** lb/in^2

f_b / F_b' = **0.305**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2

Applied compressive stress $f_c = P / A$ = **184** lb/in^2

f_c / F_c' = **0.169**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.371** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

STRUCTURAL WOOD MEMBER DESIGN (NDS)

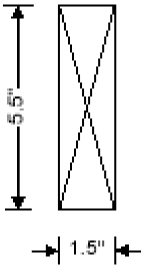
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **195** lb_ft

Design axial compression P = **2289** lb



Sawn lumber section details

Nominal breadth of sections b_{nom} = **2** in

Dressed breadth of sections b = **1.5** in

Nominal depth of sections d_{nom} = **6** in

Dressed depth of sections d = **5.5** in

Number of sections in member N = **1**

Overall breadth of member $b_b = N \times b$ = **1.5** in

Species, grade and size classification Douglas Fir-Larch, No.2 grade, 2" & wider

Bending parallel to grain F_b = **900** lb/in^2

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Compression perpendicular to grain F_{c_perp} = **625** lb/in^2

Shear parallel to grain F_v = **180** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Modulus of elasticity, stability calculations E_{min} = **580000** lb/in^2

Mean shear modulus $G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition **Dry**

Load duration **Ten years**

Unbraced length in x-axis L_x = **7.72** ft

Effective length factor in x-axis K_x = **1**

Effective length in x-axis $L_{ex} = L_x \times K_x$ = **7.72** ft

Unbraced length in y-axis L_y = **1** ft

Effective length factor in y-axis K_y = **1**

Effective length in y-axis $L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member $A = N \times b \times d$ = **8.25** in^2

Section modulus $S_x = N \times b \times d^2 / 6$ = **7.56** in^3

$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3

Second moment of area $I_x = N \times b \times d^3 / 12$ = **20.80** in^4

$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1346** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **309** lb/in^2

f_b / F_b' = **0.230**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2

Applied compressive stress $f_c = P / A$ = **277** lb/in^2

f_c / F_c' = **0.255**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.341** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

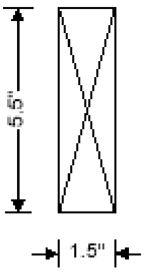
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression	P = **767** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.30**
Size factor for tension - Table 4A	C_{Ft} = **1.30**
Size factor for compression - Table 4A	C_{Fc} = **1.10**
Flat use factor - Table 4A	C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.15**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2
	c = **0.80**

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.73}$$

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **3.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2
Applied compressive stress	$f_c = P / A$ = **93** lb/in^2
	f_c / F_c' = **0.086**

PASS - Design compressive stress exceeds applied compressive stress

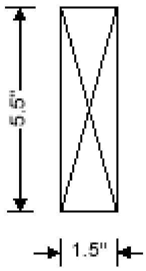
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **189** lb_ft
Design axial compression	P = **767** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **8.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **7.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **20.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1346** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **300** lb/in^2

f_b / F_b' = **0.223**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2

Applied compressive stress $f_c = P / A$ = **93** lb/in^2

f_c / F_c' = **0.086**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.243 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

STRUCTURAL WOOD MEMBER DESIGN (NDS)

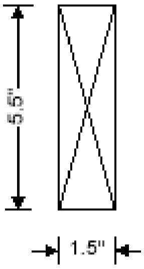
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **141** lb_ft

Design axial compression P = **1026** lb



Sawn lumber section details

Nominal breadth of sections b_{nom} = **2** in

Dressed breadth of sections b = **1.5** in

Nominal depth of sections d_{nom} = **6** in

Dressed depth of sections d = **5.5** in

Number of sections in member N = **1**

Overall breadth of member $b_b = N \times b$ = **1.5** in

Species, grade and size classification Douglas Fir-Larch, No.2 grade, 2" & wider

Bending parallel to grain F_b = **900** lb/in^2

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Compression perpendicular to grain F_{c_perp} = **625** lb/in^2

Shear parallel to grain F_v = **180** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Modulus of elasticity, stability calculations E_{min} = **580000** lb/in^2

Mean shear modulus G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition **Dry**

Load duration **Ten years**

Unbraced length in x-axis L_x = **7.72** ft

Effective length factor in x-axis K_x = **1**

Effective length in x-axis $L_{ex} = L_x \times K_x$ = **7.72** ft

Unbraced length in y-axis L_y = **1** ft

Effective length factor in y-axis K_y = **1**

Effective length in y-axis $L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member $A = N \times b \times d$ = **8.25** in^2

Section modulus $S_x = N \times b \times d^2 / 6$ = **7.56** in^3

$S_y = d \times (N \times b)^2 / 6$ = **2.06** in^3

Second moment of area $I_x = N \times b \times d^3 / 12$ = **20.80** in^4

$I_y = d \times (N \times b)^3 / 12$ = **1.55** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.73**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **3.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1346** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **224** lb/in^2

f_b / F_b' = **0.166**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2

Applied compressive stress $f_c = P / A$ = **124** lb/in^2

f_c / F_c' = **0.114**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.193** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

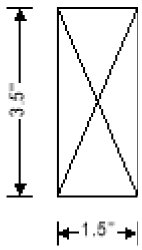
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression — P = **1363** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **9.55** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **9.55** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **5.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **3.06** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **1.31** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **5.36** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in²

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **445** lb/in²

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.27**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **415** lb/in²

Applied compressive stress $f_c = P / A$ = **260** lb/in²

f_c / F_c' = **0.626**

PASS - Design compressive stress exceeds applied compressive stress

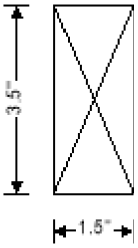
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis	M_x = **45** lb_ft
Design axial compression	P = **803** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **9.55** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **9.55** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **5.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **3.06** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **1.31** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **5.36** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **445** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.27**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1552** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **176** lb/in^2

f_b / F_b' = **0.114**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **415** lb/in^2

Applied compressive stress $f_c = P / A$ = **153** lb/in^2

f_c / F_c' = **0.369**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **445** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.309** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

STRUCTURAL WOOD MEMBER DESIGN (NDS)

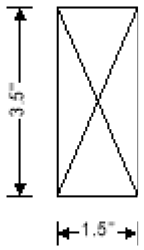
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **34** lb_ft

Design axial compression P = **1263** lb



Sawn lumber section details

Nominal breadth of sections b_{nom} = **2** in

Dressed breadth of sections b = **1.5** in

Nominal depth of sections d_{nom} = **4** in

Dressed depth of sections d = **3.5** in

Number of sections in member N = **1**

Overall breadth of member $b_b = N \times b$ = **1.5** in

Species, grade and size classification Douglas Fir-Larch, No.2 grade, 2" & wider

Bending parallel to grain F_b = **900** lb/in^2

Tension parallel to grain F_t = **575** lb/in^2

Compression parallel to grain F_c = **1350** lb/in^2

Compression perpendicular to grain F_{c_perp} = **625** lb/in^2

Shear parallel to grain F_v = **180** lb/in^2

Modulus of elasticity E = **1600000** lb/in^2

Modulus of elasticity, stability calculations E_{min} = **580000** lb/in^2

Mean shear modulus $G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition **Dry**

Load duration **Ten years**

Unbraced length in x-axis L_x = **9.55** ft

Effective length factor in x-axis K_x = **1**

Effective length in x-axis $L_{ex} = L_x \times K_x$ = **9.55** ft

Unbraced length in y-axis L_y = **1** ft

Effective length factor in y-axis K_y = **1**

Effective length in y-axis $L_{ey} = L_y \times K_y$ = **1** ft

The beam is one of three or more repetitive members

Section properties

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

Section modulus $S_x = N \times b \times d^2 / 6$ = **3.06** in^3

$S_y = d \times (N \times b)^2 / 6$ = **1.31** in^3

Second moment of area $I_x = N \times b \times d^3 / 12$ = **5.36** in^4

$I_y = d \times (N \times b)^3 / 12$ = **0.98** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.10**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **445** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.27**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1552** lb/in^2

Actual bending stress $f_b = M_x / S_x$ = **133** lb/in^2

f_b / F_b' = **0.086**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **415** lb/in^2

Applied compressive stress $f_c = P / A$ = **241** lb/in^2

f_c / F_c' = **0.580**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **445** lb/in^2

Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})])$ = **0.524** **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Level			
Member Name	Results	Current Solution	Comments
Guest Room Door Header	Passed	2 piece(s) 2 x 8 DF No.2	
Cross Stair	Passed	2 piece(s) 2 x 8 DF No.2	
Exterior Hallway Window	Passed	3 piece(s) 2 x 8 DF No.2	
Exterior Windows 4'	Passed	3 piece(s) 2 x 8 DF No.2	
Elevator Lobby	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Vestibule Entrance	Passed	1 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Vestibule Window	Passed	2 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Long Corridor Opening	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Vestibule to Lobby Entrance	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
room windows -(3) 2x10 4'	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Elevator Lobby F1	Passed	4 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Lobby to Corridor Door	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Lobby to Corridor Window	Passed	3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL	
Staff Laundry Door	Passed	3 piece(s) 2 x 8 DF No.1	

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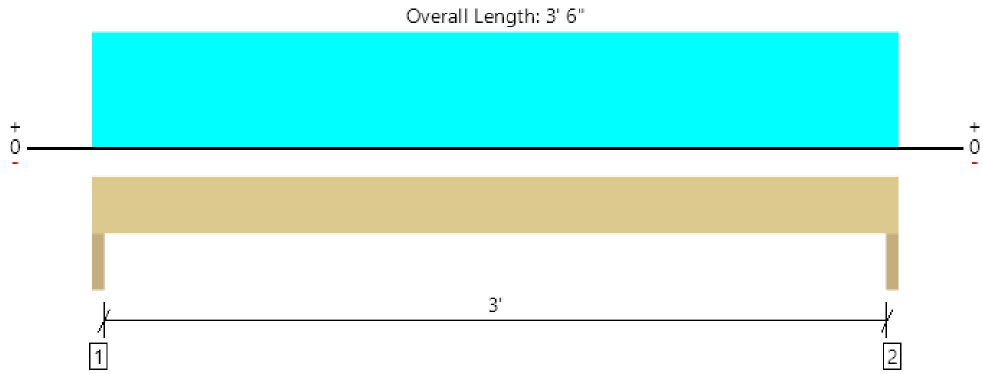
MEMBER REPORT

PASSED

Level, Guest Room Door Header
2 piece(s) 2 x 8 DF No.2



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2095 @ 1 1/2"	5625 (3.00")	Passed (37%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1073 @ 10 1/4"	2610	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1581 @ 1' 9"	2365	Passed (67%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.013 @ 1' 9"	0.108	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.020 @ 1' 9"	0.162	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - DF	3.00"	3.00"	1.50"	698	1397	2095	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	698	1397	2095	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 6" o/c	
Bottom Edge (Lu)	3' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3' 6"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 3' 6"	12' 11 1/8"	25.0	40.0	Default Load
2 - Uniform (PSF)	0 to 3' 6"	2' 9 3/4"	25.0	100.0	

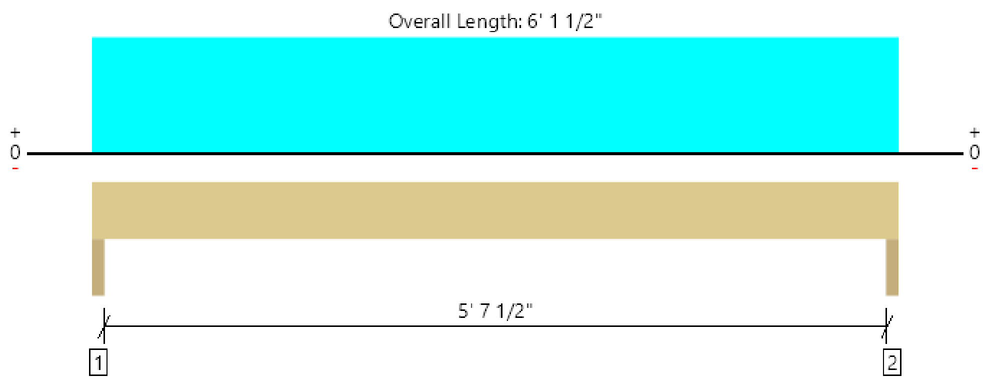
Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

MEMBER REPORT

PASSED

Level, Cross Stair
2 piece(s) 2 x 8 DF No.2



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	782 @ 1 1/2"	5625 (3.00")	Passed (14%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	564 @ 10 1/4"	3263	Passed (17%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	1102 @ 3' 3/4"	2957	Passed (37%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.022 @ 3' 3/4"	0.196	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.045 @ 3' 3/4"	0.294	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Factored	
1 - Trimmer - DF	3.00"	3.00"	1.50"	400	383	782	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	400	383	782	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 2" o/c	
Bottom Edge (Lu)	6' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 6' 1 1/2"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 6' 1 1/2"	5'	25.0	25.0	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

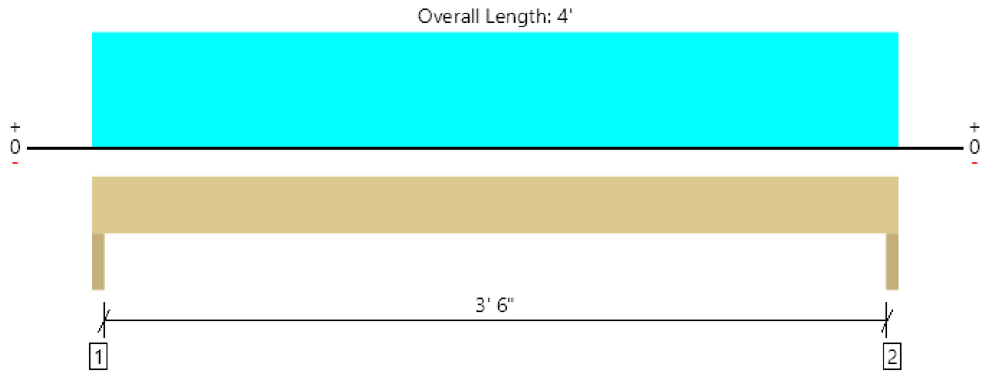
FORTEWEB

MEMBER REPORT

PASSED

Level, Exterior Hallway Window

3 piece(s) 2 x 8 DF No.2



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1067 @ 1 1/2"	8438 (3.00")	Passed (13%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	611 @ 10 1/4"	3915	Passed (16%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	937 @ 2'	3548	Passed (26%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.008 @ 2'	0.075	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.010 @ 2'	0.188	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/600) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Trimmer - DF	3.00"	3.00"	1.50"	267	800	40	1067	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	267	800	40	1067	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	
Bottom Edge (Lu)	4' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 4'	N/A	8.3	--	--	
1 - Uniform (PSF)	0 to 4'	1'	100.0	300.0	20.0	default
2 - Uniform (PSF)	0 to 4'	1'	25.0	100.0	-	

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

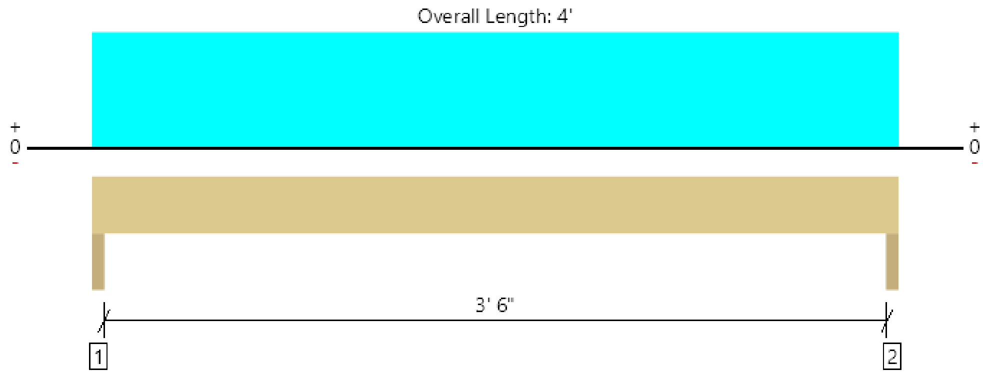
Weyerhaeuser

MEMBER REPORT

PASSED

Level, Exterior Windows 4'

3 piece(s) 2 x 8 DF No.2



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1652 @ 1 1/2"	8438 (3.00")	Passed (20%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	947 @ 10 1/4"	3915	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1452 @ 2'	3548	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.010 @ 2'	0.125	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.016 @ 2'	0.075	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/600).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Trimmer - DF	3.00"	3.00"	1.50"	646	1007	1652	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	646	1007	1652	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' o/c	
Bottom Edge (Lu)	4' o/c	

•Maximum allowable bracing intervals based on applied load.

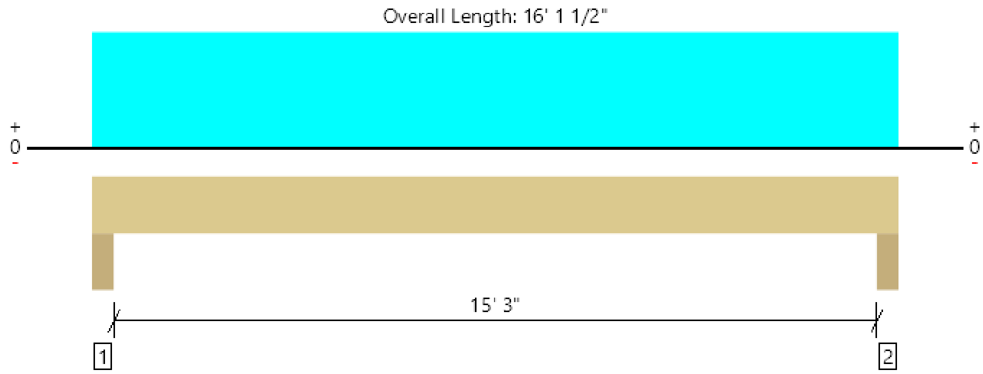
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 4'	N/A	8.3	--	
1 - Uniform (PSF)	0 to 4'	12' 7"	25.0	40.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Elevator Lobby

3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	11724 @ 3 3/4"	20672 (5.25")	Passed (57%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	9149 @ 1' 9 1/4"	15960	Passed (57%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	43671 @ 8' 3/4"	46671	Passed (94%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.462 @ 8' 3/4"	0.517	Passed (L/403)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.587 @ 8' 3/4"	0.775	Passed (L/317)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Trimmer - DF	5.25"	5.25"	2.98"	2503	9221	615	11724	None
2 - Trimmer - DF	5.25"	5.25"	2.98"	2503	9221	615	11724	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 8" o/c	
Bottom Edge (Lu)	16' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

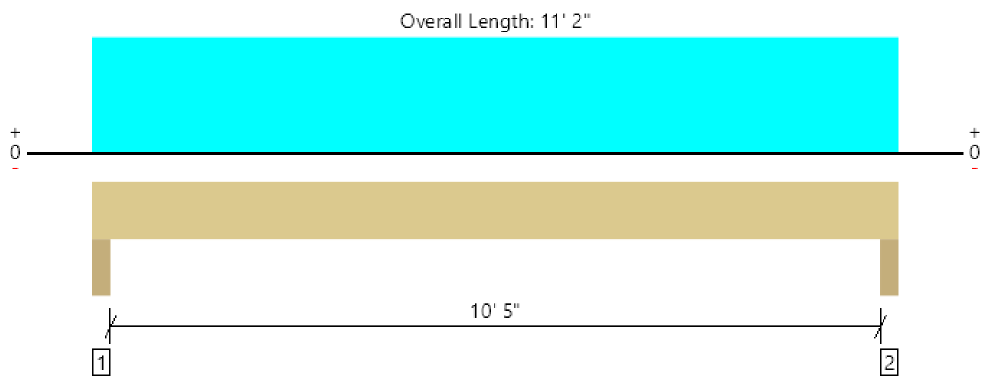
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 16' 1 1/2"	N/A	24.5	--	--	
1 - Uniform (PSF)	0 to 16' 1 1/2"	3' 9 3/4"	75.0	300.0	20.0	Default Load
2 - Uniform (PSF)	0 to 16' 1 1/2"	2' 9 3/4"	-	-	-	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Vestibule Entrance

1 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5593 @ 3"	5906 (4.50")	Passed (95%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	3800 @ 1' 8 1/2"	5320	Passed (71%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	13946 @ 5' 7"	15557	Passed (90%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.058 @ 5' 7"	0.356	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.303 @ 5' 7"	0.533	Passed (L/423)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Snow	Factored	
1 - Trimmer - DF	4.50"	4.50"	4.26"	4526	949	475	475	5593	None
2 - Trimmer - DF	4.50"	4.50"	4.26"	4526	949	475	475	5593	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	2' 10" o/c	
Bottom Edge (Lu)	11' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

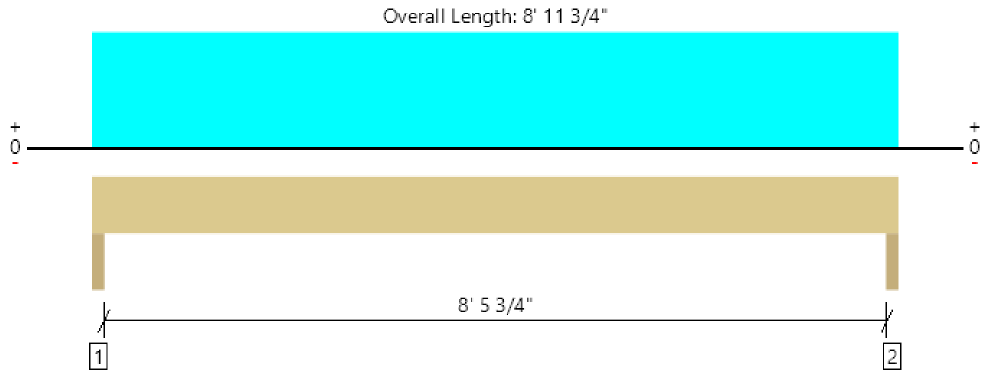
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 11' 2"	N/A	8.2	--	--	--	
1 - Uniform (PSF)	0 to 11' 2"	4' 3"	92.8	40.0	20.0	20.0	Three Floors and Roof
2 - Uniform (PLF)	0 to 11' 2"	N/A	408.0	-	-	-	34' of exterior wall

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Vestibule Window

2 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4028 @ 1 1/2"	7875 (3.00")	Passed (51%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	2607 @ 1' 7"	12236	Passed (21%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	8545 @ 4' 5 7/8"	35781	Passed (24%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.019 @ 4' 5 7/8"	0.291	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.067 @ 4' 5 7/8"	0.436	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Trimmer - DF	3.00"	3.00"	1.53"	2898	1130	1130	4028	None
2 - Trimmer - DF	3.00"	3.00"	1.53"	2898	1130	1130	4028	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' o/c	
Bottom Edge (Lu)	9' o/c	

•Maximum allowable bracing intervals based on applied load.

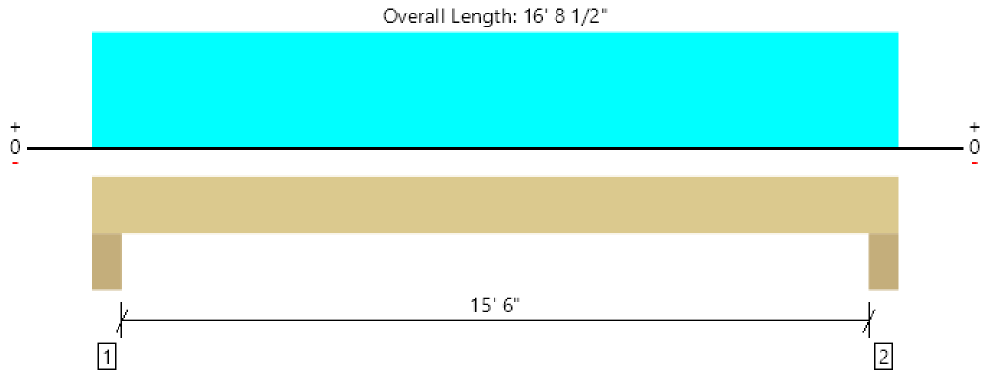
Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 8' 11 3/4"	N/A	16.3	--	--	
1 - Uniform (PSF)	0 to 8' 11 3/4"	12' 7"	50.0	20.0	20.0	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Long Corridor Opening

3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	12377 @ 5 3/4"	28547 (7.25")	Passed (43%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	9506 @ 1' 11 1/4"	15960	Passed (60%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	45939 @ 8' 4 1/4"	46671	Passed (98%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.500 @ 8' 4 1/4"	0.525	Passed (L/378)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.635 @ 8' 4 1/4"	0.788	Passed (L/297)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Snow	Factored	
1 - Trimmer - DF	7.25"	7.25"	3.14"	2639	9738	649	649	12377	None
2 - Trimmer - DF	7.25"	7.25"	3.14"	2639	9738	649	649	12377	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	1' 10" o/c	
Bottom Edge (Lu)	16' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

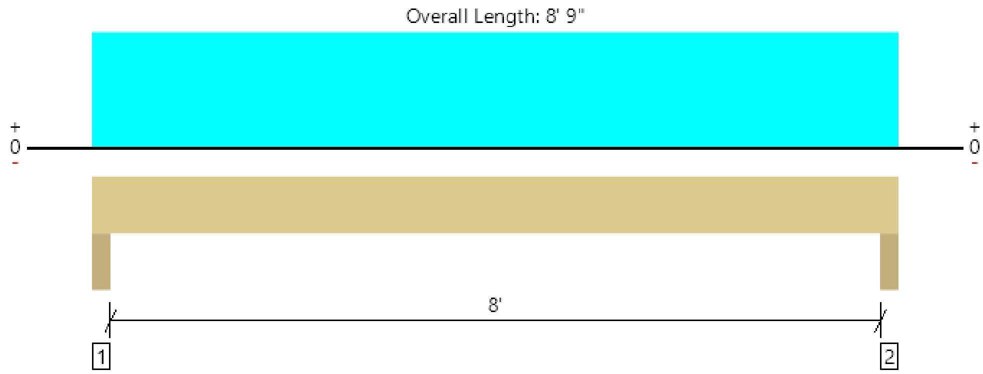
Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 16' 8 1/2"	N/A	24.5	--	--	--	
1 - Uniform (PSF)	0 to 16' 8 1/2"	3' 10 5/8"	75.0	300.0	20.0	20.0	Two floors and roof

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Vestibule to Lobby Entrance

3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	11430 @ 3"	17719 (4.50")	Passed (65%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	6967 @ 1' 8 1/2"	15960	Passed (44%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	22228 @ 4' 4 1/2"	46671	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.062 @ 4' 4 1/2"	0.275	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.106 @ 4' 4 1/2"	0.412	Passed (L/930)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Trimmer - DF	4.50"	4.50"	2.90"	4748	6683	371	11430	None
2 - Trimmer - DF	4.50"	4.50"	2.90"	4748	6683	371	11430	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 9" o/c	
Bottom Edge (Lu)	8' 9" o/c	

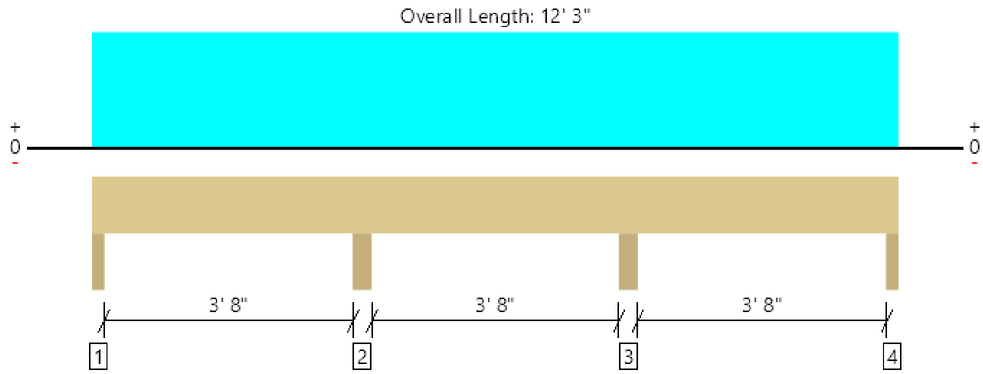
•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 8' 9"	N/A	24.5	--	--	
1 - Uniform (PSF)	0 to 8' 9"	12' 8 3/4"	75.0	120.0	-	Default Load
2 - Uniform (PSF)	0 to 8' 9"	4' 2 7/8"	25.0	-	20.0	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, room windows -(3) 2x10 4'
3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	14478 @ 4' 1 1/4"	17128 (4.50")	Passed (85%)	--	1.0 D + 1.0 L (Adj Spans)
Shear (lbs)	2845 @ 2' 7"	15960	Passed (18%)	1.00	1.0 D + 1.0 L (Adj Spans)
Moment (Ft-lbs)	-5452 @ 8' 1 3/4"	46671	Passed (12%)	1.00	1.0 D + 1.0 L (Adj Spans)
Live Load Defl. (in)	0.006 @ 10' 2 5/16"	0.133	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)
Total Load Defl. (in)	0.012 @ 10' 2 7/8"	0.080	Passed (L/999+)	--	1.0 D + 1.0 L (Alt Spans)

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/600).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Trimmer - DF	3.00"	3.00"	1.50"	2800	2895/-312	571	5695	None
2 - Trimmer - SPF	4.50"	4.50"	3.80"	7222	7256	1450	14478	None
3 - Trimmer - SPF	4.50"	4.50"	3.80"	7222	7256	1450	14478	None
4 - Trimmer - DF	3.00"	3.00"	1.50"	2800	2895/-312	571	5695	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 3" o/c	
Bottom Edge (Lu)	12' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 12' 3"	N/A	24.5	--	--	
1 - Uniform (PSF)	0 to 12' 3"	12' 7"	92.8	120.0	25.0	Three floors and roof
2 - Uniform (PLF)	0 to 12' 3"	N/A	444.0	-	-	37' exterior wall

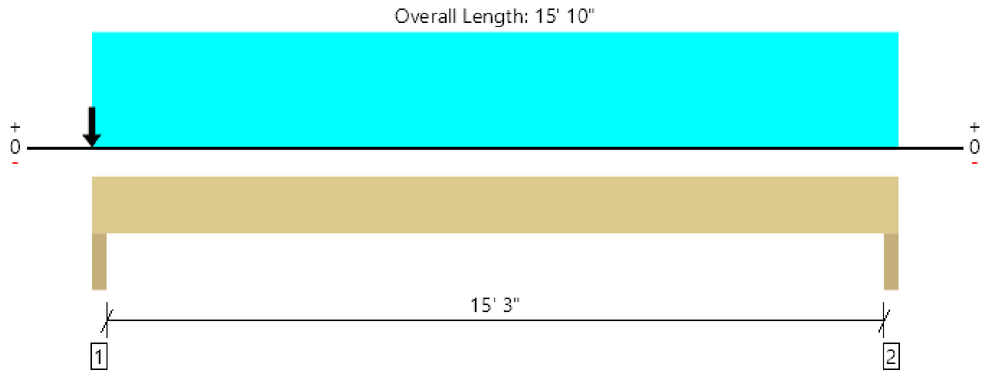
Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Elevator Lobby F1

4 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	15904 @ 2"	18375 (3.50")	Passed (87%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	12314 @ 1' 7 1/2"	21280	Passed (58%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	58776 @ 7' 11"	62228	Passed (94%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.466 @ 7' 11"	0.517	Passed (L/399)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.592 @ 7' 11"	0.775	Passed (L/314)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)					Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Snow	Factored	
1 - Trimmer - DF	3.50"	3.50"	3.03"	3716	12188	328	459	15904	None
2 - Trimmer - DF	3.50"	3.50"	2.95"	3306	12188	-	-	15494	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 3" o/c	
Bottom Edge (Lu)	15' 10" o/c	

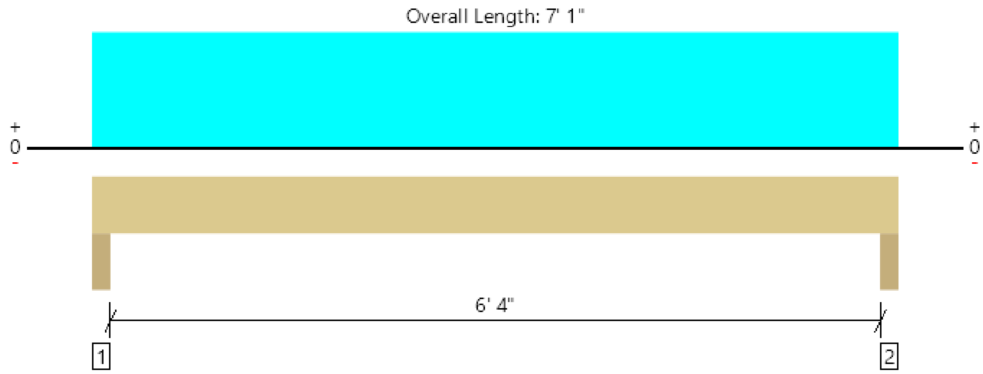
•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 15' 10"	N/A	32.7	--	--	--	
1 - Uniform (PSF)	0 to 15' 10"	12' 7"	25.0	100.0	-	-	Default Load
2 - Uniform (PSF)	0 to 15' 10"	2' 9 3/4"	25.0	100.0	-	-	
3 - Point (PLF)	0	16' 4 3/4"	25.0	-	20.0	28.0	(1) H5 Roof Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Lobby to Corridor Door
3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	15041 @ 3"	17719 (4.50")	Passed (85%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	7786 @ 1' 8 1/2"	15960	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	23007 @ 3' 6 1/2"	46671	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.046 @ 3' 6 1/2"	0.219	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.082 @ 3' 6 1/2"	0.329	Passed (L/968)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Trimmer - DF	4.50"	4.50"	3.82"	6569	8471	1105	15041	None
2 - Trimmer - DF	4.50"	4.50"	3.82"	6569	8471	1105	15041	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 1" o/c	
Bottom Edge (Lu)	7' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 7' 1"	N/A	24.5	--	--	
1 - Uniform (PSF)	0 to 7' 1"	12' 8 5/8"	100.0	120.0	20.0	
2 - Uniform (PSF)	0 to 7' 1"	2' 10 5/8"	100.0	300.0	20.0	
3 - Uniform (PLF)	0 to 7' 1"	N/A	270.0	-	-	

Weyerhaeuser Notes

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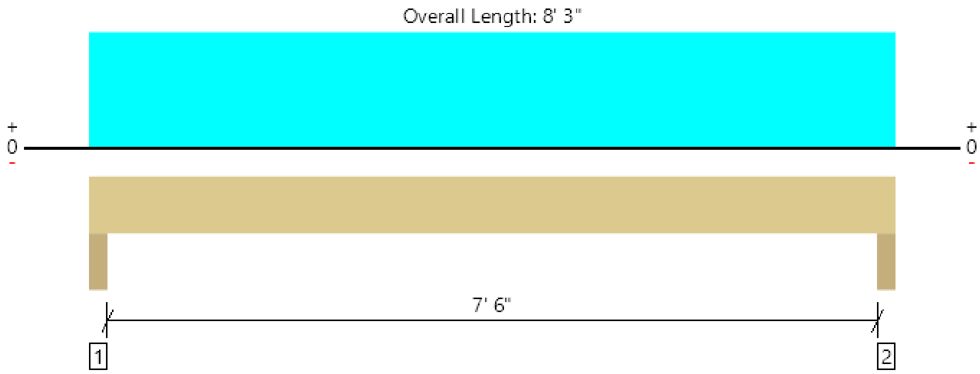
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Lobby to Corridor Window

3 piece(s) 1 3/4" x 16" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	17518 @ 3"	17719 (4.50")	Passed (99%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	10263 @ 1' 8 1/2"	15960	Passed (64%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	31884 @ 4' 1 1/2"	46671	Passed (68%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.079 @ 4' 1 1/2"	0.258	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.140 @ 4' 1 1/2"	0.155	Passed (L/665)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/600).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Trimmer - DF	4.50"	4.50"	4.45"	7652	9866	1287	17518	None
2 - Trimmer - DF	4.50"	4.50"	4.45"	7652	9866	1287	17518	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 3" o/c	
Bottom Edge (Lu)	8' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 8' 3"	N/A	24.5	--	--	
1 - Uniform (PSF)	0 to 8' 3"	12' 8 5/8"	100.0	120.0	20.0	Three Floors and Roof
2 - Uniform (PSF)	0 to 8' 3"	2' 10 5/8"	100.0	300.0	20.0	3 floors hallway
3 - Uniform (PLF)	0 to 8' 3"	N/A	270.0	-	-	

Weyerhaeuser Notes

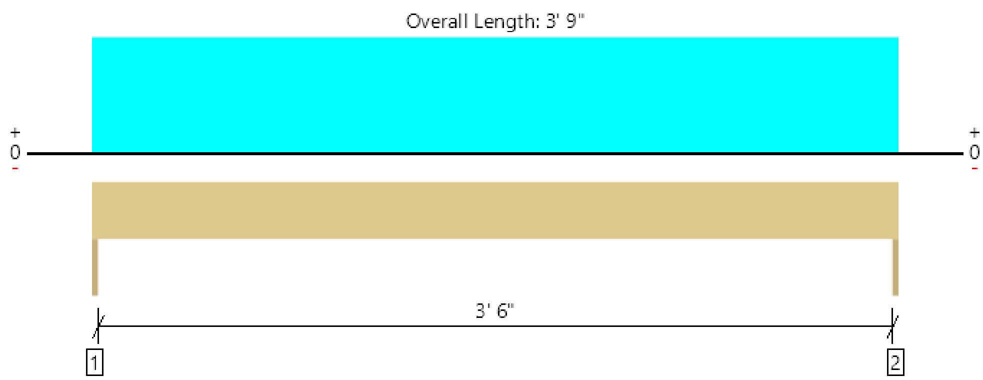
ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

MEMBER REPORT

PASSED

Level, Staff Laundry Door

3 piece(s) 2 x 8 DF No.1



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2307 @ 0	4219 (1.50")	Passed (55%)	--	1.0 D + 0.75 L + 0.75 Lr (All Spans)
Shear (lbs)	1370 @ 8 3/4"	3915	Passed (35%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2102 @ 1' 10 1/2"	3942	Passed (53%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.015 @ 1' 10 1/2"	0.125	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 Lr (All Spans)
Total Load Defl. (in)	0.023 @ 1' 10 1/2"	0.188	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 Lr (All Spans)

System : Wall
Member Type : Header
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Roof Live	Factored	
1 - Trimmer - DF	1.50"	1.50"	1.50"	747	1495	585	2307	None
2 - Trimmer - DF	1.50"	1.50"	1.50"	747	1495	585	2307	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 9" o/c	
Bottom Edge (Lu)	3' 9" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 3' 9"	N/A	8.3	--	--	
1 - Uniform (PSF)	0 to 3' 9"	12' 8 5/8"	25.0	40.0	20.0	Guest room load
2 - Uniform (PSF)	0 to 3' 9"	2' 10 5/8"	25.0	100.0	20.0	Corridor Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

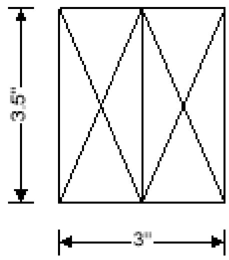
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression P = **6398** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b =$ **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **10.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **6.12** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **5.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **10.72** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.50**
Size factor for tension - Table 4A	C_{Ft} = **1.50**
Size factor for compression - Table 4A	C_{Fc} = **1.15**
Flat use factor - Table 4A	C_{fu} = **1.10**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.00**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Column stability coefficient – cl.15.3.2	K_f = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in²
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **828** lb/in²
	c = **0.80**

Column stability factor - eq.15.3-1

$$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}] = \mathbf{0.46}$$

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **1.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **709** lb/in²
Applied compressive stress	$f_c = P / A$ = **609** lb/in²
	f_c / F_c' = **0.860**

PASS - Design compressive stress exceeds applied compressive stress

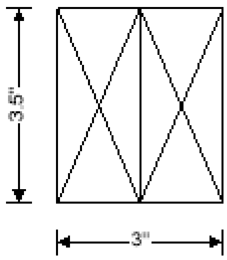
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression — P = **3199** lb



Sawn lumber section details

Nominal breadth of sections — b_{nom} = **2** in

Dressed breadth of sections — b = **1.5** in

Nominal depth of sections — d_{nom} = **4** in

Dressed depth of sections — d = **3.5** in

Number of sections in member — N = **2**

Overall breadth of member — $b_b = N \times b$ = **3** in

Species, grade and size classification — Douglas Fir-Larch, No.2 grade, 2" & wider

Bending parallel to grain — F_b = **900** lb/in^2

Tension parallel to grain — F_t = **575** lb/in^2

Compression parallel to grain — F_c = **1350** lb/in^2

Compression perpendicular to grain — F_{c_perp} = **625** lb/in^2

Shear parallel to grain — F_v = **180** lb/in^2

Modulus of elasticity — E = **1600000** lb/in^2

Modulus of elasticity, stability calculations — E_{min} = **580000** lb/in^2

Mean shear modulus — $G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition — **Dry**

Load duration — **Ten years**

Unbraced length in x-axis — L_x = **7** ft

Effective length factor in x-axis — K_x = **1**

Effective length in x-axis — $L_{ex} = L_x \times K_x$ = **7** ft

Unbraced length in y-axis — L_y = **1** ft

Effective length factor in y-axis — K_y = **1**

Effective length in y-axis — $L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member — $A = N \times b \times d$ = **10.50** in^2

Section modulus — $S_x = N \times b \times d^2 / 6$ = **6.12** in^3

$S_y = d \times (N \times b)^2 / 6$ = **5.25** in^3

Second moment of area — $I_x = N \times b \times d^3 / 12$ = **10.72** in^4

$I_y = d \times (N \times b)^3 / 12$ = **7.87** in^4

Adjustment factors

Load duration factor - Table 2.3.2 — C_D = **1.00**

Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.50**
Size factor for tension - Table 4A	C_{Ft} = **1.50**
Size factor for compression - Table 4A	C_{Fc} = **1.15**
Flat use factor - Table 4A	C_{fu} = **1.10**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.00**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Column stability coefficient – cl.15.3.2	K_f = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **828** lb/in^2
	c = **0.80**

Column stability factor - eq.15.3-1

$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.46**

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **1.00**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **709** lb/in^2
Applied compressive stress	$f_c = P / A$ = **305** lb/in^2
	f_c / F_c' = **0.430**

PASS - Design compressive stress exceeds applied compressive stress

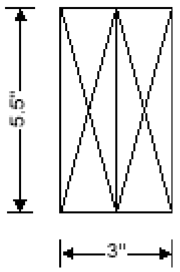
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression — P = **8438** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b =$ **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **16.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **15.12** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **8.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **41.59** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **12.37** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Column stability coefficient – cl.15.3.2 K_f = **0.60**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b_b)^2$ = **29797** lb/in^2

c = **0.80**

Column stability factor - eq.15.3-1

$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.59**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.50**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **882** lb/in^2

Applied compressive stress $f_c = P / A$ = **511** lb/in^2

f_c / F_c' = **0.580**

PASS - Design compressive stress exceeds applied compressive stress

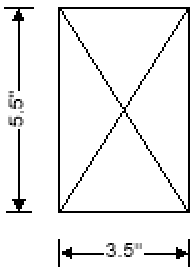
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression	P = **13781** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **4** in
Dressed breadth of sections	b = **3.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **3.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.72** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.72** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **19.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **17.65** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **11.23** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **48.53** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **19.65** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.30**
Size factor for tension - Table 4A	C_{Ft} = **1.30**
Size factor for compression - Table 4A	C_{Fc} = **1.10**
Flat use factor - Table 4A	C_{fu} = **1.05**
Incising factor for modulus of elasticity - Table 4.3.8	
	C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8	
	C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8	
	C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9	C_r = **1.00**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Adjusted modulus of elasticity for column stability	$E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression	$F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **1680** lb/in^2
	c = **0.80**

Column stability factor - eq.3.7-1

$$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} = \mathbf{0.73}$$

Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **1.50**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress	$F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1087** lb/in^2
Applied compressive stress	$f_c = P / A$ = **716** lb/in^2
	f_c / F_c' = **0.659**

PASS - Design compressive stress exceeds applied compressive stress

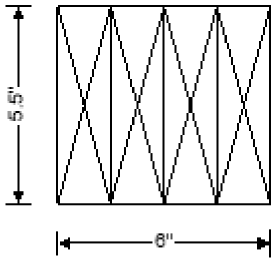
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression	P = **18543** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **4**
Overall breadth of member	$b_b = N \times b$ = **6** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **8.22** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **8.22** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **33.00** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **30.25** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **33.00** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **83.19** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **99.00** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**

Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for bending - Table 4A C_{Fb} = **1.30**
Size factor for tension - Table 4A C_{Ft} = **1.30**
Size factor for compression - Table 4A C_{Fc} = **1.10**
Flat use factor - Table 4A C_{fu} = **1.15**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compression - Table 4.3.8
C_i = **1.00**
Incising factor for perpendicular compression - Table 4.3.8
C_{ic_perp} = **1.00**
Repetitive member factor - cl.4.3.9 C_r = **1.15**
Bearing area factor - cl.3.10.4 C_b = **1.00**
Column stability coefficient – cl.15.3.2 K_f = **0.60**
Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2
Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in^2
Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b_b)^2$ = **119190** lb/in^2
c = **0.80**
Column stability factor - eq.15.3-1
$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.60**
Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **0.75**
 - Beam is fully restrained
Beam stability factor - cl.3.3.3 C_L = **1.00**

Strength in compression parallel to grain - cl.3.6.3
Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **889** lb/in^2
Applied compressive stress $f_c = P / A$ = **562** lb/in^2
f_c / F_c' = **0.632**
PASS - Design compressive stress exceeds applied compressive stress

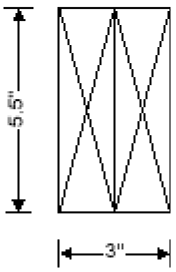
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression   P = **4028** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b =$ **3** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **16.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **15.12** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **8.25** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **41.59** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **12.37** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Column stability coefficient – cl.15.3.2 K_f = **0.60**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in²

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b_b)^2$ = **29797** lb/in²

c = **0.80**

Column stability factor - eq.15.3-1

$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.59**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.50**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **882** lb/in²

Applied compressive stress $f_c = P / A$ = **244** lb/in²

f_c / F_c' = **0.277**

PASS - Design compressive stress exceeds applied compressive stress

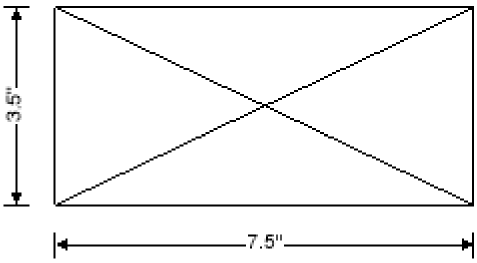
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression P = **12377** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **8** in
Dressed breadth of sections	b = **7.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **7.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	G_{def} = E / 16 = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.79** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.79** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **26.25** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **15.31** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **32.81** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **26.80** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **123.05** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.50**

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

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

Flat use factor - Table 4A C_{fu} = **1.00**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1552** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **668** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.38**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **0.50**

 - Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **595** lb/in^2

Applied compressive stress $f_c = P / A$ = **472** lb/in^2

f_c / F_c' = **0.793**

PASS - Design compressive stress exceeds applied compressive stress

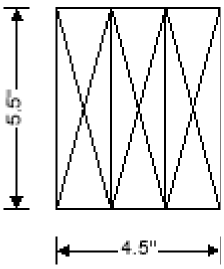
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression P = **11430** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Dressed depth of sections	d = **5.5** in
Number of sections in member	N = **3**
Overall breadth of member	$b_b = N \times b =$ **4.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16 =$ **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x =$ **7** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y =$ **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d =$ **24.75** in^2
Section modulus	$S_x = N \times b \times d^2 / 6 =$ **22.69** in^3
	$S_y = d \times (N \times b)^2 / 6 =$ **18.56** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12 =$ **62.39** in^4
	$I_y = d \times (N \times b)^3 / 12 =$ **41.77** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.30**

Size factor for compression - Table 4A C_{Fc} = **1.10**

Flat use factor - Table 4A C_{fu} = **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Column stability coefficient – cl.15.3.2 K_f = **0.60**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in²

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1485** lb/in²

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b_b)^2$ = **67044** lb/in²

c = **0.80**

Column stability factor - eq.15.3-1

$C_P = K_f \times [(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}]$ = **0.60**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **887** lb/in²

Applied compressive stress $f_c = P / A$ = **462** lb/in²

f_c / F_c' = **0.521**

PASS - Design compressive stress exceeds applied compressive stress

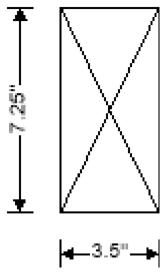
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression P = **15904** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **4** in
Dressed breadth of sections	b = **3.5** in
Nominal depth of sections	d_{nom} = **8** in
Dressed depth of sections	d = **7.25** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **3.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **25.37** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **30.66** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **14.80** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **111.15** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **25.90** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.20**

Size factor for compression - Table 4A C_{Fc} = **1.05**

Flat use factor - Table 4A C_{fu} = **1.05**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1418** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **3552** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.90**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1275** lb/in^2

Applied compressive stress $f_c = P / A$ = **627** lb/in^2

f_c / F_c' = **0.492**

PASS - Design compressive stress exceeds applied compressive stress

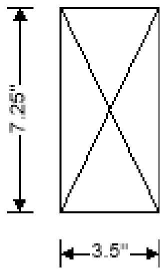
STRUCTURAL WOOD MEMBER DESIGN (NDS)

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

Tedds calculation version 1.7.10

Analysis results

Design axial compression | P = **18375** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **4** in
Dressed breadth of sections	b = **3.5** in
Nominal depth of sections	d_{nom} = **8** in
Dressed depth of sections	d = **7.25** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **3.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Load duration	**Ten years**
Unbraced length in x-axis	L_x = **7.79** ft
Effective length factor in x-axis	K_x = **1**
Effective length in x-axis	$L_{ex} = L_x \times K_x$ = **7.79** ft
Unbraced length in y-axis	L_y = **1** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	$L_{ey} = L_y \times K_y$ = **1** ft

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **25.37** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **30.66** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **14.80** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **111.15** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **25.90** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**

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

Size factor for bending - Table 4A C_{Fb} = **1.30**

Size factor for tension - Table 4A C_{Ft} = **1.20**

Size factor for compression - Table 4A C_{Fc} = **1.05**

Flat use factor - Table 4A C_{fu} = **1.05**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Adjusted modulus of elasticity for column stability $E_{min}' = E_{min} \times C_{ME} \times C_t \times C_{iE}$ = **580000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t \times C_{Fc} \times C_i$ = **1418** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{min}' / (L_{ex} / d)^2$ = **2868** lb/in^2

c = **0.80**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]}$ = **0.87**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **2.00**

- Beam is fully restrained

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

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_{Fc} \times C_i \times C_P$ = **1232** lb/in^2

Applied compressive stress $f_c = P / A$ = **724** lb/in^2

f_c / F_c' = **0.588**

PASS - Design compressive stress exceeds applied compressive stress

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JOB SUMMARY REPORT

Front Canopy_Imported

Level			
Member Name	Results	Current Solution	Comments
2x10	Passed	1 piece(s) 2 x 10 DF No.1 @ 24" OC	
2x10 Cant	Passed	1 piece(s) 2 x 10 DF No.1 @ 24" OC	
Canopy Girder 1	Passed	1 piece(s) 8 3/4" x 16 1/2" 24F-V4 DF Glulam	
Canopy Girder 2	Passed	1 piece(s) 8 3/4" x 16 1/2" 24F-V4 DF Glulam	

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

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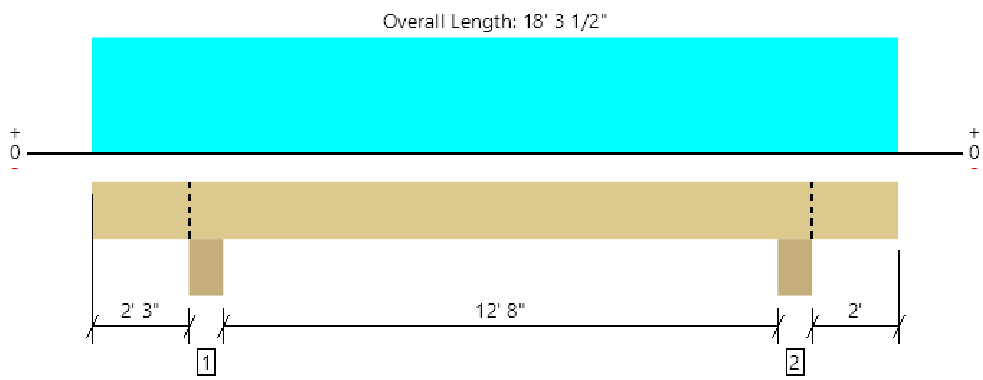
ForteWEB v3.6

File Name: Front Canopy_Imported

Level, 2x10

1 piece(s) 2 x 10 DF No.1 @ 24" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	843 @ 2' 7 1/8"	7734 (8.25")	Passed (11%)	--	1.0 D + 1.0 S (Adj Spans)
Shear (lbs)	509 @ 3' 8 1/2"	1915	Passed (27%)	1.15	1.0 D + 1.0 S (Adj Spans)
Moment (Ft-lbs)	1792 @ 9' 3 11/16"	2593	Passed (69%)	1.15	1.0 D + 1.0 S (Alt Spans)
Live Load Defl. (in)	0.156 @ 9' 3 3/8"	0.334	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.334 @ 9' 3 7/16"	0.668	Passed (L/480)	--	1.0 D + 1.0 S (Alt Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Beam - DF	8.25"	8.25"	1.50"	466	377	377	843	Blocking
2 - Beam - DF	8.25"	8.25"	1.50"	449	364	364	813	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	6' 10" o/c	
Bottom Edge (Lu)	18' 4" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 18' 3 1/2"	24"	25.0	20.0	20.0	Default Load

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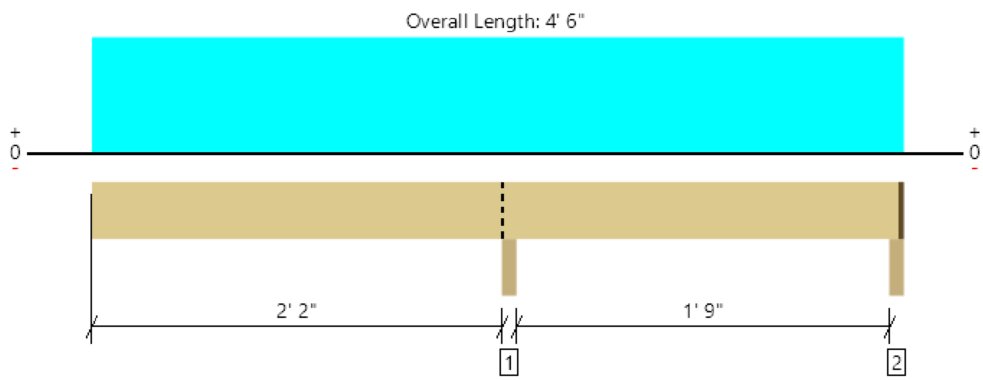
ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, 2x10 Cant

1 piece(s) 2 x 10 DF No.1 @ 24" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	419 @ 2' 3 3/4"	3281 (3.50")	Passed (13%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	128 @ 3' 2 3/4"	1915	Passed (7%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	-241 @ 2' 3 3/4"	2593	Passed (9%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.003 @ 0	0.200	Passed (2L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.007 @ 0	0.231	Passed (2L/999+)	--	1.0 D + 1.0 S (Alt Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Overhang deflection criteria: LL (0.2") and TL (2L/240).
- Left cantilever length exceeds 1/3 member length or 1/2 back span length. Additional bracing should be considered.
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usage.
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Beam - DF	3.50"	3.50"	1.50"	233	186	186	419	Blocking
2 - Beam - DF	3.50"	2.25"	1.50"	-8	21/-28	21/-28	13/-36	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 5" o/c	
Bottom Edge (Lu)	4' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 4' 6"	24"	25.0	20.0	20.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

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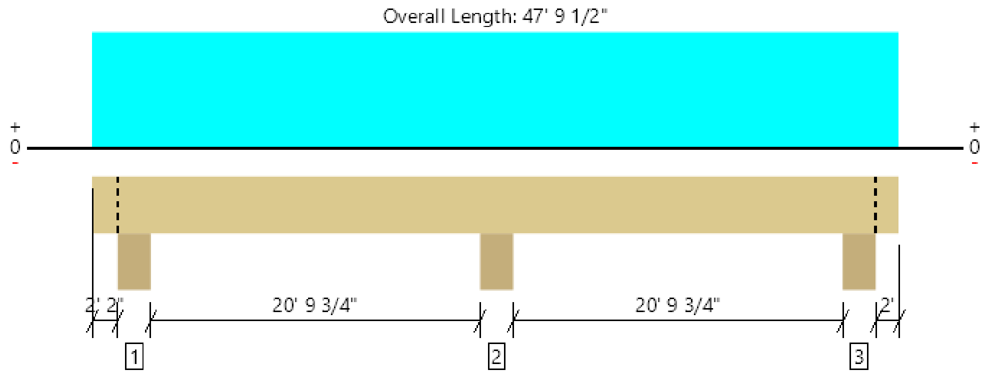
MEMBER REPORT

PASSED

Level, Canopy Girder 1

1 piece(s) 8 3/4" x 16 1/2" 24F-V4 DF Glulam



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	11076 @ 23' 11 3/4"	45500 (8.00")	Passed (24%)	--	1.0 D + 1.0 S (Adj Spans)
Shear (lbs)	4828 @ 25' 8 1/4"	29332	Passed (16%)	1.15	1.0 D + 1.0 S (Adj Spans)
Pos Moment (Ft-lbs)	14146 @ 36' 11 5/8"	85914	Passed (16%)	1.15	1.0 D + 1.0 S (Alt Spans)
Neg Moment (Ft-lbs)	-23596 @ 23' 11 3/4"	69163	Passed (34%)	1.15	1.0 D + 1.0 S (Adj Spans)
Live Load Defl. (in)	0.077 @ 35' 8 7/16"	0.537	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.157 @ 36' 7/16"	1.074	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 0.94 that was calculated using length L = 16' 5 9/16".
- Critical negative moment adjusted by a volume/size factor of 0.98 that was calculated using length L = 10' 8".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Column - SPF	8.00"	8.00"	1.50"	2666	1945	1945	4611	Blocking
2 - Column - SPF	8.00"	8.00"	1.95"	6546	4530	4530	11076	None
3 - Column - SPF	8.00"	8.00"	1.50"	2620	1913	1913	4533	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	47' 10" o/c	
Bottom Edge (Lu)	47' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 47' 9 1/2"	N/A	35.1	--	--	
1 - Uniform (PSF)	0 to 47' 9 1/2" (Front)	8' 6"	25.0	20.0	20.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

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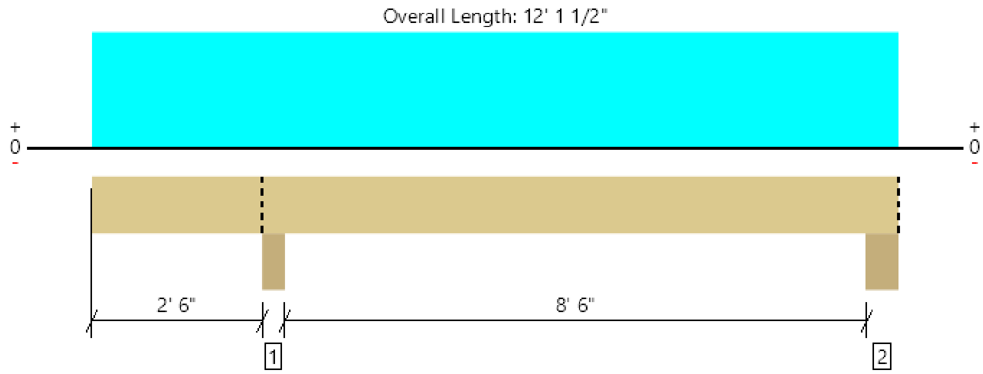
Level, Canopy Girder 2

1 piece(s) 8 3/4" x 16 1/2" 24F-V4 DF Glulam



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1630 @ 2' 8 3/4"	31281 (5.50")	Passed (5%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	698 @ 4' 4"	29332	Passed (2%)	1.15	1.0 D + 1.0 S (All Spans)
Pos Moment (Ft-lbs)	1794 @ 7' 6"	91317	Passed (2%)	1.15	1.0 D + 1.0 S (Alt Spans)
Neg Moment (Ft-lbs)	-801 @ 2' 8 3/4"	70390	Passed (1%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.002 @ 7' 2 11/16"	0.221	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)
Total Load Defl. (in)	0.004 @ 7' 3 5/16"	0.443	Passed (L/999+)	--	1.0 D + 1.0 S (Alt Spans)

System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2015
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Overhang deflection criteria: LL (2L/480) and TL (2L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 8' 2 1/16".
- Critical negative moment adjusted by a volume/size factor of 1.00 that was calculated using length L = 3' 6 7/8".
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Column - DF	5.50"	5.50"	1.50"	1024	606	606	1630	Blocking
2 - Column - DF	8.00"	8.00"	1.50"	614	381	381	995	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 2" o/c	
Bottom Edge (Lu)	12' 2" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 12' 1 1/2"	N/A	35.1	--	--	
1 - Uniform (PSF)	0 to 12' 1 1/2" (Top)	4'	25.0	20.0	20.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

STRUCTURAL GLUED LAMINATED TIMBER (GLULAM) MEMBER DESIGN (NDS)

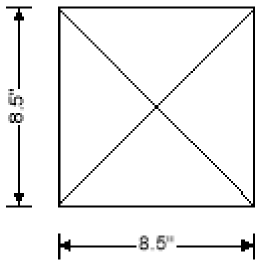
In accordance with the ANSI/AF&PA NDS-2018 using the ASD method

Tedds calculation version 1.7.10

Analysis results

Design moment in major axis M_x = **9450** lb_ft

Design axial compression P = **11076** lb



Glulam section details

Net finished breadth of sections b = **8.5** in

Net finished depth of sections d = **8.5** in

Number of sections in member N = **1**

Overall breadth of member $b_b = N \times b$ = **8.5** in

Alignment of laminations **Horizontal**

Stress class 24F-V4 DF/DF

Tension parallel to grain F_t = **1100** lb/in²

Compression parallel to grain F_c = **1650** lb/in²

Bending about X-X axis properties (loaded perpendicular to wide faces of laminations):

Positive bending F_{bx_pos} = **2400** lb/in²

Negative bending F_{bx_neg} = **1850** lb/in²

Compression perpendicular to grain F_{c_perp} = **650** lb/in²

Shear parallel to grain F_v = **265** lb/in²

Modulus of elasticity E = **1800000** lb/in²

Modulus of elasticity, stability calculations E_{min} = **950000** lb/in²

Mean shear modulus G_{def} = E / 16 = **112500** lb/in²

Bending about Y-Y axis properties (loaded parallel to wide faces of laminations):

Bending F_{by} = **1450** lb/in²

Modulus of elasticity stability calculations E_{ymin} = **850000** lb/in²

Member details

Service condition **Dry**

Load duration **Ten years**

Unbraced length in x-axis L_x = **8** ft

Effective length factor in x-axis K_x = **1**

Effective length in x-axis $L_{ex} = L_x \times K_x$ = **8** ft

Unbraced length in y-axis L_y = **8** ft

Effective length factor in y-axis K_y = **1**

Effective length in y-axis $L_{ey} = L_y \times K_y$ = **8** ft

Section properties

Cross sectional area of member $A = N \times b \times d$ = **72.25** in²

Section modulus $S_x = N \times b \times d^2 / 6$ = **102.35** in³

$S_y = d \times (N \times b)^2 / 6 =$ **102.35** in^3

Second moment of area $I_x = N \times b \times d^3 / 12 =$ **435.01** in^4

$I_y = d \times (N \times b)^3 / 12 =$ **435.01** in^4

Adjustment factors

Load duration factor - Table 2.3.2 $C_D =$ **1.00**

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

Flat use factor - Table 5A $C_{fu} =$ **1.04**

Bearing area factor - cl.3.10.4 $C_b =$ **1.00**

Length of beam between points of zero moment $L_0 =$ **21** ft

For species other than Southern Pine x = **10**

Volume factor - eq.5.3-1 $C_V = \min((21\text{ ft} / L_0)^{1/x} \times (12\text{ in} / d)^{1/x} \times (5.125\text{ in} / b)^{1/x}, 1) =$ **0.98**

Adjusted modulus of elasticity for column stability $E_{ymin}' = E_{ymin} \times C_{ME} \times C_t =$ **850000** lb/in^2

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_t =$ **1650** lb/in^2

Critical buckling design value for compression $F_{cE} = 0.822 \times E_{ymin}' / (L_{ey} / b)^2 =$ **5478** lb/in^2

c = **0.90**

Column stability factor - eq.3.7-1

$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[((1 + (F_{cE} / F_c^*)) / (2 \times c))^2 - (F_{cE} / F_c^*) / c]} =$ **0.96**

Depth-to-breadth ratio $d / (N \times b) =$ **1.00**

- Beam is fully restrained

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

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_{bx\_pos} \times C_D \times C_t \times \min(C_L, C_V) \times C_c =$ **2362** lb/in^2

Actual bending stress $f_b = M_x / S_x =$ **1108** lb/in^2

$f_b / F_b' =$ **0.469**

PASS - Design bending stress exceeds actual bending stress

Strength in compression parallel to grain - cl.3.6.3

Design compressive stress $F_c' = F_c \times C_D \times C_t \times C_P =$ **1585** lb/in^2

Applied compressive stress $f_c = P / A =$ **153** lb/in^2

$f_c / F_c' =$ **0.097**

PASS - Design compressive stress exceeds applied compressive stress

Bending and axial compression - cl.3.9.2

Critical buckling design value about x-x axis $F_{cE1} = 0.822 \times E_{min}' / (L_{ex} / d)^2 =$ **6122** lb/in^2

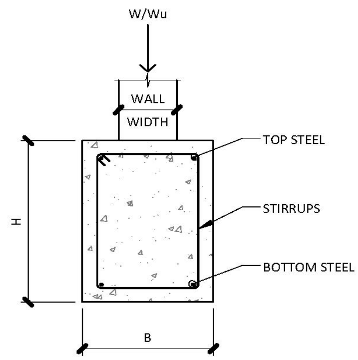
Bending and compression check - eq.3.9-3 $[f_c / F_c']^2 + f_{b1} / (F_{b1}' \times [1 - (f_c / F_{cE1})]) =$ **0.491 < 1**

PASS - Combined compressive and bending stresses are within permissible limits

FOUNDATIONS

Grade Beam Design

Grade Beam Location: Exterior Grade Beam



General Information:

Footing Width, B =	18	in	
Footing Depth, H =	36	in	
Steel Depth, d =	31.875	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	5.5	in	
Soil Bearing Pressure =	4	ksf	
Allowable or Effective SBC?	Allowable		
Footing Concrete Strength =	3.5	ksi	
Wall Concrete Strength =	4	ksi	

Loading:

Vertical Loads:

Applied Dead Load =	2.26	klf	
Footing Weight =	0.675	klf	
Applied Live Load =	1.55	klf	
ASD Total Load, W =	4.48	klf	
LRFD Total Load, Wu =	6.00	klf	

LRFD Factors: (ASCE 7 Combo)

Dead =	1.2
Live =	1.6

ASD Soil Pressures:

Required Footing Width =	1.120365625	ft	(Footing Width = W / Soil Bearing Pressure)
Actual Soil Bearing Pressure =	2987.641667	psf	(Actual Soil Bearing Pressure = W / Footing Width)
Chosen Footing Width =	1.5	ft	
Assumed Footing Span =	4	ft	
Is Footing Width Adequate?	**YES**		

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFD Bearing Pressure =	3998.61	psf	
h=	34	in	(h = H - 2 in.)
Cantilever =	6.25	in	(Cantilever = B/2 - Wall Width/2)
Vu1 =	2.08	k/ft	(LRFD Bearing Pressure* Cantilever)
ΦVn =	25.80	k/ft	(ACI 318-11 Equation 22-9, ΦVn = 0.75*(4/3)*sqrt(f'c)*
Adequate in One-Way Shear?	**YES**		

Plain Concrete Flexure Check: Cantilevered Side of Footing

h=	34	in	(h = H - 2 in.)
Cantilever =	6.25	in	(Cantilever = B/2 - Wall Width/2)
Mu =	0.54	k-ft/ft	(Actual Soil Bearing Pressure* Cantilever)
S=	3468.00	in^3	(S = 12*h^2 / 4)
ΦMn =	76.94	k-ft/ft	(ACI 318-11 Equation 22-2, ΦMn = 0.9*5*sqrt(f'c)*S / 1
Adequate in Flexure?	**YES**		

One-Way Shear Check: For Spanning "X" Distance Listed

Vu1 =	12.00	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	50.92	klf	(ACI 318-11 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-11 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Oth

Use #3 Stirrups at 18 in. O.C. (Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	145.86	klf	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall
Adequate in Bearing?	**YES**		

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

Mu =	12.00	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	20.168		(m = fy/(0.85*f'c))
Ru =	0.009	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0001		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030		(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 2
4/3*Mu ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0002		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Re
A's Required =	0.111	in^2	(As = Governing ρ*B*d)
Bar # =	6		
Number of Bars =	5	bars	
As Provided =	2.20	in^2	
Is Steel Adequate ?	**YES**		

Top Steel:

Bar # =	6	
Number of Bars =	5	bars
As Provided =	2.20	in^2

Temperature & Shrinkage Steel:

Minimum Steel =	1.1664	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 in
As Provided Top =	2.20	in^2/ft	
As Provided Bott =	2.20	in^2/ft	
As Provided Total =	4.40	in^2/ft	
T&S Steel Provided?	**YES**		

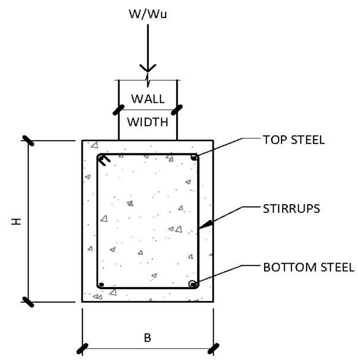
Final Footing Design:

Footing Width, B =	**18**	in
Footing Depth, H =	**36**	in

Top Steel = **(5) #6 bars**
Bottom Steel = **(5) #6 bars**
Stirrups = **#3 Stirrups at 18 in. O.C.**

Grade Beam Design

Grade Beam Location: Corridor Thickened Slab



General Information:

Footing Width, B =	18	in	
Footing Depth, H =	16	in	
Steel Depth, d =	12.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	5.5	in	
Soil Bearing Pressure =	4	ksf	
Allowable or Effective SBC?	Effective		
Footing Concrete Strength =	3.5	ksi	
Wall Concrete Strength =	4	ksi	

Loading:

Vertical Loads:			LRFD Factors: (ASCE 7 Combo)
Applied Dead Load =	1.98	klf	
Footing Weight =	0.3	klf	Dead = 1.2
Applied Live Load =	2.39	klf	Live = 1.6
ASD Total Load, W =	4.37	klf	
LRFD Total Load, Wu =	6.20	klf	

ASD Soil Pressures:

Required Footing Width =	1.092396875	ft	(Footing Width = W / Soil Bearing Pressure)
Actual Soil Bearing Pressure =	2913.058333	psf	(Actual Soil Bearing Pressure = W / Footing Width)
Chosen Footing Width =	1.5	ft	
Assumed Footing Span =	3	ft	
Is Footing Width Adequate?	**YES**		

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFD Bearing Pressure =	4134.11	psf	
h=	14	in	(h = H - 2 in.)
Cantilever =	6.25	in	(Cantilever = B/2 - Wall Width/2)
Vu1 =	2.15	k/ft	(LRFD Bearing Pressure* Cantilever)
ΦVn =	10.63	k/ft	(ACI 318-11 Equation 22-9, ΦVn = 0.75*(4/3)*sqrt(f'c)*
Adequate in One-Way Shear?	**YES**		

Plain Concrete Flexure Check: Cantilevered Side of Footing

h=	14	in	(h = H - 2 in.)
Cantilever =	6.25	in	(Cantilever = B/2 - Wall Width/2)
Mu =	0.56	k-ft/ft	(Actual Soil Bearing Pressure* Cantilever)
S=	588.00	in^3	(S = 12*h^2 / 4)
ΦMn =	13.04	k-ft/ft	(ACI 318-11 Equation 22-2, ΦMn = 0.9*5*sqrt(f'c)*S / 1
Adequate in Flexure?	**YES**		

One-Way Shear Check: For Spanning "X" Distance Listed

Vu1 =	9.30	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	19.27	klf	(ACI 318-11 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-11 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Oth
	Use #3 Stirrups at 18 in. O.C.		(Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	145.86	klf	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall
Adequate in Bearing?	**YES**		

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

Mu =	6.98	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	20.168		(m = fy/(0.85*f'c))
Ru =	0.036	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0006		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030		(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 2
4/3*Mu ρ Req'd =	0.0008		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0008		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Re
A's Required =	0.172	in^2	(As = Governing ρ*B*d)
Bar # =	5		
Number of Bars =	3	bars	
As Provided =	0.93	in^2	
Is Steel Adequate ?	**YES**		

Top Steel:

Bar # =	5	
Number of Bars =	3	bars
As Provided =	0.93	in^2

Temperature & Shrinkage Steel:

Minimum Steel =	0.5184	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 in
As Provided Top =	0.93	in^2/ft	
As Provided Bott =	0.93	in^2/ft	
As Provided Total =	1.86	in^2/ft	
T&S Steel Provided?	**YES**		

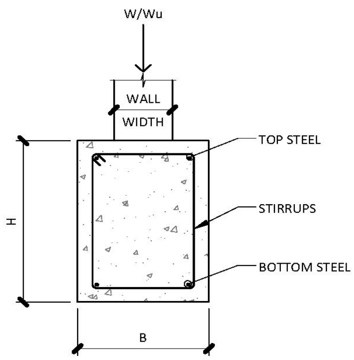
Final Footing Design:

Footing Width, B =	**18**	in
Footing Depth, H =	**16**	in

Top Steel = **(3) #5 bars**
Bottom Steel = **(3) #5 bars**
Stirrups = **#3 Stirrups at 18 in. O.C.**

Grade Beam Design

Grade Beam Location: Shear Wall Thickened Slab



General Information:

Footing Width, B =	18	in	
Footing Depth, H =	16	in	
Steel Depth, d =	12.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	5.5	in	
Soil Bearing Pressure =	4	ksf	
Allowable or Effective SBC?	Effective		
Footing Concrete Strength =	3.5	ksi	
Wall Concrete Strength =	4	ksi	

Loading:

Vertical Loads:			LRFD Factors:	(ASCE 7 Combo)
Applied Dead Load =	0.60	klf		
Footing Weight =	0.3	klf	Dead =	1.2
Applied Live Load =	0.42	klf	Live =	1.6
ASD Total Load, W =	1.02	klf		
LRFD Total Load, Wu =	1.39	klf		

ASD Soil Pressures:

Required Footing Width =	0.255546875	ft	(Footing Width = W / Soil Bearing Pressure)
Actual Soil Bearing Pressure =	681.4583333	psf	(Actual Soil Bearing Pressure = W / Footing Width)
Chosen Footing Width =	1.5	ft	
Assumed Footing Span =	4	ft	
Is Footing Width Adequate?	**YES**		

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFD Bearing Pressure =	929.75	psf	
h=	14	in	(h = H - 2 in.)
Cantilever =	6.25	in	(Cantilever = B/2 - Wall Width/2)
Vu1 =	0.48	k/ft	(LRFD Bearing Pressure* Cantilever)
ΦVn =	10.63	k/ft	(ACI 318-11 Equation 22-9, ΦVn = 0.75*(4/3)*sqrt(f'c)*
Adequate in One-Way Shear?	**YES**		

Plain Concrete Flexure Check: Cantilevered Side of Footing

h=	14	in	(h = H - 2 in.)
Cantilever =	6.25	in	(Cantilever = B/2 - Wall Width/2)
Mu =	0.13	k-ft/ft	(Actual Soil Bearing Pressure* Cantilever)
S=	588.00	in^3	(S = 12*h^2 / 4)
ΦMn =	13.04	k-ft/ft	(ACI 318-11 Equation 22-2, ΦMn = 0.9*5*sqrt(f'c)*S / 1
Adequate in Flexure?	**YES**		

One-Way Shear Check: For Spanning "X" Distance Listed

Vu1 =	2.79	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	19.27	klf	(ACI 318-11 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-11 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Oth
	Use #3 Stirrups at 18 in. O.C.		(Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	145.86	klf	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall
Adequate in Bearing?	**YES**		

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

Mu =	2.79	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	20.168		(m = fy/(0.85*f'c))
Ru =	0.014	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030		(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 2
4/3*Mu ρ Req'd =	0.0003		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0003		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Re
A's Required =	0.069	in^2	(As = Governing ρ*B*d)
Bar # =	5		
Number of Bars =	3	bars	
As Provided =	0.93	in^2	
Is Steel Adequate ?	**YES**		

Top Steel:

Bar # =	5	
Number of Bars =	3	bars
As Provided =	0.93	in^2

Temperature & Shrinkage Steel:

Minimum Steel =	0.5184	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 in
As Provided Top =	0.93	in^2/ft	
As Provided Bott =	0.93	in^2/ft	
As Provided Total =	1.86	in^2/ft	
T&S Steel Provided?	**YES**		

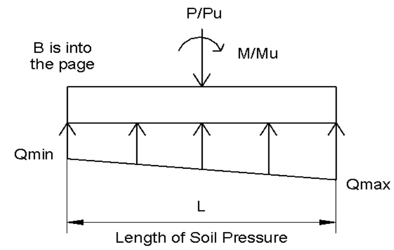
Final Footing Design:

Footing Width, B =	**18**	in
Footing Depth, H =	**16**	in

Top Steel = **(3) #5 bars**
Bottom Steel = **(3) #5 bars**
Stirrups = **#3 Stirrups at 18 in. O.C.**

Footing Designation: F1

Footing Location: Lobby



General Information:

Footing Length, L =	3	ft			
Footing Width, B =	3	ft			
Footing Depth, H =	36	in			
Location =	Edge	in			
Steel Depth, d =	32.0625	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in			
Slab Depth Above Footing =	8	in			
Area of Footing =	9	ft^2			(B*L)
Soil Bearing Pressure =	4	ksf			
Allowable or Effective SBC?	Allowable				
Concrete Strength =	3.5	ksi			
	B Direction			L Direction	
Column Size =	5.50 in		X	5.50 in	
Base Plate Size =	11.00 in		X	11.00 in	
Critical Section =	8.25 in		X	8.25 in	

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	9.4625	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	4.5	k	Live =	1.6	
Applied Live Load =	11.925	k	Uplift=	1.67	
ASD Total Load, P =	25.8875	k			
LRFD Total Load, Pu =	35.835	k	***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accor**		
ASD Uplift Load =	0	k			
LRFD Uplift Load =	0	k			

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.500	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	2.876	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	2.876	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(Mu / Pu)
Kern =	0.500	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	3.982	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	3.982	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	3.982	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	-0.180	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	-2.15	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	102.43	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	40.31	in	(Critical Section B + d)
b2 =	40.31	in	(Column Height L + d)
b0 =	161.25	in	(2*b1 + 2*b2)
Vu2 =	-9.10	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	30		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1376.40	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1827.19	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	917.60	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	467.9675	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	0	k	(From Above)
Required Dead Load =	0.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	13.9625	k	
Additional Slab Used =	0	ft	(Length of Additional Slab Past Edge of Footing in <u>Each Direction</u>
Additional Slab Area =	0	ft^2	From <u>Each Edge of Footing</u>)
Additional Slab Weight =	0	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	13.96	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.00	k-ft / ft		(Mu = (Pu/A)*0.5*Crit. L^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.000	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	6	in		
As Provided =	0.62	in^2/ft	=	7 Bars in B Direction
			=	7 Bars in L Direction

Bottom Steel:

Mu =	0.06	k-ft / ft		(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.000	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.001	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	6	in		
As Provided =	0.62	in^2/ft	=	7 Bars in B Direction
			=	7 Bars in L Direction

Temperature & Shrinkage Steel:

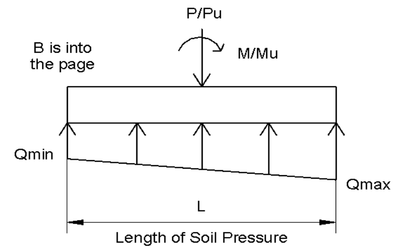
Minimum Steel =	0.7128	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.62	in^2/ft	
As Provided Bott =	0.62	in^2/ft	
As Provided Total =	1.24	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

Footing Width, B =	**3**	ft
Footing Length L =	**3**	ft
Footing Depth, H =	**36**	in
Top Steel =	**#5 bars**	**@6 inches O.C.**
Bottom Steel =	**#5 bars**	**@6 inches O.C.**

Footing Designation: F2

Footing Location: Post



General Information:

Footing Length, L =	3	ft			
Footing Width, B =	3	ft			
Footing Depth, H =	36	in			
Location =	Edge	in			
Steel Depth, d =	32.0625	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in			
Slab Depth Above Footing =	8	in			
Area of Footing =	9	ft^2			(B*L)
Soil Bearing Pressure =	4	ksf			
Allowable or Effective SBC?	Allowable				
Concrete Strength =	3.5	ksi			
	B Direction			L Direction	
Column Size =	5.50 in		X	5.50 in	
Base Plate Size =	11.00 in		X	11.00 in	
Critical Section =	8.25 in		X	8.25 in	

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	3.375	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	4.5	k	Live =	1.6	
Applied Live Load =	2.7	k	Uplift=	1.67	
ASD Total Load, P =	10.575	k			
LRFD Total Load, Pu =	13.77	k			
ASD Uplift Load =	2.0231	k			
LRFD Uplift Load =	3.371833333	k			

***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accor**

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.500	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.175	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.175	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(Mu / Pu)
Kern =	0.500	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.530	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.530	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.530	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	-0.180	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	-0.82	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	102.43	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	40.31	in	(Critical Section B + d)
b2 =	40.31	in	(Column Height L + d)
b0 =	161.25	in	(2*b1 + 2*b2)
Vu2 =	-3.50	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	30		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1376.40	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1827.19	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	917.60	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	467.9675	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	2.0231	k	(From Above)
Required Dead Load =	3.37	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	7.875	k	
Additional Slab Used =	0	ft	(Length of Additional Slab Past Edge of Footing in Each Direction From Each Edge of Footing)
Additional Slab Area =	0	ft^2	
Additional Slab Weight =	0	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	7.88	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.01	k-ft / ft		(Mu = (Pu/A)*0.5*Crit. L^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.000	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	6	in		
As Provided =	0.62	in^2/ft	=	7 Bars in B Direction
			=	7 Bars in L Direction

Bottom Steel:

Mu =	0.02	k-ft / ft		(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.000	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	6	in		
As Provided =	0.62	in^2/ft	=	7 Bars in B Direction
			=	7 Bars in L Direction

Temperature & Shrinkage Steel:

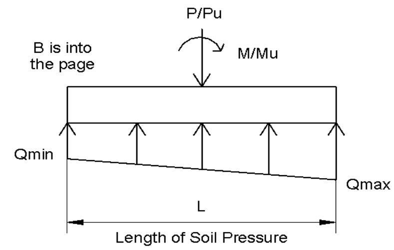
Minimum Steel =	0.7128	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.62	in^2/ft	
As Provided Bott =	0.62	in^2/ft	
As Provided Total =	1.24	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

Footing Width, B =	**3**	ft
Footing Length L =	**3**	ft
Footing Depth, H =	**36**	in
Top Steel =	**#5 bars**	**@6 inches O.C.**
Bottom Steel =	**#5 bars**	**@6 inches O.C.**

Footing Designation: F3

Footing Location: Shear Wall Footing at Grid J and Elevator Header



General Information:

Footing Length, L =	3	ft	
Footing Width, B =	3	ft	
Footing Depth, H =	16	in	
Location =	Edge	in	
Steel Depth, d =	12.0625	in	(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in	
Slab Depth Above Footing =	8	in	
Area of Footing =	9	ft^2	(B*L)
Soil Bearing Pressure =	4	ksf	
Allowable or Effective SBC?	Allowable		
Concrete Strength =	3.5	ksi	

	B Direction		L Direction
Column Size =	5.50 in	X	3.50 in
Base Plate Size =	11.00 in	X	11.00 in
Critical Section =	8.25 in	X	7.25 in

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	12.415	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	2.25	k	Live =	1.60	
Applied Wind/Seismic Load =	11.183	k	Uplift=	1.67	
ASD Total Load, P =	25.848	k			
LRFD Total Load, Pu =	35.4908	k			
ASD Uplift Load =	8.751	k			
LRFD Uplift Load =	14.585	k			

***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accor**

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.500	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	2.872	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	2.872	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(Mu / Pu)
Kern =	0.500	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	3.943	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	3.943	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	3.943	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	0.695	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	8.23	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	38.54	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	20.31	in	(Critical Section B + d)
b2 =	19.31	in	(Column Height L + d)
b0 =	79.25	in	(2*b1 + 2*b2)
Vu2 =	24.75	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	30		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	254.50	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	278.52	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	169.66	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	467.9675	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	8.751	k	(From Above)
Required Dead Load =	14.59	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	14.665	k	
Additional Slab Used =	0	ft	(Length of Additional Slab Past Edge of Footing in Each Direction
Additional Slab Area =	0	ft^2	From Each Edge of Footing)
Additional Slab Weight =	0	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	14.67	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.39	k-ft / ft		(Mu = (Pu/A)*0.5*Crit. L^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.003	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.010	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in^2/ft	=	4 Bars in B Direction
			=	4 Bars in L Direction

Bottom Steel:

Mu =	0.95	k-ft / ft		(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.007	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0001			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0002			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0002			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.023	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in^2/ft	=	4 Bars in B Direction
			=	4 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.2808	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

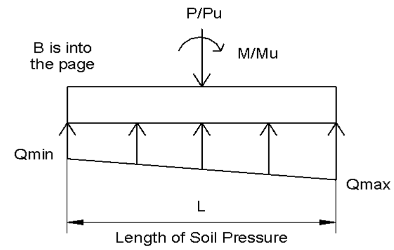
Final Footing Design:

Footing Width, B =	**3**	ft
Footing Length L =	**3**	ft
Footing Depth, H =	**16**	in

Top Steel =	**#5 bars**	**@12 inches O.C.**
Bottom Steel =	**#5 bars**	**@12 inches O.C.**

Footing Designation: F4

Footing Location: Shear Wall Footing Typical



General Information:

Footing Length, L =	5	ft			
Footing Width, B =	1.5	ft			
Footing Depth, H =	36	in			
Location =	Edge	in			
Steel Depth, d =	31.875	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in			
Slab Depth Above Footing =	8	in			
Area of Footing =	7.5	ft^2			(B*L)
Soil Bearing Pressure =	4	ksf			
Allowable or Effective SBC?	Allowable				
Concrete Strength =	3.5	ksi			
	B Direction			L Direction	
Column Size =	5.50 in		X	5.50 in	
Base Plate Size =	11.00 in		X	11.00 in	
Critical Section =	8.25 in		X	8.25 in	

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	6.059	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	6.75	k	Live =	1.67	
Applied Wind/Seismic Load =	15.589	k	Uplift=	1.67	
ASD Total Load, P =	28.398	k			
LRFD Total Load, Pu =	41.35246667	k			
ASD Uplift Load =	9353.4	k			
LRFD Uplift Load =	15589	k			

***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accor**

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.833	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	3.786	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	3.786	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(Mu / Pu)
Kern =	0.833	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	5.514	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	5.514	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	5.514	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	0.828	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	6.85	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	50.92	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	40.13	in	(Critical Section B + d)
b2 =	40.13	in	(Column Height L + d)
b0 =	160.50	in	(2*b1 + 2*b2)
Vu2 =	-20.29	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	30		(ACI 318-11 Section 11.11.2.1)
β =	3		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	726.39	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1806.43	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	907.99	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	467.9675	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	9353.4	k	(From Above)
Required Dead Load =	15589.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	9.809	k	
Additional Slab Used =	5	ft	(Length of Additional Slab Past Edge of Footing in Each Direction
Additional Slab Area =	82.5	ft^2	From Each Edge of Footing)
Additional Slab Weight =	4.125	k	
Wall Weight Over Footing =	2	klf	
Applied Wall Load on Footing =	3	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	5	ft	(B or L Depending on the Case)
Area of Cont. Footing =	4.5	ft^2	
Length of Cont. Footing/Wall Used =	5	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	13.375	k	
Total Dead Load =	30.31	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**More Dead Load is Req'd**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	712.72	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	20.168		(m = fy/(0.85*f'c))
Ru =	0.779	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0154		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030		(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0223		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0154		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	5.881	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	6		
Bar Spacing =	6	in	
As Provided =	0.88	in^2/ft	= 4 Bars in B Direction
			= 11 Bars in L Direction

Bottom Steel:

Mu =	1.89	k-ft / ft	(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	20.168		(m = fy/(0.85*f'c))
Ru =	0.002	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030		(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.018	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	6		
Bar Spacing =	6	in	
As Provided =	0.88	in^2/ft	= 4 Bars in B Direction
			= 11 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.7128	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.88	in^2/ft	
As Provided Bott =	0.88	in^2/ft	
As Provided Total =	1.76	in^2/ft	
T&S Steel Provided?	**YES**		

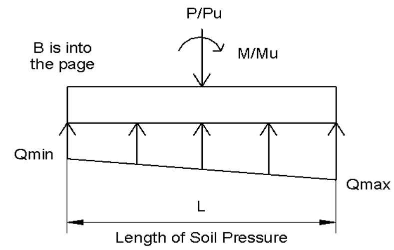
Final Footing Design:

Footing Width, B =	**1.5**	ft
Footing Length L =	**5**	ft
Footing Depth, H =	**36**	in

Top Steel =	**#6 bars**	**@6 inches O.C.**
Bottom Steel =	**#6 bars**	**@6 inches O.C.**

Footing Designation: F4

Footing Location: Shear Wall Footing Typical



General Information:

Footing Length, L =	5	ft			
Footing Width, B =	1.5	ft			
Footing Depth, H =	16	in			
Location =	Interior	in			
Steel Depth, d =	12.0625	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in			
Slab Depth Above Footing =	8	in			
Area of Footing =	7.5	ft^2			(B*L)
Soil Bearing Pressure =	4	ksf			
Allowable or Effective SBC?	Allowable				
Concrete Strength =	3.5	ksi			
	B Direction			L Direction	
Column Size =	5.50 in		X	5.50 in	
Base Plate Size =	11.00 in		X	11.00 in	
Critical Section =	8.25 in		X	8.25 in	

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	6.059	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	2.25	k	Live =	1.67	
Applied Wind/Seismic Load =	9.3534	k	Uplift=	1.67	
ASD Total Load, P =	17.6624	k			
LRFD Total Load, Pu =	25.5598	k			
ASD Uplift Load =	9.3534	k			
LRFD Uplift Load =	15.589	k			

***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accor**

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.833	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	2.355	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	2.355	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(Mu / Pu)
Kern =	0.833	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	3.408	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	3.408	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	3.408	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.654	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	8.45	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	19.27	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	20.31	in	(Critical Section B + d)
b2 =	20.31	in	(Column Height L + d)
b0 =	81.25	in	(2*b1 + 2*b2)
Vu2 =	15.80	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-11 Section 11.11.2.1)
β =	3		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	139.16	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	345.22	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	173.95	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	467.9675	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	9.3534	k	(From Above)
Required Dead Load =	15.59	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	8.309	k	
Additional Slab Used =	5	ft	(Length of Additional Slab Past Edge of Footing in Each Direction
Additional Slab Area =	165	ft^2	From Each Edge of Footing)
Additional Slab Weight =	8.25	k	
Wall Weight Over Footing =	2	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	5	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	5	ft	(B or L Depending on the Case)
Area of Cont. Footing =	2	ft^2	
Length of Cont. Footing/Wall Used =	2	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	4.6	k	
Total Dead Load =	21.16	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	2.84	k-ft / ft		(Mu = (Pu/A)*0.5*Crit. L^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.022	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0004			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0005			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0005			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.070	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in^2/ft	=	3 Bars in B Direction
			=	6 Bars in L Direction

Bottom Steel:

Mu =	4.66	k-ft / ft		(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	20.168			(m = fy/(0.85*f'c))
Ru =	0.036	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0006			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0030			(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0008			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0008			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.115	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in^2/ft	=	3 Bars in B Direction
			=	6 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.2808	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

Footing Width, B =	**1.5**	ft
Footing Length L =	**5**	ft
Footing Depth, H =	**16**	in

Top Steel =	**#5 bars**	**@12 inches O.C.**
Bottom Steel =	**#5 bars**	**@12 inches O.C.**

Sill Plate Anchor Design

	LRFD	ASD
Wind Load Factor:	1	0.6
Seismic Load Factor:	1	0.7

Overstrength, Ω = 1 (1 indicates not required)

Anchor	Max Wind (lb/ft_{sw})	Max Seismic (lb/ft_{sw})	Corr. Co	Anchor Spa. (in)	Shear per Anchor (lb) Capacity	Shear per Anchor (lb) Demand
	299.61	64.36				
5/8"Ø SCREW ANCHOR w/ 3-1/4" EMBED	300	64	1.00	32	3800.00	798.95
					PASS	

Sill Plate Screw Design

4th Floor

Anchor	Max Wind (lb/ft_{sw})	Max Seismic (lb/ft_{sw})	Corr. Co	Anchor Spa. (in)	Shear per Anchor (lb) Capacity	Shear per Anchor (lb) Demand	Anchor	Max Wind (lb/ft_{sw})	Max Seismic (lb/ft_{sw})	Corr. Co	Anchor Spa. (in)	Shear per Anchor (lb) Capacity	Shear per Anchor (lb) Demand
	168.8	26.5						168.8	26.5	1.00			
SDS25500	168.8	26.5	1.00	24	560.00	337.58	*5/8"Ø THRU BOLT*	168.8	26.5	1.00	48	960.00	675.17
					PASS							PASS	

3rd Floor

Anchor	Max Wind (lb/ft_{sw})	Max Seismic (lb/ft_{sw})	Corr. Co	Anchor Spa. (in)	Shear per Anchor (lb) Capacity	Shear per Anchor (lb) Demand	Anchor	Max Wind (lb/ft_{sw})	Max Seismic (lb/ft_{sw})	Corr. Co	Anchor Spa. (in)	Shear per Anchor (lb) Capacity	Shear per Anchor (lb) Demand
	221.1	46.5						221.1	46.5	1.00			
SDS25500	221.1	46.5	1.00	24	560.00	442.24	*5/8"Ø THRU BOLT*	221.1	46.5	1.00	48	960.00	884.47
					PASS							PASS	

2nd Floor

Anchor	Max Wind (lb/ft_{sw})	Max Seismic (lb/ft_{sw})	Corr. Co	Anchor Spa. (in)	Shear per Anchor (lb) Capacity	Shear per Anchor (lb) Demand	Anchor	Max Wind (lb/ft_{sw})	Max Seismic (lb/ft_{sw})	Corr. Co	Anchor Spa. (in)	Shear per Anchor (lb) Capacity	Shear per Anchor (lb) Demand
	273.4	59.4						273.4	59.4	1.00000			
SDS25500	273.4	59.4	1.00	24	560.00	546.89	*5/8"Ø THRU BOLT*	273.4	59.4	1.00000	32	960.00	729.18
					PASS							PASS	

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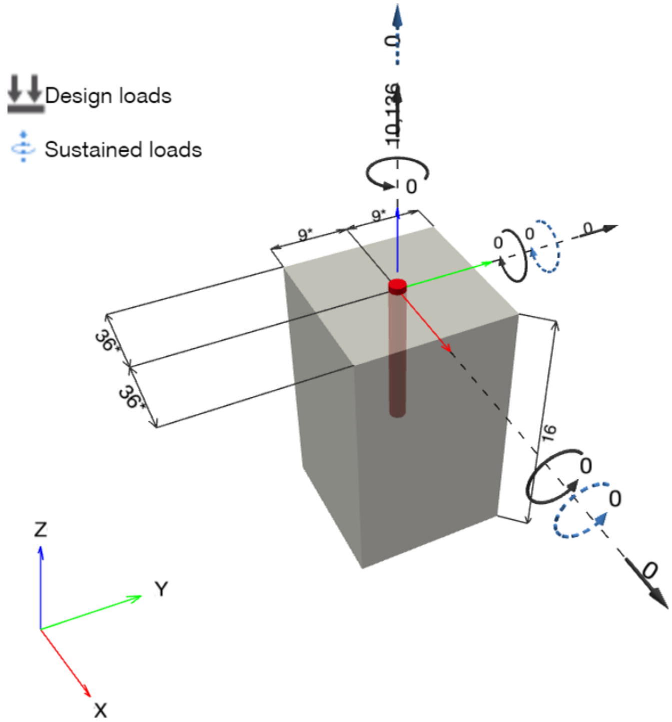
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 1**
Item number:	2198033 HAS-V-36 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 9.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 36
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 16.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 10{,}136$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	96

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	10,136	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	10,136	26,348	39	OK
Bond Strength**	10,136	13,991	73	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	10,136	10,590	96	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value   refer to ICC-ES ESR-4868

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,130

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
35,130	0.750	26,348	10,136

3.2 Bond Strength

N_a	$= \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na} \ \psi_{cp,Na} \ N_{ba}$	ACI 318-19 Eq. (17.6.5.1a)
$\phi \ N_a$	$\geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Na}	see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)	
A_{Na0}	$= (2 \ c_{Na})^2$	ACI 318-19 Eq. (17.6.5.1.2a)
c_{Na}	$= 10 \ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-19 Eq. (17.6.5.1.2b)
$\psi_{ed,Na}$	$= 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.4.1b)
$\psi_{cp,Na}$	$= \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.5.1b)
N_{ba}	$= \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	1.000	9.000	9.000	1.000	1,370

c_{ac} [in.]	λ_a
17.775	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
14.382	517.75	827.38	0.888

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	38,745

Results

N_a [lb]	ϕ_{bond}	$\phi \ N_a$ [lb]	N_{ua} [lb]
21,524	0.650	13,991	10,136

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3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
9.000	9.000	1.000	17.775	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
486.00	729.00	0.900	1.000	27,155

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
16,293	0.650	10,590	10,136

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 1

Item number: 2198033 HAS-V-36 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Maximum installation torque: 1,800 in.lb

Hole diameter in the base material: 1.125 in.

Hole depth in the base material: 9.000 in.

Minimum thickness of the base material: 11.250 in.

1 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	36.000	36.000	9.000	9.000

7 Remarks; Your Cooperation Duties

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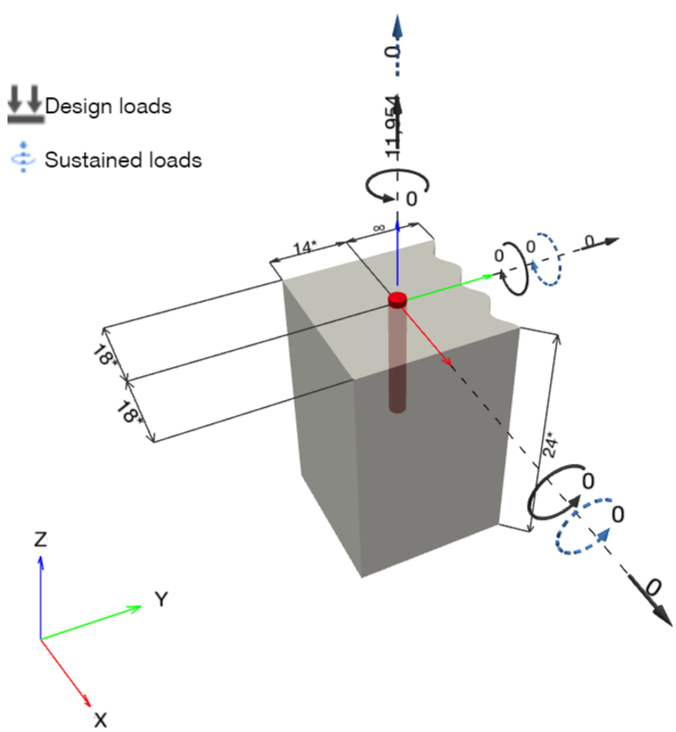
Fastening point:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 1**
Item number:	2198009 HAS-E-55 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 7.500 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 55
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 24.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: > No. 4 bar with stirrups

Geometry [in.] & Loading [lb, in.lb]



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Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 11{,}954$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	90

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11,954	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	11,954	34,072	36	OK
Bond Strength**	11,954	20,543	59	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	11,954	13,427	90	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4868

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	75,000

Calculations

N_{sa} [lb]
45,430

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
45,430	0.750	34,072	11,954

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Fastening point:			

3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$ ACI 318-19 Eq. (17.6.5.1a)

$\phi\ N_a \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	1.000	7.500	14.000	1.000	1,370

c_{ac} [in.]	λ_a
10.776	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
14.382	816.39	827.38	0.992

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	32,288

Results

N_a [lb]	ϕ_{bond}	$\phi\ N_a$ [lb]	N_{ua} [lb]
31,605	0.650	20,543	11,954

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3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-19 Eq. (17.6.2.1a)
$\phi\ N_{cb} \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$	ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
7.500	14.000	1.000	10.776	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
506.25	506.25	1.000	1.000	20,657

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
20,657	0.650	13,427	11,954

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 1

Item number: 2198009 HAS-E-55 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Maximum installation torque: 1,800 in.lb

Hole diameter in the base material: 1.125 in.

Hole depth in the base material: 7.500 in.

Minimum thickness of the base material: 9.750 in.

1 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	18.000	18.000	14.000	-

7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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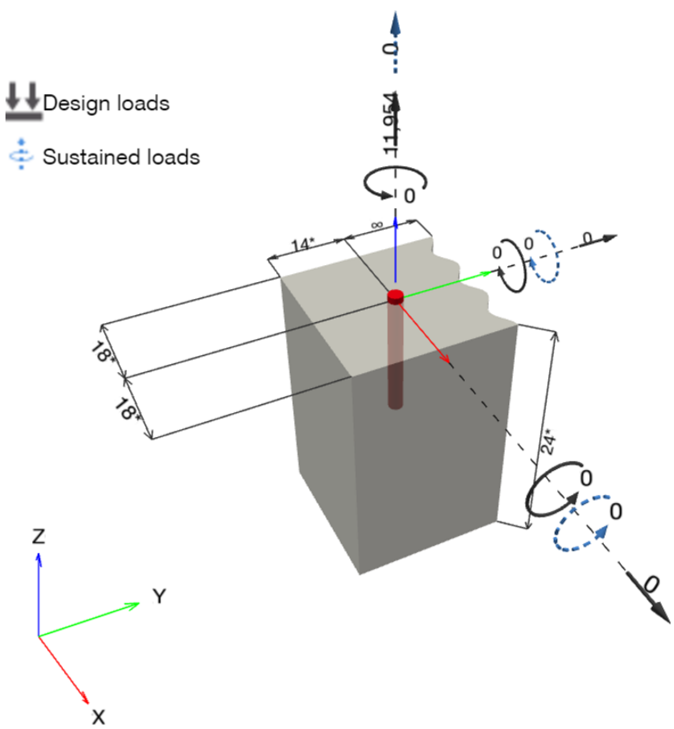
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 7/8**
Item number:	2198007 HAS-E-55 7/8"x10" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 7.500 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 55
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 24.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: > No. 4 bar with stirrups

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 11{,}954$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	90

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11,954	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	11,954	25,973	47	OK
Bond Strength**	11,954	17,879	67	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	11,954	13,427	90	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4868

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	75,000

Calculations

N_{sa} [lb]
34,630

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
34,630	0.750	25,973	11,954

3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$	ACI 318-19 Eq. (17.6.5.1a)
$\phi\ N_a \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)	
$A_{Na0} = (2\ c_{Na})^2$	ACI 318-19 Eq. (17.6.5.1.2a)
$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-19 Eq. (17.6.5.1.2b)
$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.4.1b)
$\psi_{cp,Na} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.5.1b)
$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	0.875	7.500	14.000	1.000	1,334

c_{ac} [in.]	λ_a
11.367	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
12.584	633.46	633.46	1.000

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	27,506

Results

N_a [lb]	ϕ_{bond}	$\phi\ N_a$ [lb]	N_{ua} [lb]
27,506	0.650	17,879	11,954

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
7.500	14.000	1.000	11.367	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
506.25	506.25	1.000	1.000	20,657

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
20,657	0.650	13,427	11,954

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Anchor type and diameter: HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 7/8

Profile: -

Item number: 2198007 HAS-E-55 7/8"x10" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Hole diameter in the fixture: -

Maximum installation torque: 1,500 in.lb

Plate thickness (input): -

Hole diameter in the base material: 1.000 in.

Hole depth in the base material: 7.500 in.

Drilling method: Hammer drilled

Minimum thickness of the base material: 9.500 in.

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

7/8 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	18.000	18.000	14.000	-

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7 Remarks; Your Cooperation Duties

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- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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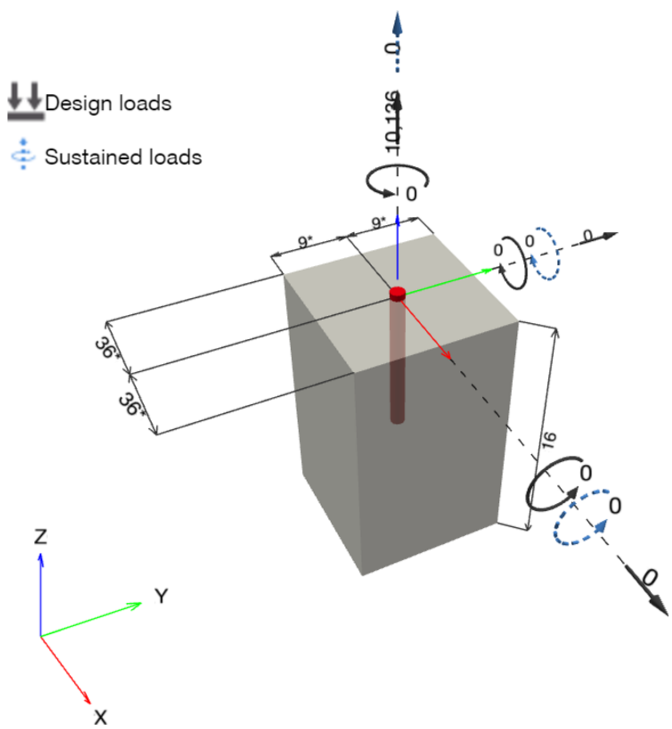
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 7/8**
Item number:	not available (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 9.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 36
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 16.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 10,136; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	96

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	10,136	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	10,136	20,085	51	OK
Bond Strength**	10,136	14,033	73	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	10,136	10,590	96	OK

* highest loaded anchor **anchor group (anchors in tension)

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3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4868

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	58,000

Calculations

N_{sa} [lb]
26,780

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
26,780	0.750	20,085	10,136

3.2 Bond Strength

N_a	$= \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$	ACI 318-19 Eq. (17.6.5.1a)
$\phi\ N_a$	$\geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Na}	see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)	
A_{Na0}	$= (2\ c_{Na})^2$	ACI 318-19 Eq. (17.6.5.1.2a)
c_{Na}	$= 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-19 Eq. (17.6.5.1.2b)
$\psi_{ed,Na}$	$= 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.4.1b)
$\psi_{cp,Na}$	$= \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.5.1b)
N_{ba}	$= \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	0.875	9.000	9.000	1.000	1,334

c_{ac} [in.]	λ_a
18.751	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
12.584	453.04	633.46	0.915

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	33,007

Results

N_a [lb]	ϕ_{bond}	$\phi\ N_a$ [lb]	N_{ua} [lb]
21,589	0.650	14,033	10,136

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
9.000	9.000	1.000	18.751	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
486.00	729.00	0.900	1.000	27,155

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
16,293	0.650	10,590	10,136

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 7/8

Item number: not available (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Maximum installation torque: 1,500 in.lb

Hole diameter in the base material: 1.000 in.

Hole depth in the base material: 9.000 in.

Minimum thickness of the base material: 11.000 in.

7/8 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	36.000	36.000	9.000	9.000

7 Remarks; Your Cooperation Duties

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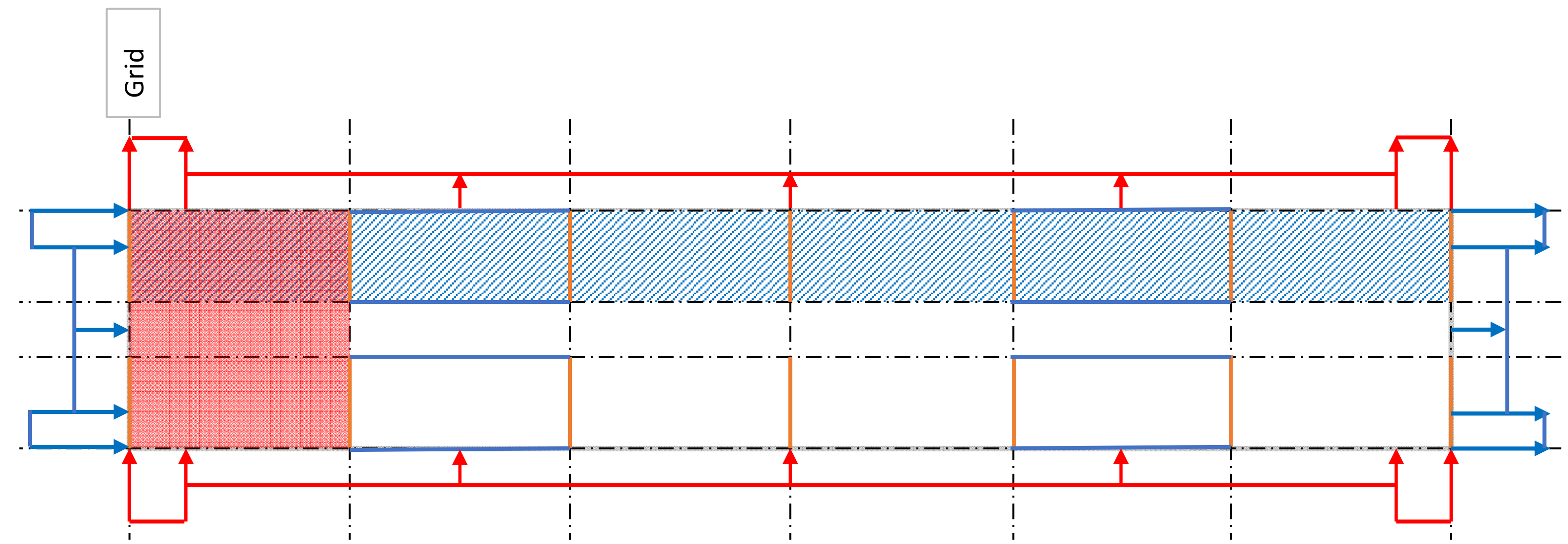
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

LATERAL STABILITY

Lateral Loads - Wind

ASCE 7-16 V= 109 mph

Zone	Combined	Windward	Leeward
Zone 1E/4E (psf)	26.7	20.3	6.4
Zone 1/4 (psf)	17.6	5.6	12.0
Parapet (psf)	65.6	39.4	26.2



Envelope Wind Pressures

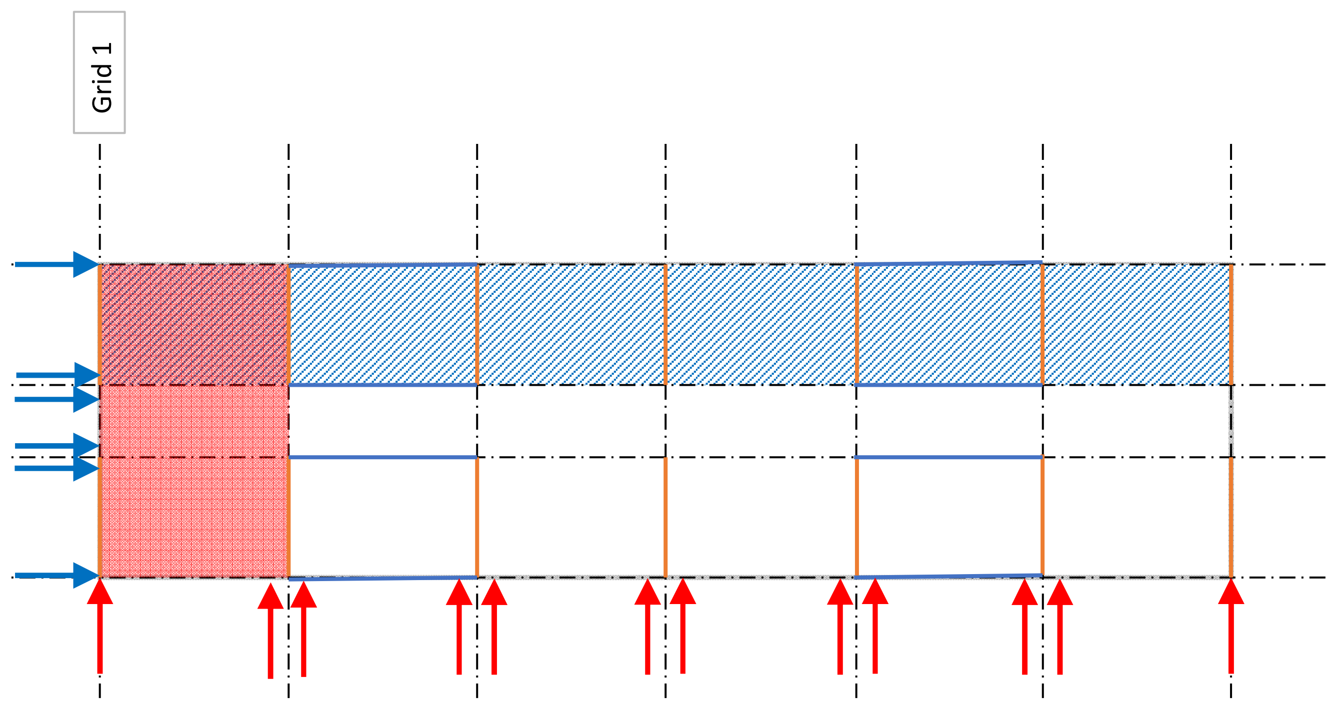
Wind diaphragm pressures

Level	Trib. Height (ft)	w endzone (plf)	w (plf)
Foundation	4.74	127	84
2nd floor	9.49	253	167
3rd floor	9.49	253	167
4th floor	8.79	235	155
Roof	5.05	135	89
Parapet	4.50	295	295
Roof total		430	384

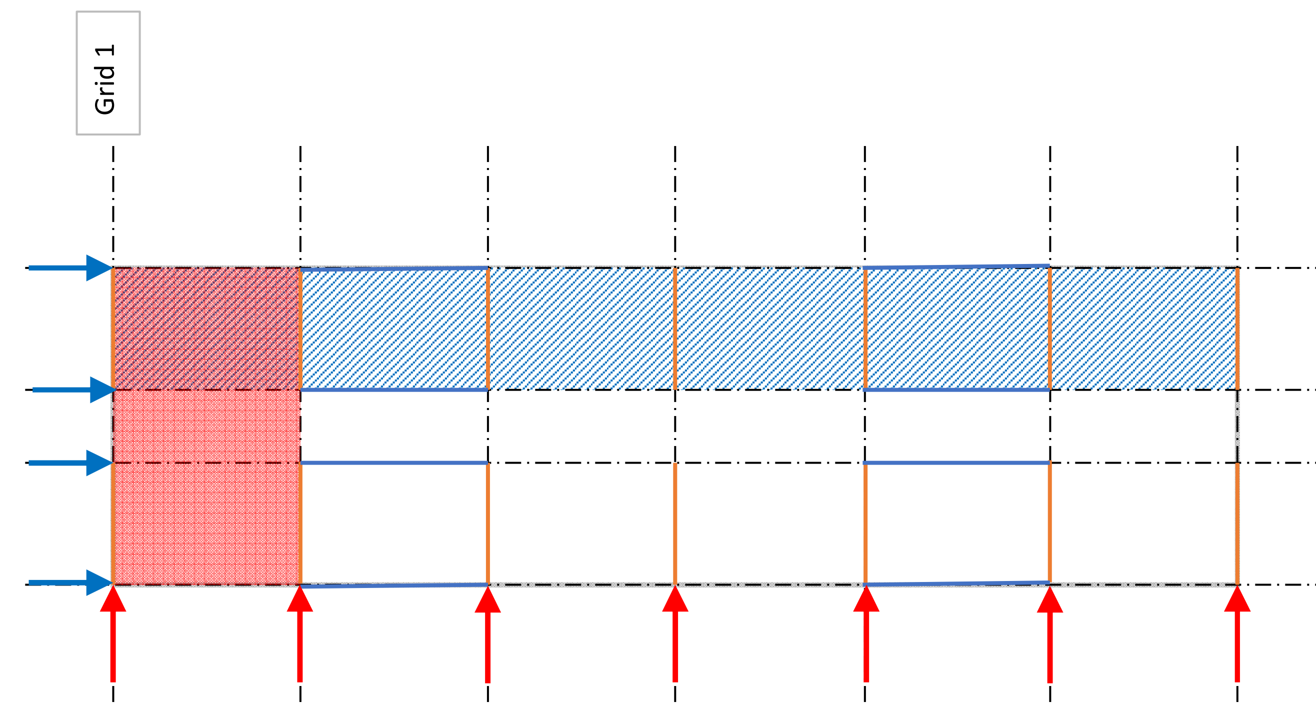
N-S Wind Direction ***(orientation of building may change - for analysis purposes Up is north)***

Diaphragm Loads (lbs)

		Tributary Width		Total Diaphragm Shear (lbs)						
Grids		End Zone	Middle Zone	Foundation	2nd floor	3rd floor	4th floor	Roof	Roof Parapet	Total Base Shear
A to B&B.2	A east	6.77	0.00	858	1715	1715	1589	912	1999	8788
	B/B.2 west	6.39	0.38	841	1683	1683	1559	895	1999	8658
B/B.2 to D/D.	B east	0.00	13.66	1141	2281	2281	2114	1213	4032	13063
	D west	0.00	13.66	1141	2281	2281	2114	1213	4032	13063
D to F	D east	0.00	6.75	564	1127	1127	1044	600	1993	6455
	F west	0.00	6.75	564	1127	1127	1044	600	1993	6455
F to G	F east	0.00	6.75	564	1127	1127	1044	600	1993	6455
	G west	0.00	6.75	564	1127	1127	1044	600	1993	6455
G to H.2	G east	0.00	6.75	564	1127	1127	1044	600	1993	6455
	H west	0.00	6.75	564	1127	1127	1044	600	1993	6455
H.2 to J	H.2 east	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
	J west	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
J to L	J east	12.93	0.00	1638	3276	3276	3035	1742	3817	16785
	L west	0.23	12.70	1090	2179	2179	2019	1159	3817	12444
L left to L rigl	L east	0.00	2.81	235	469	469	435	250	830	2687
	L west	0.00	2.81	235	469	469	435	250	830	2687
L right to N	L east	0.23	12.70	1090	2179	2179	2019	1159	3817	12444
	N west	12.93	0.00	1638	3276	3276	3035	1742	3817	16785



Total Diaphragm Shear



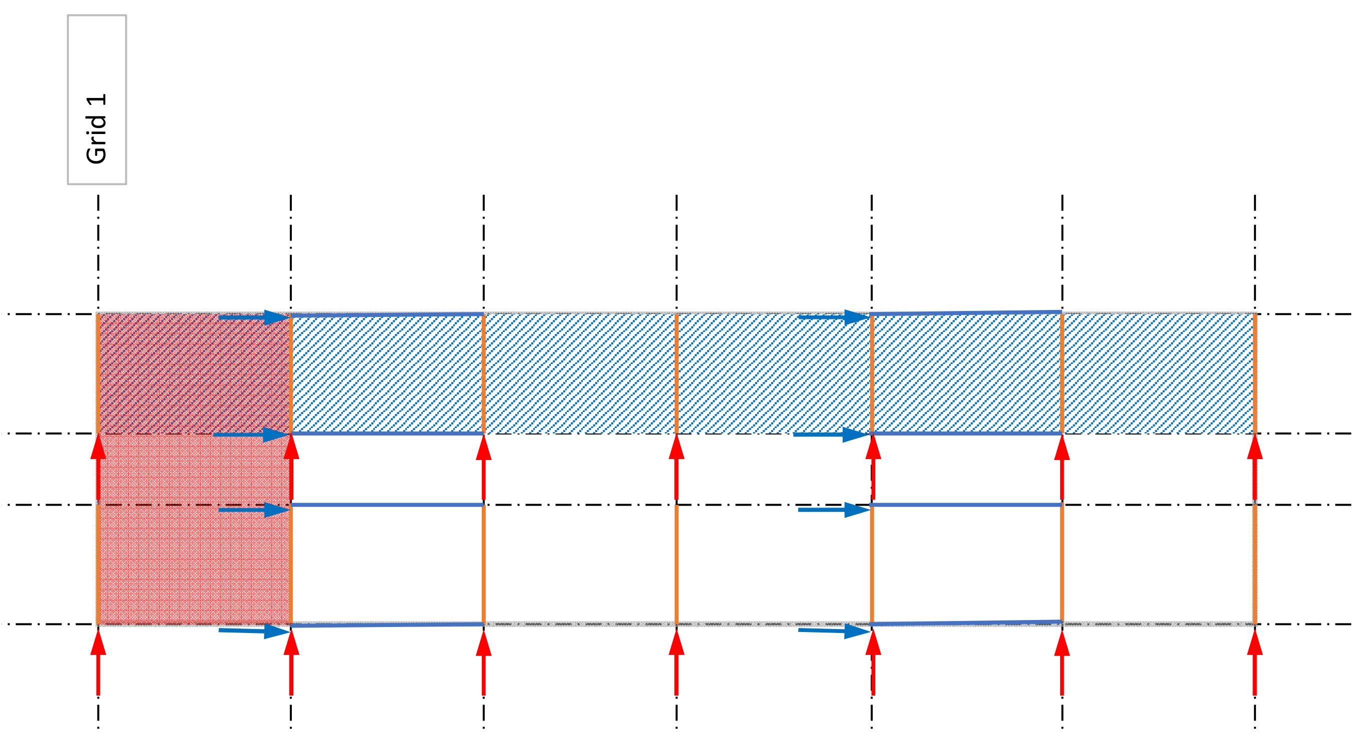
Total Shear

Total Shear at each grid (lbs)

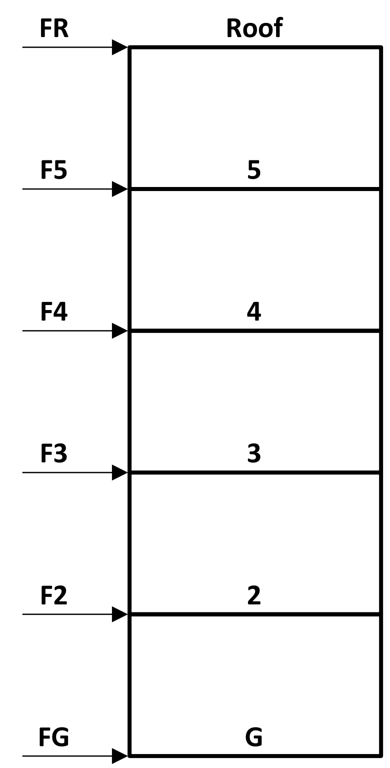
Grid	Foundation	2nd floor	3rd floor	4th floor	Roof	Parapet	Total Base Shear
A	858	1715	1715	1589	912	1999	8788
B/B.2	1982	3964	3964	3672	2108	6031	21721
D	1704	3409	3409	3158	1813	6025	19518
F	1127	2255	2255	2089	1199	3985	12910
G	1127	2255	2255	2089	1199	3985	12910
H	1691	3382	3382	3133	1799	5978	19365
J	2765	5531	5531	5124	2941	7802	29695
L	1324	2649	2649	2454	1409	4646	15131
L Left	1324	2649	2649	2454	1409	4646	15131
L Right	1638	3276	3276	3035	1742	3817	16785
	0	0	0	0	0	0	0
	0	0	0	0	0	0	0

Shearwall lengths - *USER INPUT*

Grid	Wall 1 (longer) length (ft) Length	Wall 2 (shorter) length (ft) Length	Total Perforated Wall Length
A	20.00	0.00	20.00
B/B.2	24.33	20.00	44.33
D	25.85	25.85	51.70
F	25.85	0.00	25.85
G	25.85	0.00	25.85
H	25.85	25.85	51.70
J	40.50	24.35	132.35
	67.50	0.00	
L	118.15	20.35	138.50
L Left	118.15	20.35	138.50
L Right	81.00	53.00	134.00
0			0.00
0			0.00
0			0.00
0			0.00
0			0.00
0			0.00
			0.00

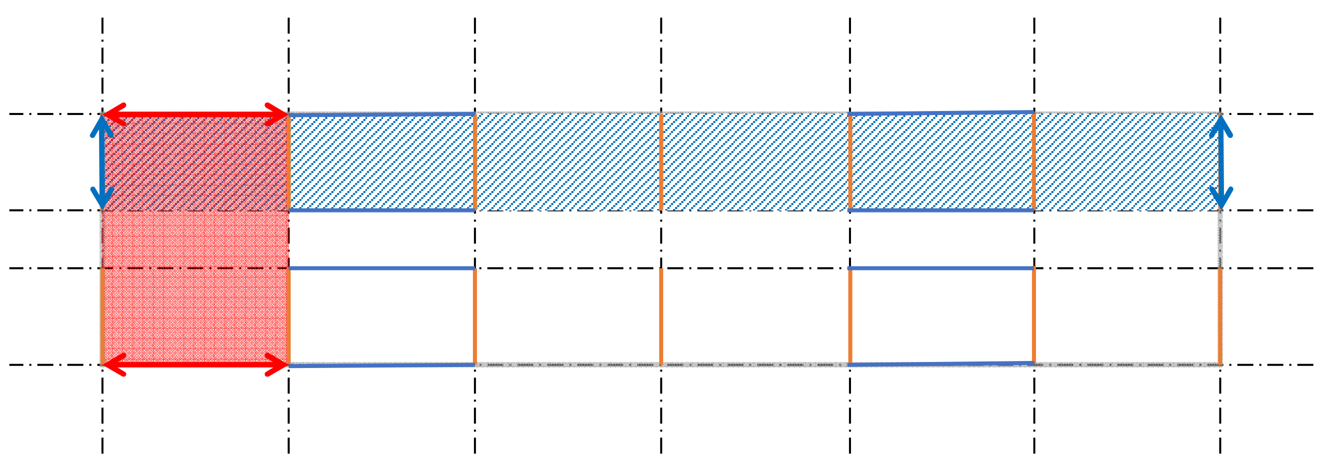


Total Individual Wall Shear



Individual Wall Cumulative Shearwall Shear (lbs)

	Shearwall 1 (longer of 2 cases)						**Shearwall 2 (shorter of 2 cases)**					
Grid	Foundation (FG)	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet	Foundation (FG)	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet
A	8788	7931	6215	4500	2911	1999	0	0	0	0	0	0
B/B.2	11922	10834	8658	6483	4467	3310	9800	8906	7117	5329	3672	2721
D	9760	8908	7203	5498	3919	3013	9758	8906	7202	5498	3919	3012
F	12910	11783	9528	7273	5184	3985	0	0	0	0	0	0
G	12910	11783	9528	7273	5184	3985	0	0	0	0	0	0
H	9683	8838	7147	5455	3889	2989	9682	8836	7145	5455	3888	2989
J	9087	8240	6548	4855	3288	2387	5464	4955	3937	2920	1977	1436
	7717	7041	5690	4340	3088	2370						
L	12907	11777	9518	7259	5165	3964	2224	2029	1640	1250	890	683
L Left	12907	11777	9518	7259	5165	3964	2224	2029	1640	1250	890	683
L Right	10146	9156	7175	5195	3360	2307	6639	5991	4695	3399	2199	1510
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!



Chord Forces

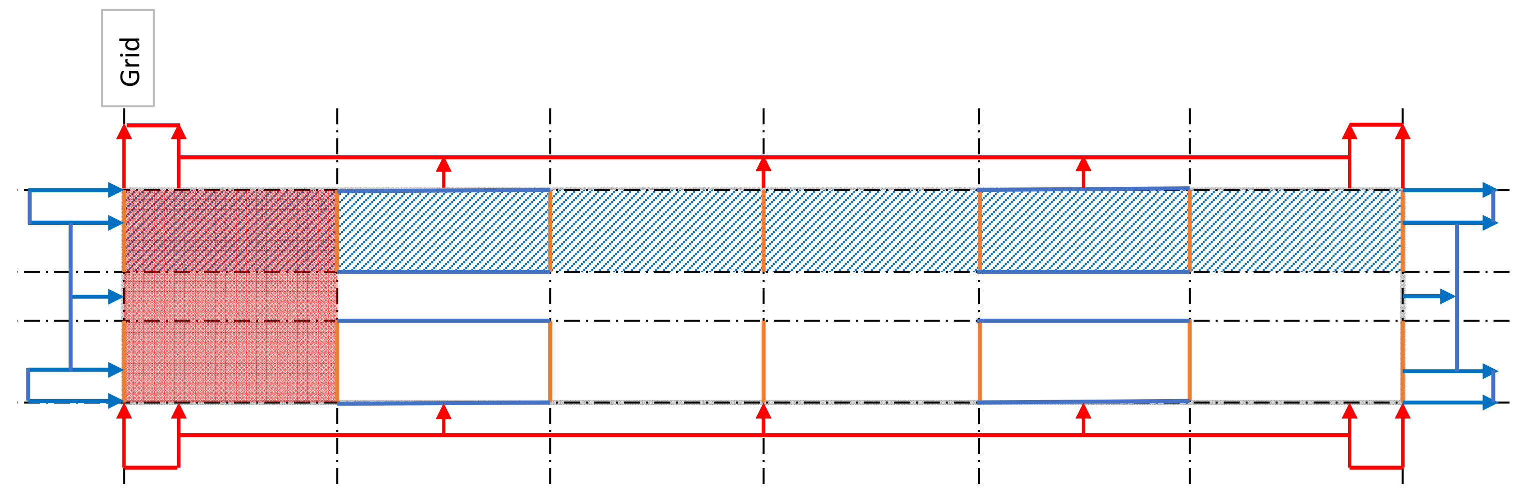
Lateral Loads - Wind

ASCE 7-16 V= 109 mph

Zone	Combined	Windward	Leeward
Zone 1E/4E (psf)	26.7	20.3	6.4
Zone 1/4 (psf)	17.6	5.6	12.0
Parapet (psf)	65.6	39.4	26.2

Wind diaphragm pressures

Level	trib ht. (ft)	w endzone (plf)	w (plf)
Foundation	4.74	127	84
2nd floor	9.49	253	167
3rd floor	9.49	253	167
4th floor	8.79	235	155
Roof	5.05	135	89
Parapet	4.50	295	295
Roof total		430	384

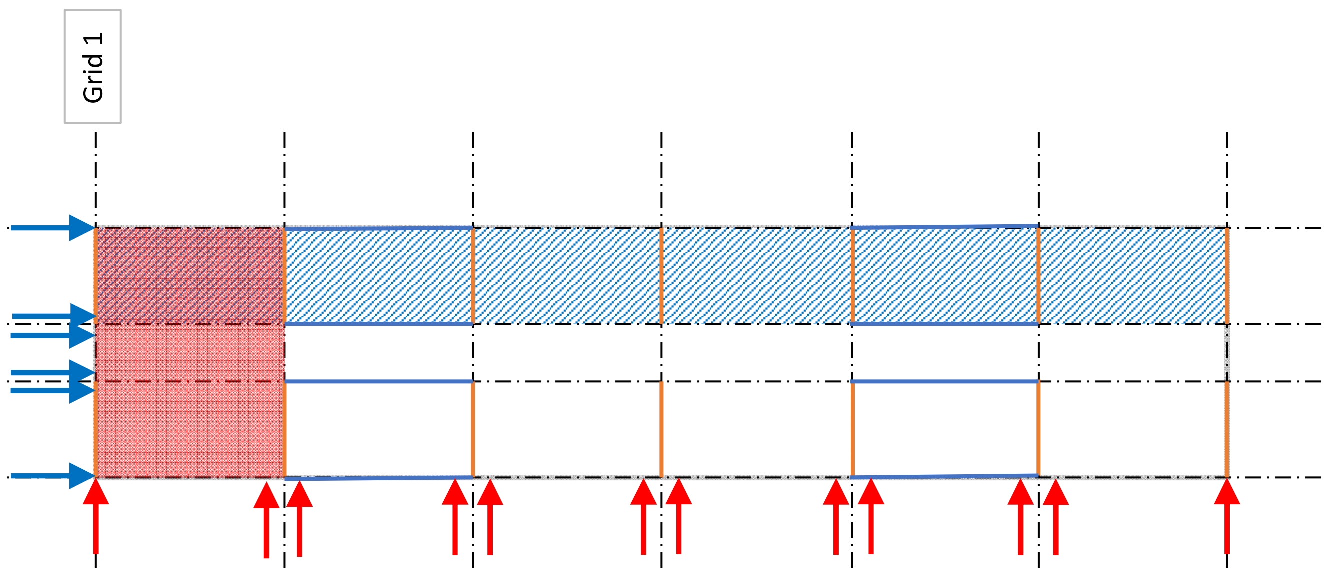


Envelope Wind Pressures

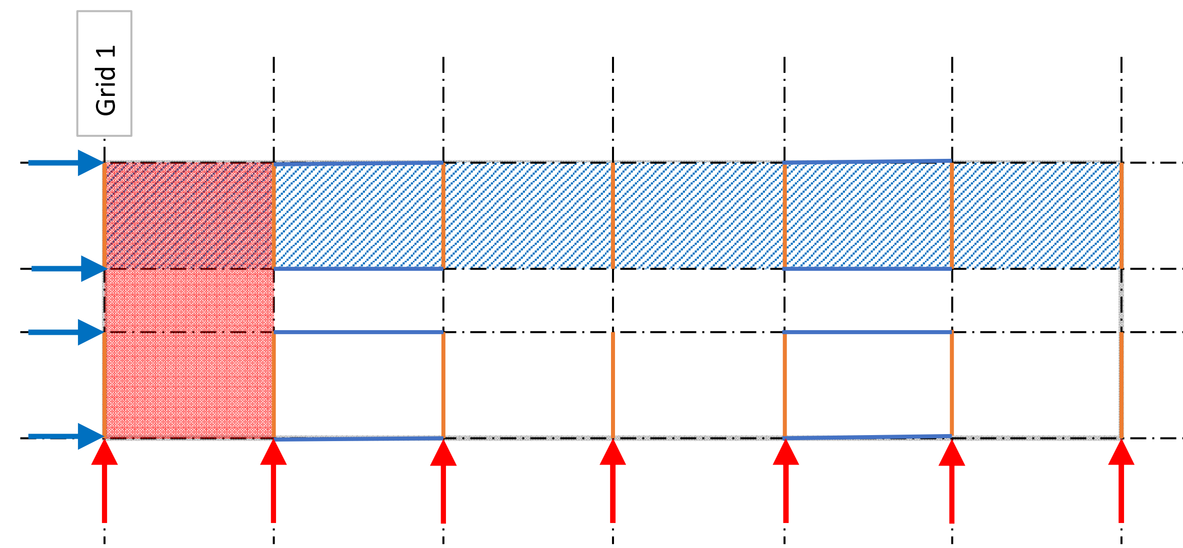
E-W Wind Direction *(orientation of building may change - for analysis purposes Up is north)*

Diaphragm Loads (lbs)

Grids		Tributary Width End Zone	Middle Zone	Total Diaphragm Shear (lbs) Foundation	2nd floor	3rd floor	4th floor	Roof	Roof Parapet	Total Base Shear
0 to 1	1 South	0.00	0.00	0	0	0	0	0	0	0
1 to 2	1 North	4.89	0.00	619	1238	1238	1147	658	1442	6341
	2 South	4.77	0.00	605	1210	1210	1121	643	1409	6198
2 to 3	2 North	0.11	13.39	1132	2264	2264	2098	1204	3985	12948
	3 South	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
3 to 4	3 North	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
	4 South	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
4 to 7	4 North	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
	7 South	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
7 to 10	7 North	0.00	13.50	1127	2255	2255	2089	1199	3985	12910
	10 South	3.19	13.50	1531	3062	3062	2837	1629	4926	17048
10 to 11	10 North	0.00	2.89	241	482	482	446	256	852	2759
	11 South	0.00	2.89	241	482	482	446	256	852	2759
11 to 12,13,1	11 North	0.23	12.69	1090	2179	2179	2019	1159	3816	12442
	12,13,14 South	12.93	0.00	1638	3275	3275	3034	1742	3816	16781



Total Diaphragm Shear



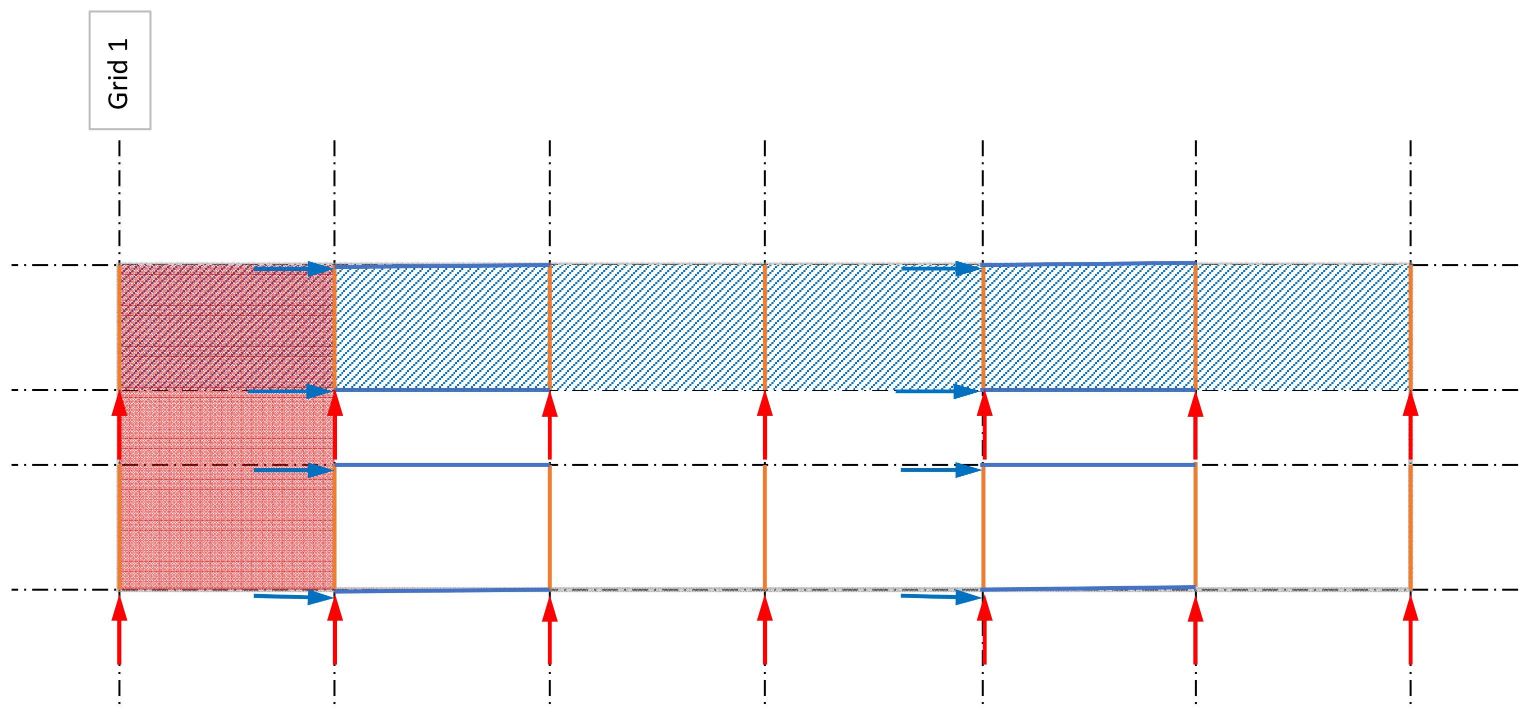
Total Shear

Total Shear at each grid (lbs)

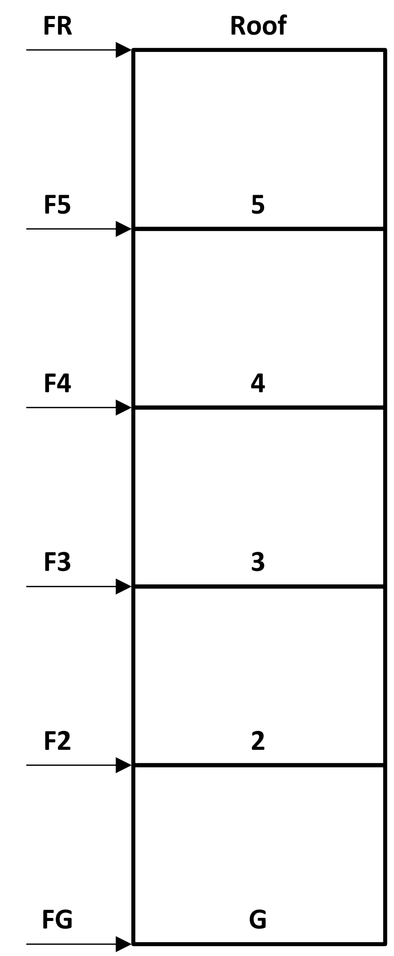
Grid	Foundation	2nd floor	3rd floor	4th floor	Roof	Parapet	Total Base Shear
1	619	1238	1238	1147	658	1442	6341
2	1737	3474	3474	3219	1848	5395	19146
3	2255	4509	4509	4178	2398	7970	25820
4	2255	4509	4509	4178	2398	7970	25820
7	2255	4509	4509	4178	2398	7970	25820
10	1772	3544	3544	3284	1885	5778	19807
11	1331	2661	2661	2465	1415	4668	15201
12,13,14	1638	3275	3275	3034	1742	3816	16781
	0	0	0	0	0	0	0

Shearwall lengths - *USER INPUT*

Grid	Wall 1 (longer) length (ft)		Wall 2 (shorter) length (ft)		Total Perforated Wall Length
	Total	Full Height Length	Total	Full Height Length	
1	22	19	0	0	22
2	22	22	20	20	42
3	20	20	20	20	40
4	20	20	20	20	40
7	25	25	24	24	48
10	108	84	25	25	133
11	162.10	129.10	0.00	0.00	162.10
12,13,14	54.00	38.00	41.13	25.13	95.13

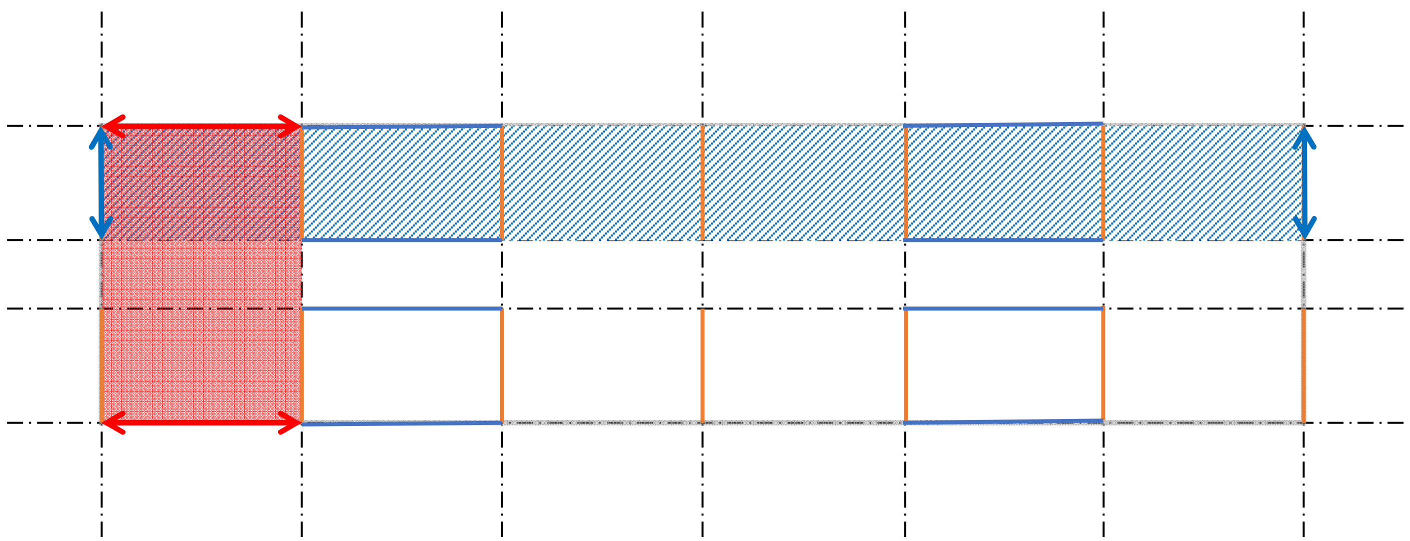


Total Individual Wall Shear



Individual Wall Cumulative Shearwall Shear (lbs)

Grid	Shearwall 1 (longer of 2 cases) Foundation (FG)	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet	Shearwall 2 (shorter of 2 cases) Foundation (F	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet
1	6341	5722	4485	3247	2100	1442	0	0	0	0	0	0
2	10026	9116	7297	5478	3793	2825	8237	7409	5754	4100	2567	1686
3	12910	11783	9528	7273	5184	3985	12910	11783	9528	7273	5184	3985
4	12910	11783	9528	7273	5184	3985	12910	11783	9528	7273	5184	3985
7	13183	12032	9729	7427	5294	4069	12637	11534	9327	7120	5075	3901
10	16119	14677	11793	8908	6236	4702	3688	3358	2698	2038	1427	1076
11	15201	13871	11209	8548	6083	4668	0	0	0	0	0	0
12,13,14	9526	8596	6737	4878	3155	2166	5113	4601	3577	2553	1605	1060



Chord Forces

BSE STRUCTURAL ENGINEERS

Project WSS LEE'S SUMMIT Project No. 23-283

Calc. By GL Checked By DN Date 7/28/2023

Seismic Weights

Floor Areas

Level	Floor Area	Roof Area	Total Area (sf)	W (lb)	Roof Snow Load (psf)
Roof	0	12941.93	12941.93	323548.25	20
4th	12941.93	0	12941.93	323548.25	
3rd	12941.93	0	12941.93	323548.25	
2nd	12941.93	880.617	13822.547	345563.675	

Interior Walls

Location	Length		Quantity
Short - Grid 1-6, 14-18	20'	4.250"	16
Med. - Grid 6-9, 12-14	24'	4.250"	12
Long - Grid 9-12	25'	10.250"	5
Corridor	162'	8.000"	3
	0'	0.000"	
Total	**1235'**	**2.250"**	

Miscellaneous Walls

Stair Exterior		Stair Interior		Elevator	
22'	1.500"	22'	1.500"	10'	2.125"
10'	1.750"	10'	1.750"	10'	2.125"
22'	1.500"	22'	1.750"	8'	2.125"
10'	1.750"	10'	1.075"	8'	2.125"
0'	0.000"				
65'	**6.500"**	**65'**	**6.075"**	**37'**	**8.500"**

Exterior Walls

Perimeter

Short - Grid 1-6, 14-18	Med. - Grid 6-9, 12-14	Long - Grid 9-12	End Walls - Grid 1, 20

	54'		27'		34'		28'	
	4'		27'		41'		28'	
	27'		27'					
	81'		27'					
	54'		14'					
	68'		27'					
			4'					
			0'					
Total	**288'**	**0.000''**	**153'**	**0.000''**	**75'**	**0.000''**	**56'**	**0.000''**

Project WSS LEE'S SUMMIT Project No. 23-283
Calc. By GL Checked By DN Date 7/28/2023

Short - Grid 1-6 & 14-18

Elevations			H (ft)	Ext. Wall Weight (psf)	Ext. Wall Weight (plf)	L (ft)	W (lb)
136.56	138.56	Roof	2.00 '	25	151.17	288.13 '	43556.4
128.47	136.56	4th FLR	8.09 '	25	219.79	288.13 '	63327.5
118.98	128.47	3rd FLR	9.49 '	25	237.24	288.13 '	68354.7
109.49	118.98	2nd FLR	9.49 '	25	237.24	288.13 '	68354.7
100.00	109.49	1st FLR	9.49 '	25			

Medium - Grid 6-9 & 12-14

Elevations			H (ft)	Ext. Wall Weight (psf)	Ext. Wall Weight (plf)	L (ft)	W (lb)
136.56	138.56	Roof	2.00 '	25	151.17	152.81 '	23101.0
128.47	136.56	4th FLR	8.09 '	25	219.79	152.81 '	33586.9
118.98	128.47	3rd FLR	9.49 '	25	237.24	152.81 '	36253.2
109.49	118.98	2nd FLR	9.49 '	25	237.24	152.81 '	36253.2
100.00	109.49	1st FLR	9.49 '	25			

Long - Grid 9-12

Elevations			H (ft)	Ext. Wall Weight (psf)	Wall Weight (plf)	L (ft)	W (lb)
136.56	138.56	Roof	2.00 '	25	151.17	74.75 '	11300.1
128.47	136.56	4th FLR	8.09 '	25	219.79	74.75 '	16429.4
118.98	128.47	3rd FLR	9.49 '	25	237.24	74.75 '	17733.7
109.49	118.98	2nd FLR	9.49 '	25	237.24	74.75 '	17733.7
100.00	109.49	1st FLR	9.49 '	25			

Ends - Grid 1, 20

Elevations			H (ft)	Ext. Wall Weight (psf)	Wall Weight (plf)	L (ft)	W (lb)

BSE STRUCTURAL ENGINEERS

Project WSS LEE'S SUMMIT Project No. 23-283

Calc. By GL Checked By DN Date 7/28/2023

136.56	138.56	Roof	2.00 '	12	72.56	56.00 '	4063.5
128.47	136.56	4th FLR	8.09 '	12	105.50	56.00 '	5908.0
118.98	128.47	3rd FLR	9.49 '	12	113.88	56.00 '	6377.0
109.49	118.98	2nd FLR	9.49 '	12	113.88	56.00 '	6377.0
100.00	109.49	1st FLR	9.49 '	12			

Stair Exterior

Elevations			H (ft)	Ext. Wall Weight (psf)	Wall Weight (plf)	L (ft)	W (lb)
136.56	138.56	Roof	2.00 '	12	72.56	65.31 '	4739.2
128.47	136.56	4th FLR	8.09 '	12	105.50	65.31 '	6890.4
118.98	128.47	3rd FLR	9.49 '	12	113.88	65.31 '	7437.3
109.49	118.98	2nd FLR	9.49 '	12	113.88	65.31 '	7437.3
100.00	109.49	1st FLR	9.49 '	12			

Project WSS LEE'S SUMMIT Project No. 23-283
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Stair Interior

Elevations			H (ft)	Int. Wall Weight (psf)	Wall Weight (plf)	L (ft)	W (lb)
136.56	138.56	Roof	2.00 '	11	66.52	65.28 '	4341.9
128.47	136.56	4th FLR	8.09 '	11	96.71	65.28 '	6312.8
118.98	128.47	3rd FLR	9.49 '	11	104.39	65.28 '	6814.0
109.49	118.98	2nd FLR	9.49 '	11	104.39	65.28 '	6814.0
100.00	109.49	1st FLR	9.49 '	11			

Masonry/Wood Elevator

Elevations			H (ft)	Wall Weight (psf)	Wall Weight (plf)	L (ft)	W (lb)
136.56	138.56	Roof	2.00 '	60.00	362.81	37.44	13582.7
128.47	136.56	4th FLR	8.09 '	60.00	527.50	37.44	19748.2
118.98	128.47	3rd FLR	9.49 '	60.00	569.38	37.44	21315.9
109.49	118.98	2nd FLR	9.49 '	60.00	569.38	37.44	21315.9
100.00	109.49	1st FLR	9.49 '	60.00			

Interior Walls

Elevations			H (ft)	Int. Wall Weight (psf)	Wall Weight (plf)	L (ft)	W (lb)
136.56	136.56	Roof	0.00 '	11	44.52	1235.19 '	54985.1
128.47	136.56	4th FLR	8.09 '	11	96.71	1235.19 '	119452.9
118.98	128.47	3rd FLR	9.49 '	11	104.39	1235.19 '	128935.6
109.49	118.98	2nd FLR	9.49 '	11	104.39	1235.19 '	128935.6
100.00	109.49	1st FLR	9.49 '	11			

Floor	Total Weight

Floor	(kips)
Roof	483.22
4th FLR	595.20
3rd FLR	616.77
2nd FLR	638.78

BSE STRUCTURAL ENGINEERS

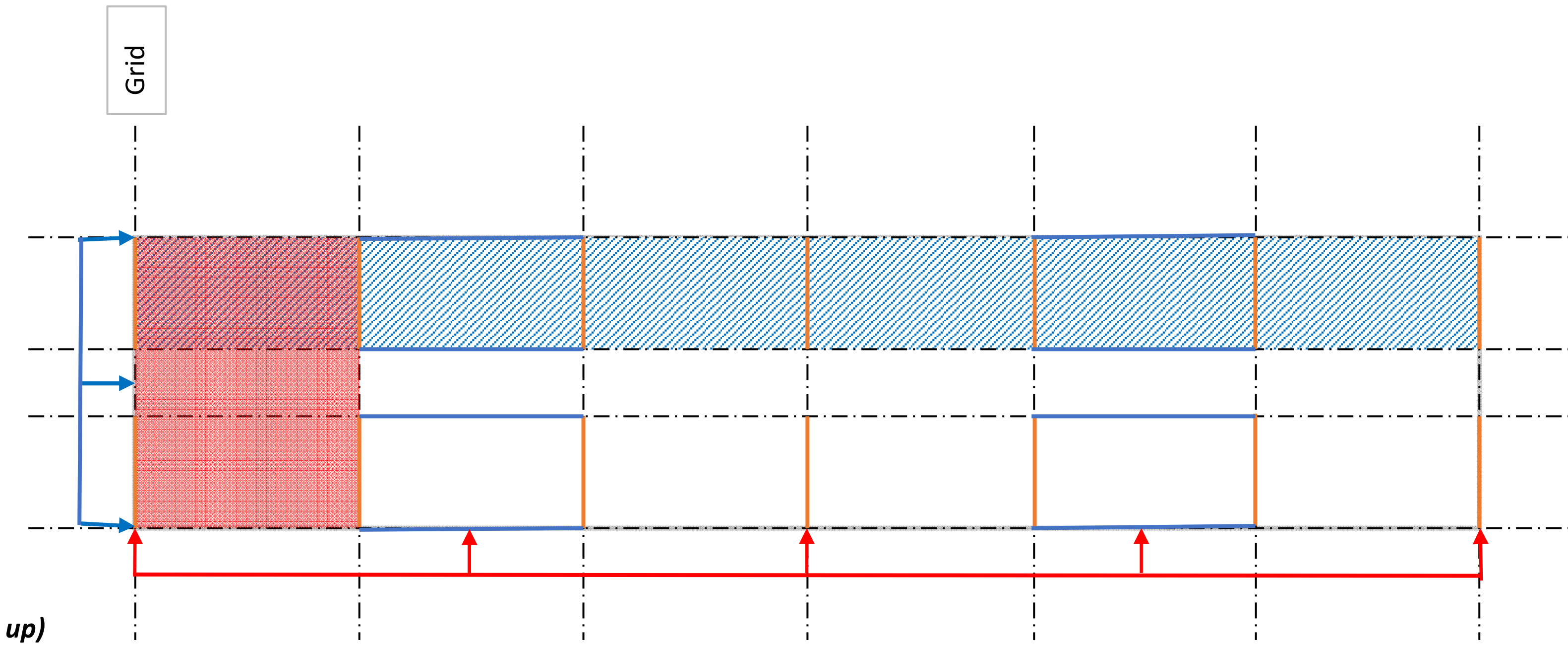
Project WSS LEE'S SUMMIT Project No. 23-283

Calc. By GL Checked By DN Date 7/28/2023

Lateral Loads - Seismic

Ω (overstrength factor)= 1.0

Irregularity Factor = 1.0 (1.0 if non-irregular)



Seismic diaphragm distributed force

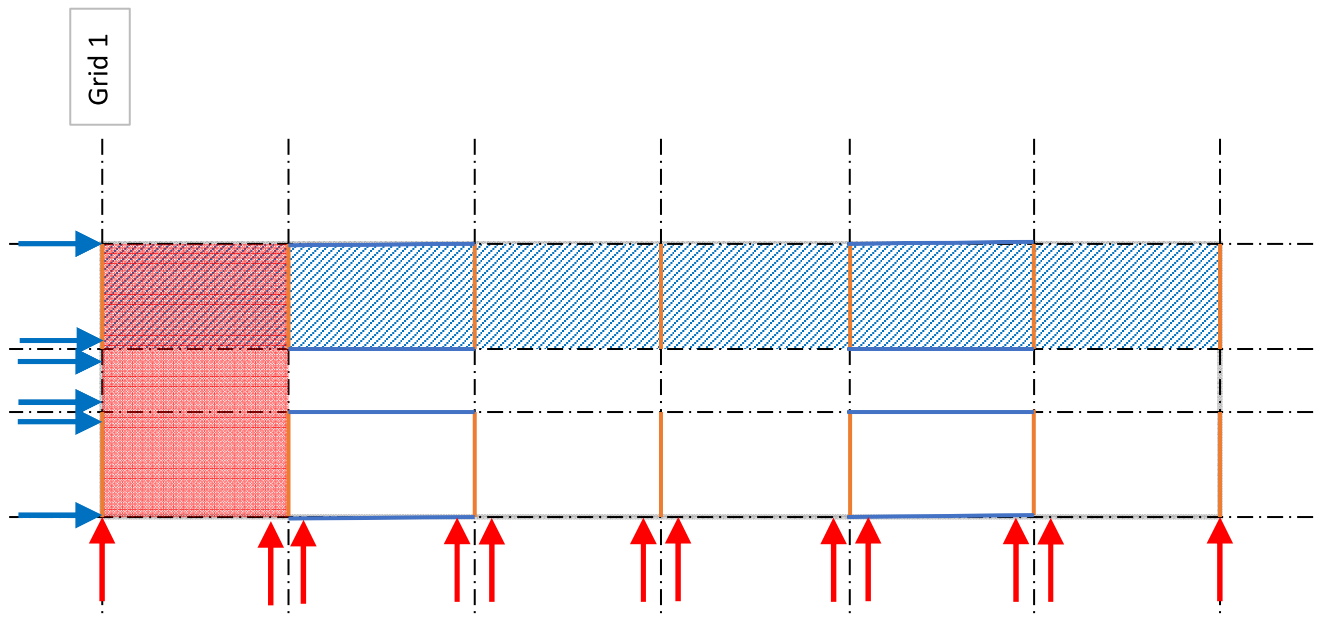
Seismic Force Distribution (all including irregularity factor)

Level	Lateral Force (lbs)	Floor Area (sf)	Distributed Force (psf)
	-		
Roof	15,600	12,942	1.21
4th floor	11,800	12,942	0.91
3rd floor	7,600	12,942	0.59
2nd floor	3,100	13,823	0.22
Total	38,100		

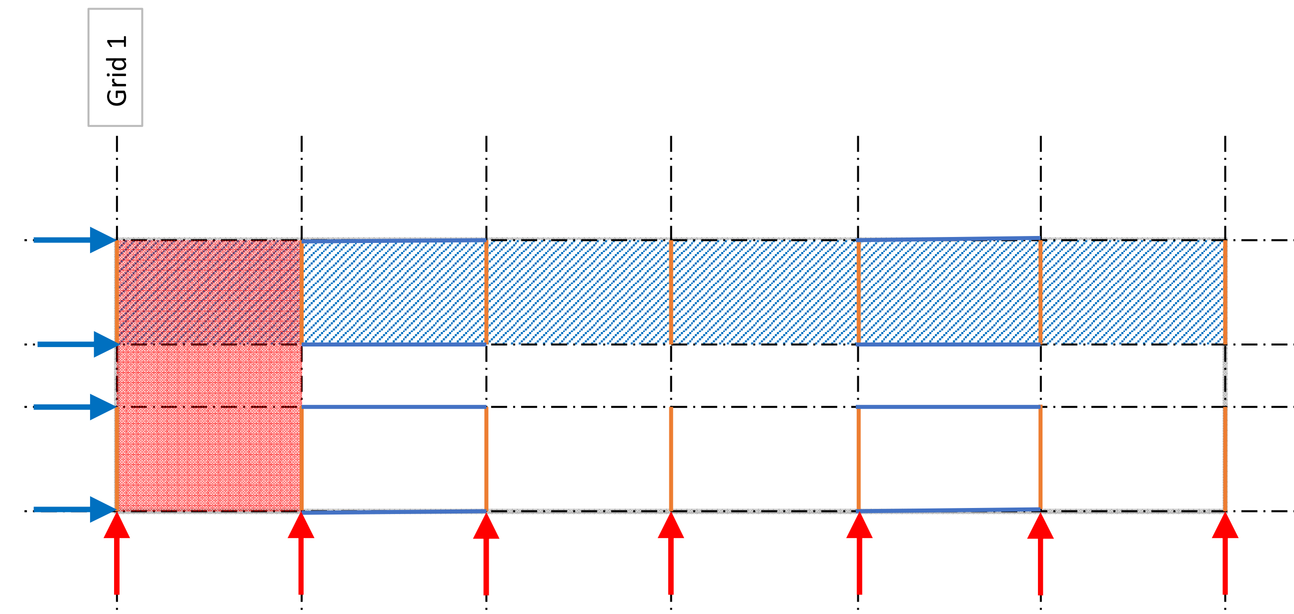
N-S Seismic ***(orientation of building may change - for analysis purposes North is up)***

Diaphragm Loads (lbs)

Grids		Tributary Width	Tributary Length	Total Diaphragm Shear (lbs) 2nd Floor	3rd Floor	4th Floor	Roof	Parapet	Total Base Shear
A to B&B.2	A east	6.77	24.35	37	97	150	199	0	483
	B/B.2 west	6.77	24.35	37	97	150	199	0	483
B/B.2 to D/D.	B east	13.66	24.35	75	195	303	401	0	974
	D west	13.66	24.35	75	195	303	401	0	974
D to F	D east	6.75	25.85	39	102	159	210	0	511
	F west	6.75	25.85	39	102	159	210	0	511
F to G	F east	6.75	25.85	39	102	159	210	0	511
	G west	6.75	60.13	91	238	370	489	0	1189
G to H.2	G east	6.75	60.13	91	238	370	489	0	1189
	H west	6.75	48.85	74	194	301	397	0	966
H.2 to J	H.2 east	13.50	48.85	148	387	601	795	0	1932
	J west	13.50	20.35	62	161	251	331	0	805
J to L	J east	12.93	20.35	59	155	240	317	0	771
	L west	12.93	20.35	59	155	240	317	0	771
L left to L righ	L east	2.81	20.35	13	34	52	69	0	168
	L west	2.81	53.06	33	88	136	180	0	437
L right to N	L east	12.93	81.00	235	615	955	1262	0	3067
	N west	12.93	108.00	313	820	1273	1683	0	4090
0	0.00	0.00	108.00	0	0	0	0	0	0
	0.00	0.00	81.00	0	0	0	0	0	0



Total Diaphragm Shear



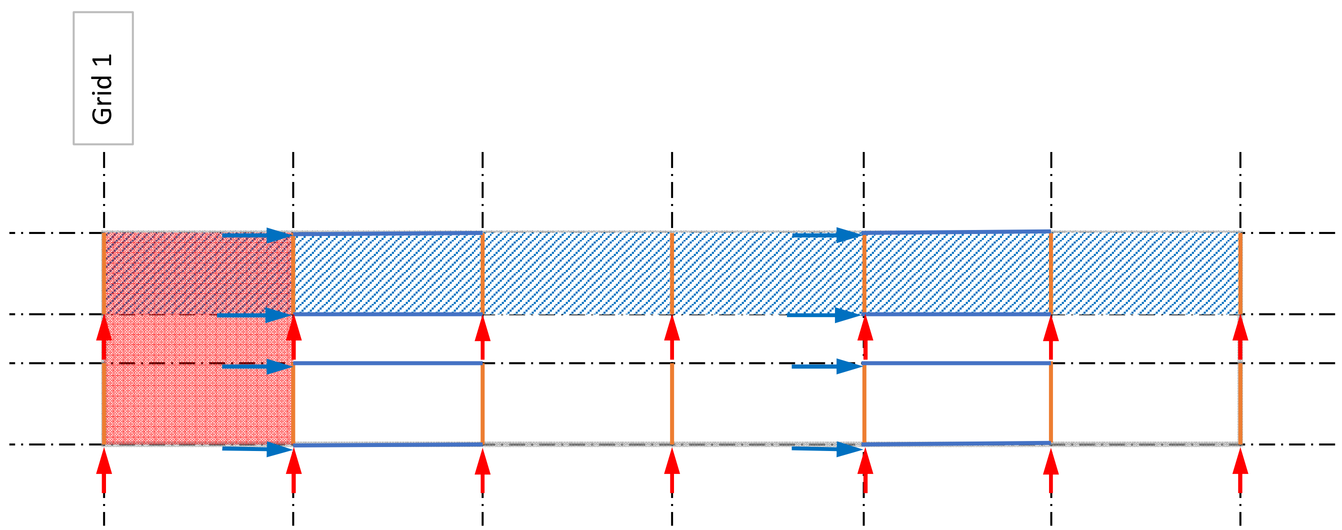
Total Shear

Total Shear at each grid (lbs)

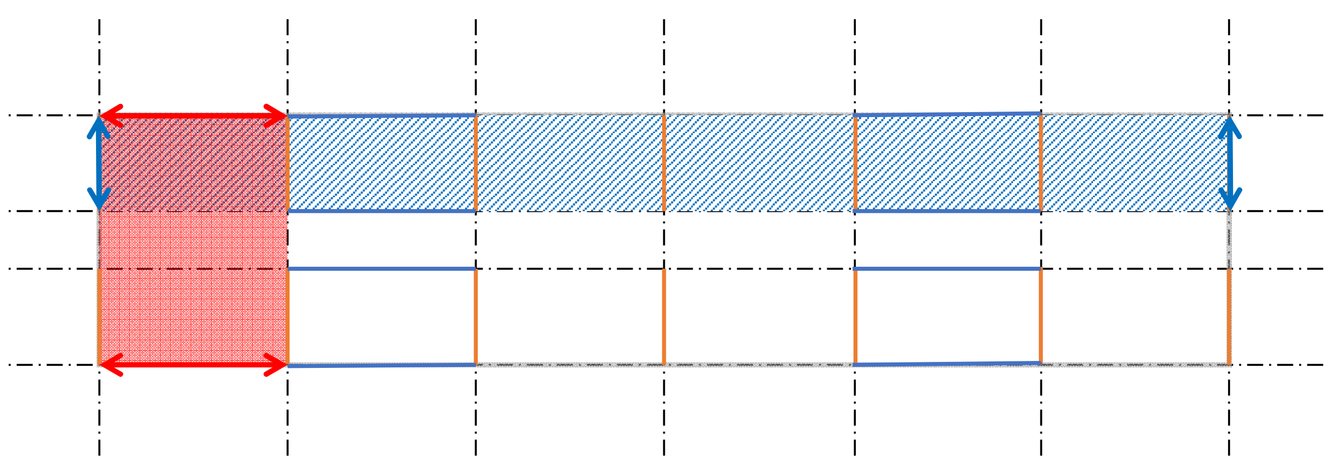
Grid	2nd Floor	3rd Floor	4th Floor	Roof	Parapet	Total Base Shear
A	37	97	150	199	0	483
B/B.2	112	292	454	600	0	1457
D	114	298	462	611	0	1485
F	78	205	318	421	0	1022
G	182	477	740	978	0	2377
H	222	581	902	1192	0	2897
J	121	316	490	648	0	1575
L	72	188	292	386	0	938
L Left	268	703	1091	1442	0	3504
L Right	313	820	1273	1683	0	4090
0	0	0	0	0	0	0

Shearwall lengths -

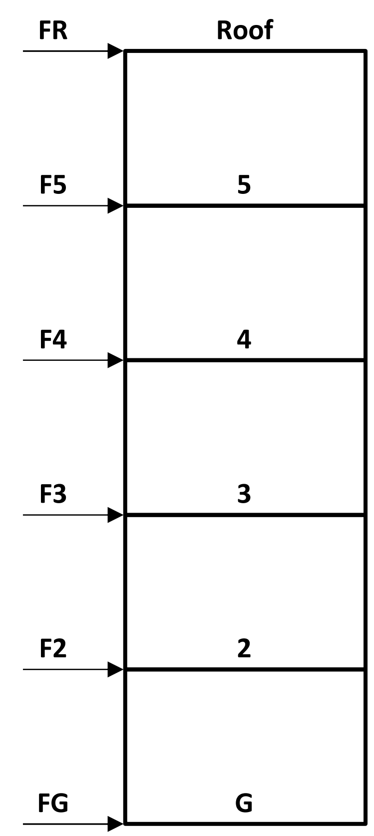
Grid	**Wall 1 (longer) length (ft)**		**Wall 2 (shorter) length (ft)**		Total Shear Wall Length
	Tributary Width	Shear Wall Length	Tributary Width	Shear Wall Length	
A	6.77	20.00	6.77	0.00	20.00
B/B.2	20.43	24.33	20.43	20.00	44.33
D	20.41	25.85	20.41	25.85	51.70
F	13.50	25.85	13.50	0.00	25.85
G	13.50	25.85	13.50	0.00	25.85
H	20.25	25.85	20.25	25.85	51.70
J	26.43	40.50	26.43	24.35	64.85
L	15.74	118.15	15.74	20.35	138.50
L Left	15.74	118.15	15.74	20.35	138.50
L Right	12.93	81.00	12.93	53.00	134.00
0	0.00	0.00	0.00	0.00	0.00
0					
0					
0					



Total Individual Wall Shear



Chord Forces



W/O OVERSTRENGTH

Individual Wall Cumulative Shearwall Shear (lbs)

	Shearwall 1 (longer of 2 cases)					**Shearwall 2 (shorter of 2 cases)**				
Grid	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	5th floor (F5)	Roof (FR)
A	483	446	349	199	0	0	0	0	0	0
B/B.2	800	739	578	329	0	657	607	475	271	0
D	743	686	537	306	0	743	686	537	306	0
F	1022	944	739	421	0	0	0	0	0	0
G	2377	2195	1718	978	0	0	0	0	0	0
H	1449	1338	1047	596	0	1449	1338	1047	596	0
J	984	909	711	405	0	592	546	428	244	0
L	800	739	579	329	0	138	127	100	57	0
L Left	2989	2760	2161	1230	0	515	476	372	212	0
L Right	2472	2283	1787	1017	0	1618	1494	1169	666	0
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

W/ OVERSTRENGTH

Individual Wall Cumulative Shearwall Shear (lbs)

	Shearwall 1 (longer of 2 cases)					**Shearwall 2 (shorter of 2 cases)**				
Grid	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet
A	483	446	349	199	0	0	0	0	0	0
B/B.2	800	739	578	329	0	657	607	475	271	0
D	743	686	537	306	0	743	686	537	306	0
F	1022	944	739	421	0	0	0	0	0	0
G	2377	2195	1718	978	0	0	0	0	0	0
H	1449	1338	1047	596	0	1449	1338	1047	596	0
J	984	909	711	405	0	592	546	428	244	0
L	800	739	579	329	0	138	127	100	57	0
L Left	2989	2760	2161	1230	0	515	476	372	212	0
L Right	2472	2283	1787	1017	0	1618	1494	1169	666	0
0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

BSE STRUCTURAL ENGINEERS

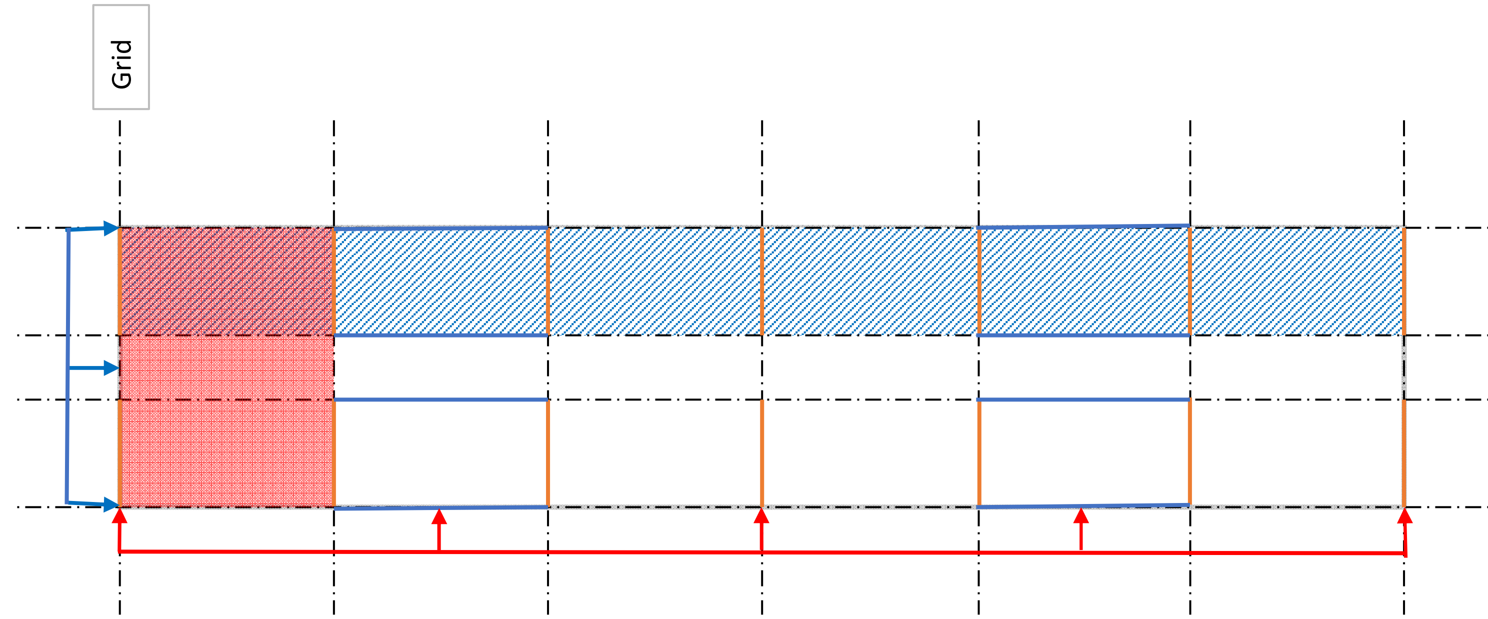
Project WSS LEE'S SUMMIT Project No. 23-283

Calc. By GL Checked By DN Date 7/28/2023

Lateral Loads - Seismic

Ω (overstrength factor)= 1.0

Irregularity Factor = 1.0 (1.0 if non-irregular)



Seismic diaphragm distributed force

Seismic Force Distribution (all including irregularity factor)

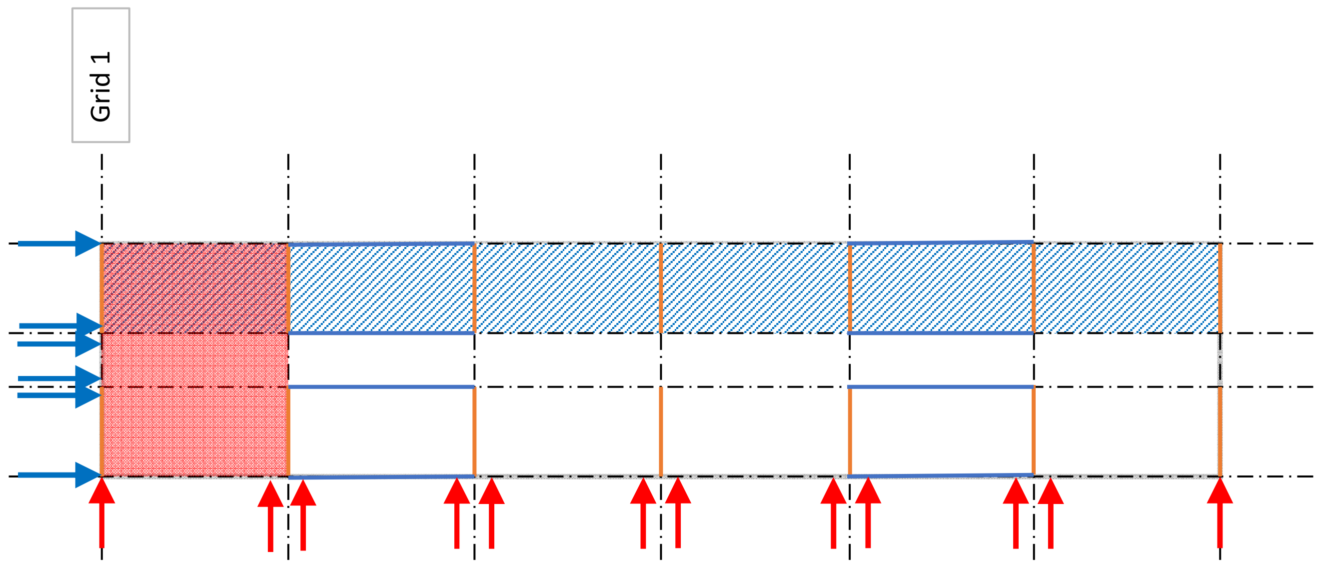
level	Lateral Force (lbs)	Floor Area (sf)	Distributed Force (psf)
	-		
Roof	15,600	12,942	1.21
4th floor	11,800	12,942	0.91
3rd floor	7,600	12,942	0.59
2nd floor	3,100	13,823	0.22
Total	38,100		

E-W Seismic

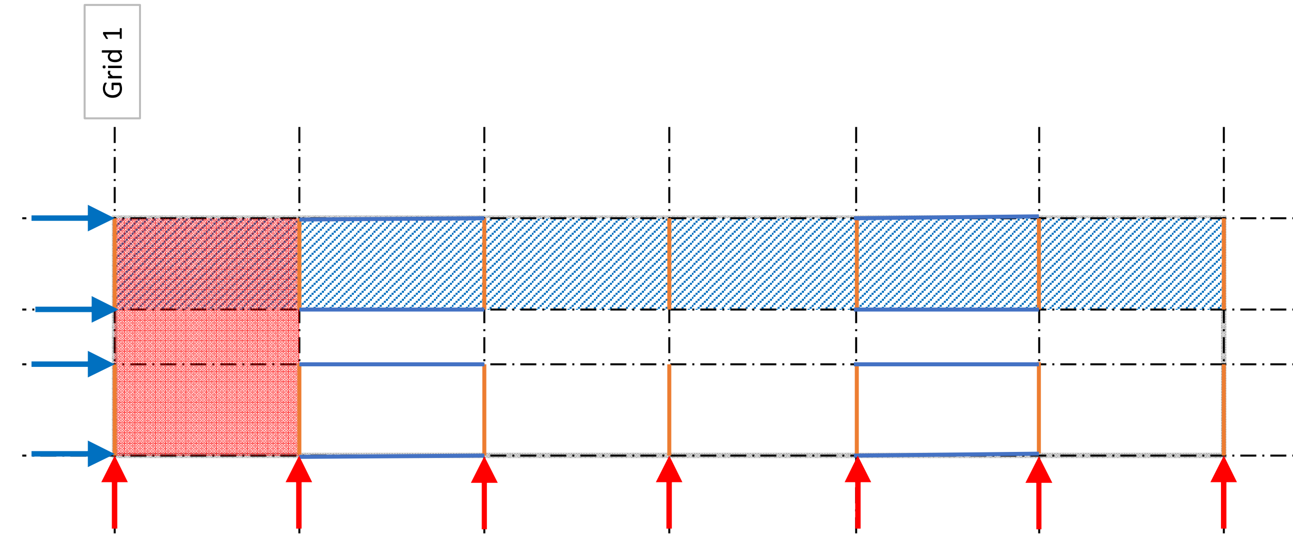
Diaphragm Loads (lbs)

(orientation of building may change - for analysis purposes North is up)

Grids		Tributary Width	Tributary Length	Total Diaphragm Shear (lbs) 2nd Floor	3rd Floor	4th Floor	Roof	Parapet	Total Base Shear
0 to 1	1 South	0.00	0.00	0	0	0	0	0	0
1 to 2	1 North	4.89	18.67	20	54	83	110	0	267
	2 South	4.89	22.13	24	63	99	130	0	317
2 to 3	2 North	13.50	22.13	67	175	272	360	0	875
	3 South	13.50	20.13	61	160	248	327	0	796
3 to 4	3 North	13.50	20.13	61	160	248	327	0	796
	4 South	13.50	20.13	61	160	248	327	0	796
4 to 7	4 North	13.50	20.13	61	160	248	327	0	796
	7 South	13.50	24.71	75	196	304	402	0	977
7 to 10	7 North	13.50	24.71	75	196	304	402	0	977
	10 South	13.50	132.83	402	1053	1635	2162	0	5252
				0	0	0	0	0	0
10 to 11	10 North	2.89	132.83	86	225	349	462	0	1122
	11 South	2.89	149.10	96	253	392	519	0	1260
11 to 12,13,1	11 North	12.93	149.10	432	1132	1757	2323	0	5645
	12,13,14 South	12.93	149.10	432	1132	1757	2323	0	5645
	12,13,14 North	0.00	0.00	0	0	0	0	0	0
				0	0	0	0	0	0
				0	0	0	0	0	0
				0	0	0	0	0	0



Total Diaphragm Shear



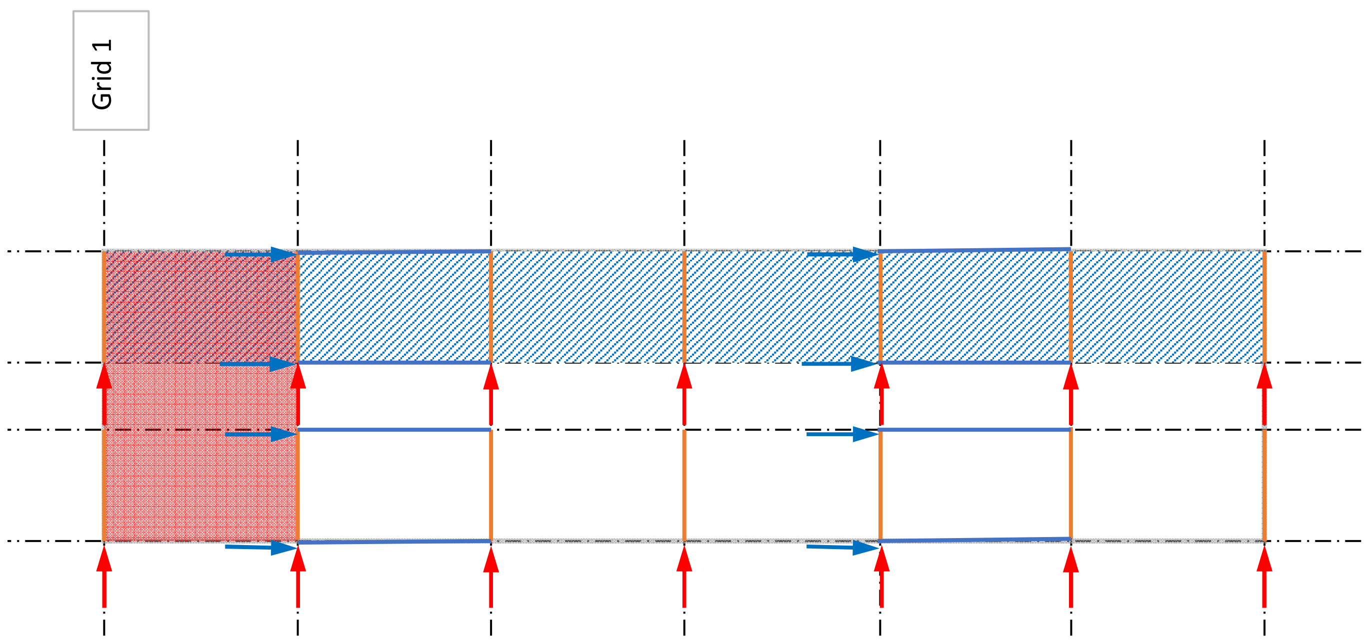
Total Shear

Total Shear at each grid (lbs)

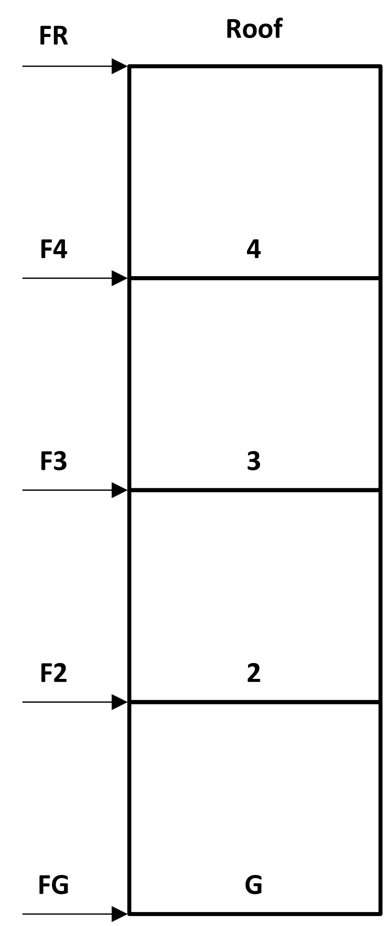
Grid	2nd Floor	3rd Floor	4th Floor	Roof	Parapet	Total Base Shear
1	20	54	83	110	0	267
2	91	239	371	490	0	1191
3	122	319	495	655	0	1591
4	122	319	495	655	0	1591
7	150	392	608	804	0	1954
10	488	1278	1984	2624	0	6374
11	529	1385	2150	2842	0	6905
12,13,14	432	1132	1757	2323	0	5645

Shearwall lengths -

Grid	Wall 1 (longer) length (ft)		Wall 2 (shorter) length (ft)		Total Shear Wall Length (ft)
	Shear Wall Length	Tributary Length	Shear Wall Length	Tributary Length	
1	22	50	0	0	22
2	22	22	20	20	42
3	20	20	20	20	40
4	20	20	20	20	40
7	25	25	24	24	48
10	108	133	25	25	133
11	162	129	0	0	162
12,13,14	54	38	41	25	95
	0	0	0	0	0
	0	0	0	0	0
	0	0	0	0	0
	0	0	0	0	0



Total Individual Wall Shear



W/O OVERSTRENGTH

Individual Wall Cumulative Shearwall Shear (lbs)

	Shearwall 1 (longer of 2 cases)					**Shearwall 2 (shorter of 2 cases)**				
Grid	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	5th floor (F5)	Roof (FR)
1	267	247	193	110	0	0	0	0	0	0
2	624	576	451	257	0	567	524	410	234	0
3	793	732	573	326	0	798	737	577	329	0
4	796	735	575	327	0	796	735	575	327	0
7	998	921	721	411	0	956	883	691	394	0
10	5366	4955	3879	2208	0	1009	931	729	415	0
11	6905	6376	4992	2842	0	0	0	0	0	0
12,13,14	3398	3138	2457	1399	0	2247	2075	1624	925	0

W/ OVERSTRENGTH

Individual Wall Cumulative Shearwall Shear (lbs)

	Shearwall 1 (longer of 2 cases)					**Shearwall 2 (shorter of 2 cases)**				
Grid	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet	2nd floor (F2)	3rd floor (F3)	4th floor (F4)	Roof	Parapet
1	267	247	193	110	0	0	0	0	0	0
2	624	576	451	257	0	567	524	410	234	0
3	793	732	573	326	0	798	737	577	329	0
4	796	735	575	327	0	796	735	575	327	0
7	998	921	721	411	0	956	883	691	394	0
10	5366	4955	3879	2208	0	1009	931	729	415	0

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8889	2680	0.302	PASS
Chord capacity	lb/in^2	1265	63	0.100	PASS
Deflection	in	0.268	0.099	0.371	PASS

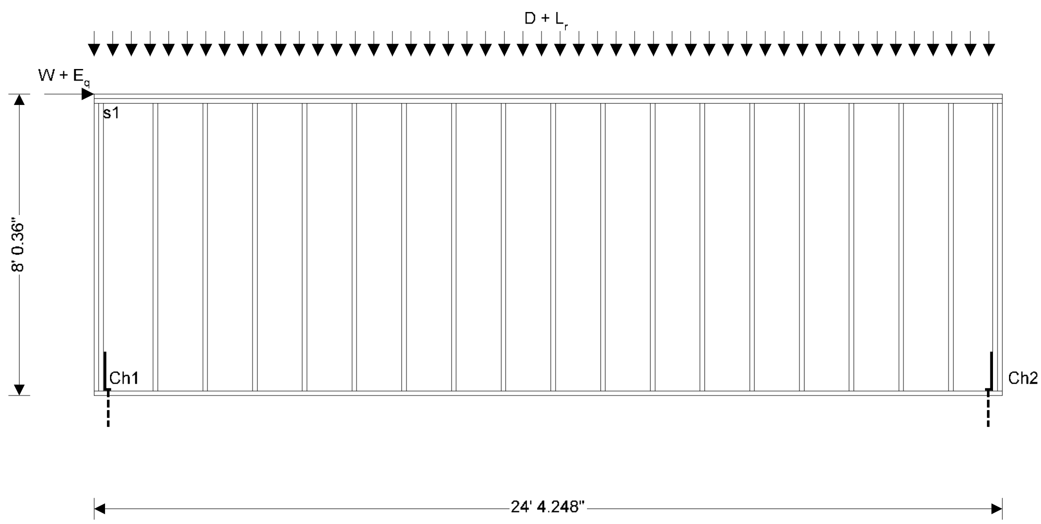
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.03** ft

Panel length; b = **24.354** ft

Total area of wall; A = h × b = **195.563** 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

Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **25** lb/ft

Roof live load acting on top of panel; L_r = **20** lb/ft

Self weight of panel; S_{wt} = **25** lb/ft^2

In plane wind load acting at head of panel; W = **4467** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **329** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1553** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.53**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.33**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **2.68** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.889** kips

V_{w_max} / V_w = **0.302**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.23** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.332** kips

V_{s_max} / V_s = **0.036**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b = **0.33**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.68** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.649** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-0.766** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-57** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.047**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **2.68** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.15** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **1.034** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **63** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **1265** lb/in^2

f_c / F_c' = **0.050**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.100**

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

Wind load deflection

Design shear force; $V_{\delta w}$ = f_{Wserv} × W = **4.467** kips

Deflection limit; Δ_{w_allow}= h / 360 = **0.268** in

Induced unit shear; $v_{\delta w}$ = $V_{\delta w}$ / b = **183.42** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta w}$ × h - 0.6 × (D + S_{wt} × h) × b / 2) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** 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_a) + h × Δ_a / b = **0.099** in

δ_{sww} / Δ_{w_allow} = **0.371**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s}$ = E_q = **0.329** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **1.927** in

Induced unit shear; $v_{\delta s}$ = $V_{\delta s}$ / b = **13.51** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s}$ × h - (0.6 - 0.2 × S_{DS}) × (D + S_{wt} × h) × b / 2) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; δ_{swse} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b = **0.007** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; δ_{sws} = $C_{d\delta}$ × δ_{swse} / I_e = **0.029** in

δ_{sws} / Δ_{s_allow} = **0.015**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8889	3890	0.438	PASS
Chord capacity	lb/in^2	1252	150	0.240	PASS
Deflection	in	0.270	0.148	0.550	PASS

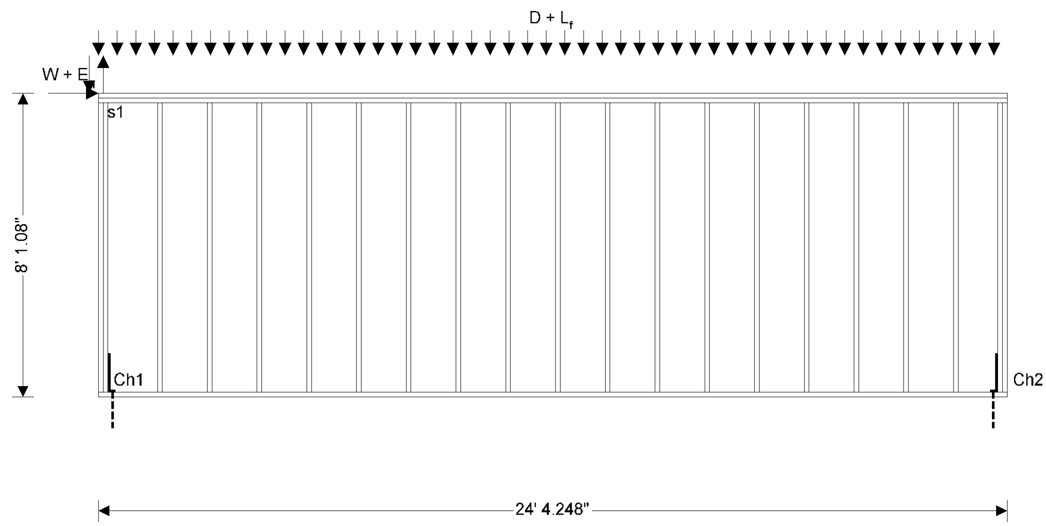
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **24.354** ft

Total area of wall; $A = h \times b$ = **197.024** 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
Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **25** lb/ft
Floor live load acting on top of panel; L_f = **40** lb/ft
Self weight of panel; S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel; W = **6483** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **578** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-1473;	-108;	150;	2749;	0;	13;	0;	0;
Ch2;	1473;	108;	150;	2749;	0;	13;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1530** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.53**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.332**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **3.89** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.889** kips

V_{w_max} / V_w = **0.438**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.405** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.332** kips

V_{s_max} / V_s = **0.064**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.332**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **3.89** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **2.426** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-1.134** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-84** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.070**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **3.89** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **1.186** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **2.478** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **150** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.120**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.240**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.89** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **2.426** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **-1.134** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-84** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.070**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **3.89** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **1.186** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **2.478** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **150** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.120**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.240**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **6.483** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.27** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **266.2** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **0.317** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.009** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.148** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.55**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.578** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **1.942** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **23.73** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.013** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.052** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.027**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8889	5195	0.584	PASS
Chord capacity	lb/in^2	1252	176	0.282	PASS
Deflection	in	0.270	0.204	0.755	PASS

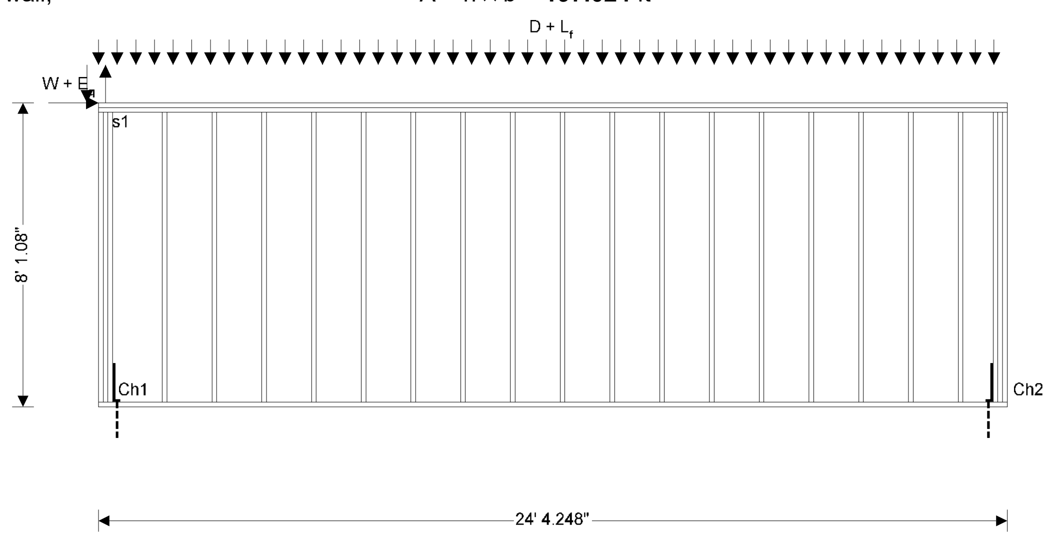
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **24.354** ft

Total area of wall; A = h × b = **197.024** 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; 3 x 2" x 6"

Dressed end post size; 3 x 1.5" x 5.5"

Cross-sectional area of end posts; A_e = **24.75** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **20.25** 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

Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **25** lb/ft

Floor live load acting on top of panel; L_f = **40** lb/ft

Self weight of panel; S_{wt} = **25** lb/ft^2

In plane wind load acting at head of panel; W = **8658** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **739** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-3626;	-300;	302;	5516;	27;	13;	0;	0;
Ch2;	3626;	300;	302;	5516;	27;	13;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1530** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.53**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.332**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **5.195** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.889** kips

V_{w_max} / V_w = **0.584**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.517** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.332** kips

V_{s_max} / V_s = **0.082**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.332**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.195** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **2.794** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-1.069** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-53** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.044**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **5.195** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1}$ = **2.629** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **4.355** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **176** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.141**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.282**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.195** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **2.794** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **-1.069** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-53** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.044**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **5.195** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **2.629** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **4.355** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **176** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.141**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.282**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **8.658** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.27** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **355.51** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **1.532** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.031** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.204** in

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

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.739** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **1.942** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **30.34** lb/ft

Anchor tension force;	T_{δ} = max(0 kips,$v_{\delta s} \times h$ - (0.6 - 0.2 $\times S_{DS}) \times (D + S_{wt} \times h) \times b$ / 2 - (0.6 - 0.2 $\times S_{DS}) \times$ min(D_{T_ch1},D_{T_ch2}) + max(abs(E_{q_ch1}),abs(E_{q_ch2}))) = **0.000** kips
Vertical elongation at anchor;	$\Delta_a = T_{\delta} / k_a$ = **0.000** in
Shear wall elastic deflection – Eqn. 4.3-1;	$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b$ = **0.016** in
Deflection ampification factor;	$C_{d\delta}$ = **4**
Seismic importance factor;	I_e = **1**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15;	$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e$ = **0.066** in
	$\delta_{sws} / \Delta_{s\_allow}$ = **0.034**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8889	6500	0.731	PASS
Chord capacity	lb/in^2	1252	220	0.353	PASS
Deflection	in	0.270	0.258	0.957	PASS

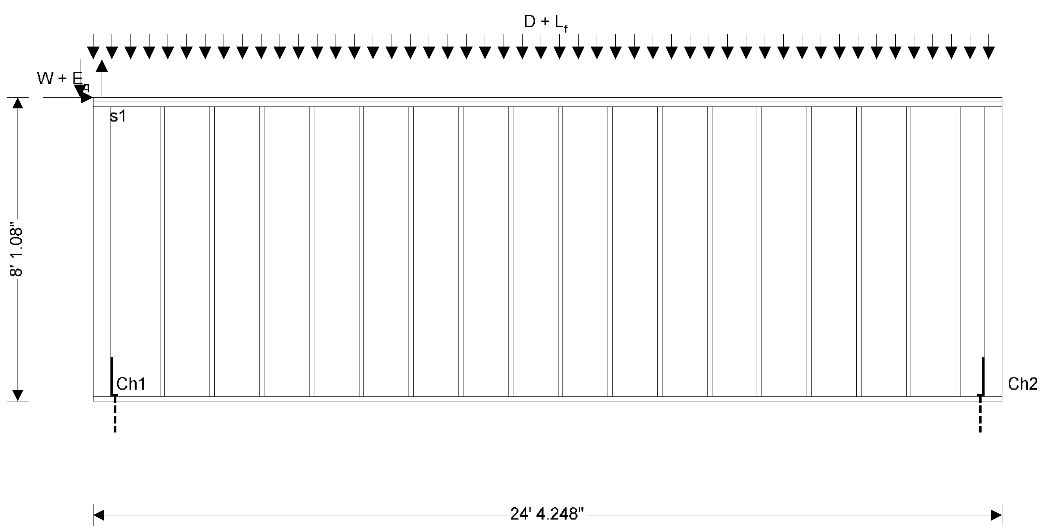
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **24.354** ft

Total area of wall; A = h $\times$ b = **197.024** 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; 6" x 6"

Dressed end post size; 5.5" x 5.5"

Cross-sectional area of end posts; A_e = **30.25** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **24.75** 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

Vertical anchor stiffness; k_a = **69464** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **25** lb/ft

Floor live load acting on top of panel; L_f = **40** lb/ft

Self weight of panel; S_{wt} = **25** lb/ft^2

In plane wind load acting at head of panel; W = **10834** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **815** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-6502;	-546;	453;	8283;	53;	13;	0;	0;
Ch2;	6502;	546;	453;	8283;	53;	13;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1530** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.53**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.332**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **6.5** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.889** kips

V_{w_max} / V_w = **0.731**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.571** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.332** kips

V_{s_max} / V_s = **0.09**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.332**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **6.5** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **2.729** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-0.570** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-23** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.019**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **6.5** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1}$ = **4.506** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **6.666** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **220** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.176**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.353**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.5** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **2.729** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **-0.570** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-23** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.019**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **6.5** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **4.506** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **6.666** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **220** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.176**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.353**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **10.834** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.27** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **444.86** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **3.471** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.050** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.258** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.957**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.815** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **1.942** in

Induced unit shear;	$v_{\delta s} = V_{\delta s} / b =$ **33.46** lb/ft
Anchor tension force;	$T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2})))$ = **0.000** kips
Vertical elongation at anchor;	$\Delta_a = T_\delta / k_a =$ **0.000** in
Shear wall elastic deflection – Eqn. 4.3-1;	$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.018** in
Deflection ampification factor;	$C_{d\delta} =$ **4**
Seismic importance factor;	$I_e =$ **1**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15;	$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.073** in
	$\delta_{sws} / \Delta_{s\_allow} =$ **0.037**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9437	2333	0.247	PASS
Chord capacity	lb/in^2	584	78	0.134	PASS
Deflection	in	0.270	0.084	0.311	PASS

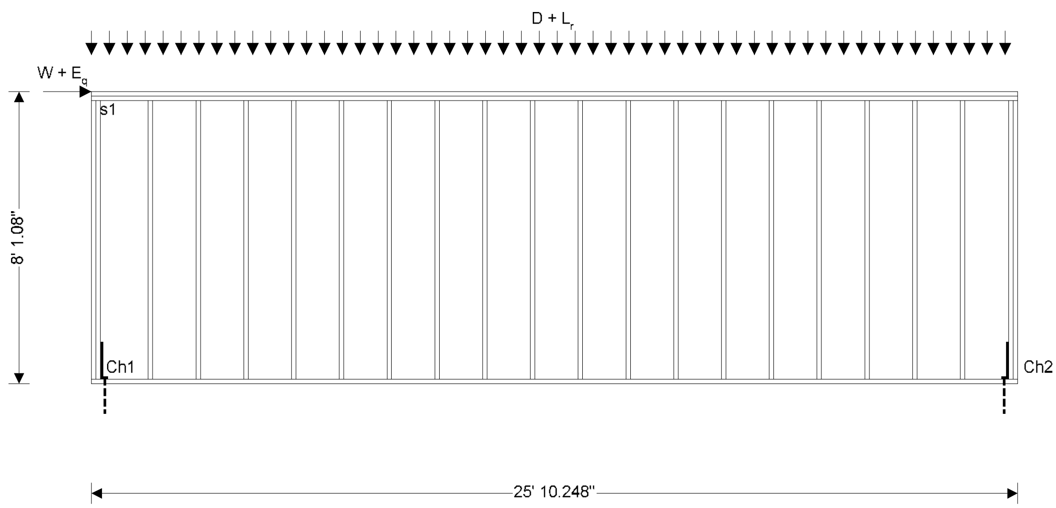
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **25.854** ft

Total area of wall; A = h × b = **209.159** 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

Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

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

Roof live load acting on top of panel; L_r = **40** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **3889** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **596** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **620** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.24**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **25.854** ft

Shear wall aspect ratio; h / b = **0.313**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **2.333** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **9.437** kips

V_{w_max} / V_w = **0.247**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.417** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.722** kips

V_{s_max} / V_s = **0.062**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b = **0.313**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.333** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.078** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-0.348** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-46** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.034**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **2.333** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.093** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **0.823** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **78** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **584** lb/in^2

f_c / F_c' = **0.134**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.125**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **3.889** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.27** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **150.42** lb/ft

Anchor tension force; $T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.139** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a =$ **0.004** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.084** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.311**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.596** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **1.942** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **23.05** lb/ft

Anchor tension force; $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.013** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.051** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.026**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9437	3273	0.347	PASS
Chord capacity	lb/in^2	432	203	0.469	PASS
Deflection	in	0.316	0.147	0.464	PASS

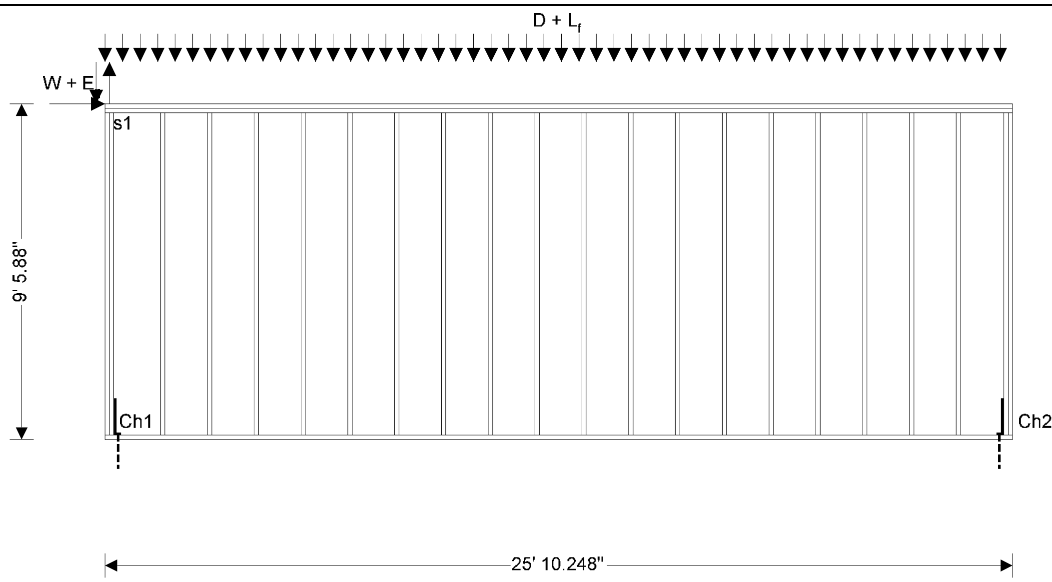
Panel details

Structural wood panel sheathing on one side

Panel height; $h =$ **9.49** ft

Panel length; $b =$ **25.854** ft

Total area of wall; $A = h \times b =$ **245.354** 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
Vertical anchor stiffness;	k_a = **34943** lb/in

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

Species, grade and size classification;	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity;	G = **0.50**
Tension parallel to grain;	F_t = **575** lb/in^2
Compression parallel to grain;	F_c = **1350** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Modulus of elasticity;	E = **1600000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material;	7/16" wood panel oriented strandboard sheathing
Fastener type;	8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

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

Floor live load acting on top of panel; L_f = **80** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **5455** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **1047** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-1217;	-186;	93;	1797;	0;	27;	0;	0;
Ch2;	1217;	186;	93;	1797;	0;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **25.854** ft

Shear wall aspect ratio; h / b = **0.367**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **3.273** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **9.437** kips

V_{w_max} / V_w = **0.347**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.733** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.722** kips

V_{s_max} / V_s = **0.109**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.367**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **3.273** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **1.545** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-0.344** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-46** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.033**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **3.273** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1}$ = **0.926** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **2.127** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **203** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.469**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.324**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **3.273** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2}$ = **1.545** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-0.344** kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-46}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.033}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W = \mathbf{3.273}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} = \mathbf{0.926}$ kips

Maximum compressive force in chord; $C = V \times h / b + P = \mathbf{2.127}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{203}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{432}$ lb/in^2

$f_c / F_c' = \mathbf{0.469}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.324}$

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W = \mathbf{5.455}$ kips

Deflection limit; $\Delta_{w\_allow} = h / 360 = \mathbf{0.316}$ in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b = \mathbf{210.99}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) = \mathbf{0.944}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.027}$ in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.147}$ in

$\delta_{sww} / \Delta_{w\_allow} = \mathbf{0.464}$

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q = \mathbf{1.047}$ kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h = \mathbf{2.278}$ in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b = \mathbf{40.5}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.026}$ in

Deflection ampification factor; $C_{d\delta} = \mathbf{4}$

Seismic importance factor; $I_e = \mathbf{1}$

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.105}$ in

$\delta_{sws} / \Delta_{s\_allow} = \mathbf{0.046}$

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9437	4288	0.454	PASS
Chord capacity	lb/in^2	432	362	0.838	PASS
Deflection	in	0.316	0.204	0.645	PASS

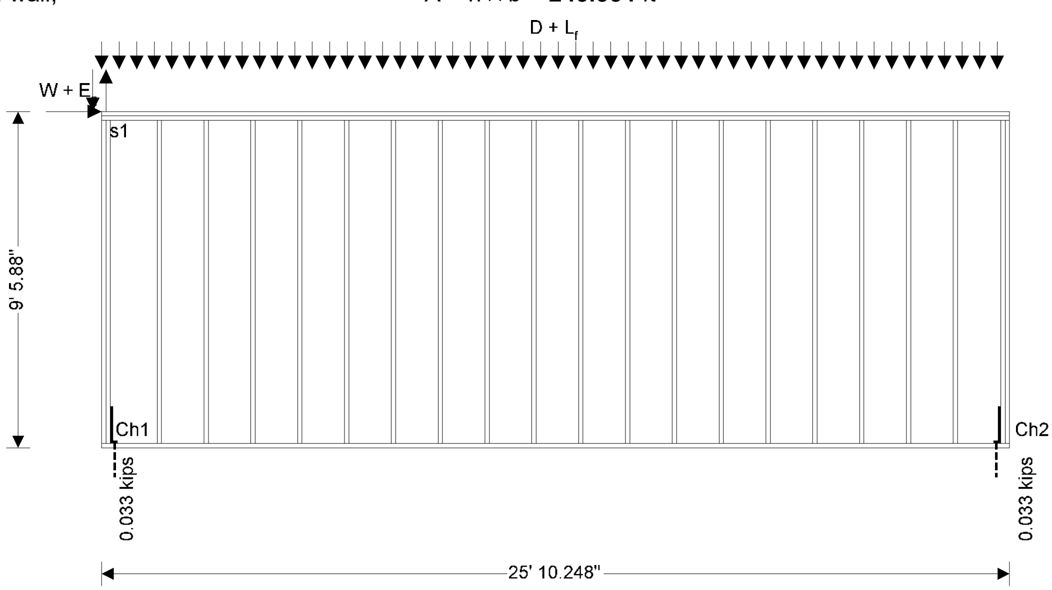
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **25.854** ft

Total area of wall; A = h × b = **245.354** 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
Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **50** lb/ft
Floor live load acting on top of panel; L_f = **80** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **7147** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **1338** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-3219;	-571;	196;	3793;	53;	27;	0;	0;
Ch2;	3219;	571;	196;	3793;	53;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **25.854** ft

Shear wall aspect ratio; h / b = **0.367**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **4.288** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **9.437** kips

V_{w_max} / V_w = **0.454**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.937** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.722** kips

V_{s_max} / V_s = **0.139**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.367**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **4.288** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **1.541** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **0.033** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **4** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.003**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **4.288** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1}$ = **2.23** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **3.804** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **362** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.838**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.580**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **4.288** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **1.541** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **0.033** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **4** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **0.003**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **4.288** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **2.23** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **3.804** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **362** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.838**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.580**

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

Hold down force

Chord 1; $T_1 =$ **0.033** kips

Chord 2; $T_2 =$ **0.033** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-5843;	-1062;	299;	5788;	107;	27;	0;	0;
Ch2;	5843;	1062;	299;	5788;	107;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **7.147** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **276.44** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **2.370** kips

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

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.204** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.645**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **1.338** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **51.75** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.034** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.134** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.059**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9437	5303	0.562	PASS
Chord capacity	lb/in^2	432	304	0.704	PASS
Deflection	in	0.316	0.242	0.767	PASS

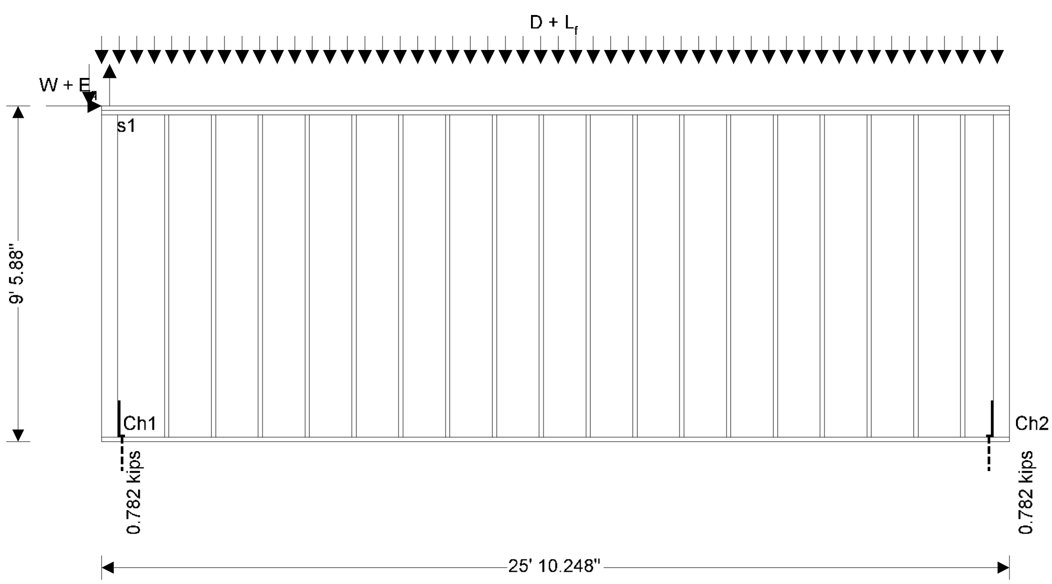
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **25.854** ft

Total area of wall; A = h × b = **245.354** 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; 6" x 4"

Dressed end post size; 5.5" x 3.5"

Cross-sectional area of end posts; A_e = **19.25** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **13.75** 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
Vertical anchor stiffness; k_a = **69646** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **50** lb/ft
Floor live load acting on top of panel; L_f = **80** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **8838** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **1449** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-5843;	-1062;	299;	5788;	107;	27;	0;	0;
Ch2;	5843;	1062;	299;	5788;	107;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **25.854** ft

Shear wall aspect ratio; h / b = **0.367**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **5.303** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **9.437** kips

V_{w_max} / V_w = **0.562**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **1.014** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.722** kips

V_{s_max} / V_s = **0.151**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.367**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.303** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **1.165** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **0.782** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **57** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.041**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **5.303** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1}$ = **3.907** kips
Maximum compressive force in chord; $C = V \times h / b + P$ = **5.853** kips
Maximum applied compressive stress; $f_c = C / A_e$ = **304** lb/in^2
Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2
f_c / F_c' = **0.704**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2
f_c / F_{c_perp}' = **0.487**

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

Chord capacity for chord 2

Load combination 5
Shear force for maximum tension; $V = 0.6 \times W$ = **5.303** kips
Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2}$ = **1.165** kips
Maximum tensile force in chord; $T = V \times h / b - P$ = **0.782** kips
Maximum applied tensile stress; $f_t = T / A_{en}$ = **57** lb/in^2
Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2
f_t / F_t' = **0.041**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1
Shear force for maximum compression; $V = 0.6 \times W$ = **5.303** kips
Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2}$ = **3.907** kips
Maximum compressive force in chord; $C = V \times h / b + P$ = **5.853** kips
Maximum applied compressive stress; $f_c = C / A_e$ = **304** lb/in^2
Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2
f_c / F_c' = **0.704**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2
f_c / F_{c_perp}' = **0.487**

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

Hold down force

Chord 1; T_1 = **0.782** kips
Chord 2; T_2 = **0.782** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-9087;	-1594;	401;	7784;	160;	27;	0;	0;
Ch2;	9087;	1594;	401;	7784;	160;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **8.838** kips
Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in
Induced unit shear; $v_{\delta w} = V_{\delta w} / b$ = **341.84** lb/ft

Anchor tension force; $T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **4.416** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a =$ **0.063** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.242** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.767**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **1.449** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **56.05** lb/ft

Anchor tension force; $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.036** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.144** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.063**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9437	3115	0.330	PASS
Chord capacity	lb/in^2	584	68	0.116	PASS
Deflection	in	0.270	0.113	0.419	PASS

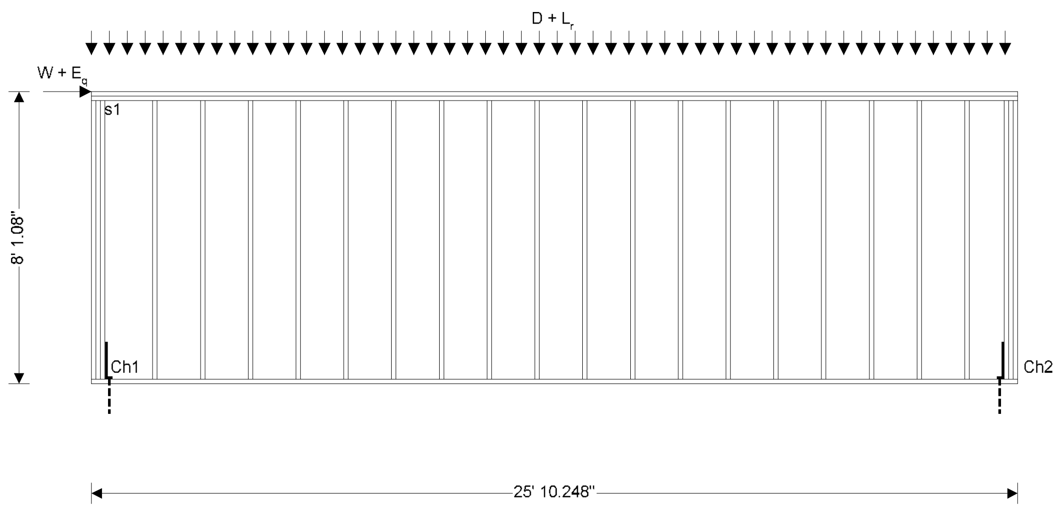
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **25.854** ft

Total area of wall; A = h × b = **209.159** 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; 3 x 2" x 4"

Dressed end post size; 3 x 1.5" x 3.5"

Cross-sectional area of end posts; A_e = **15.75** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **11.25** 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

Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **50** lb/ft
Roof live load acting on top of panel; L_r = **40** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **5191** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **978** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

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;	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi
Critical buckling design value;	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **620** psi
Reference compression design value;	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi
For sawn lumber;	c = **0.8**
Column stability factor – eqn.3.7-1;	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.24**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio;	3.5
Shear wall length;	b = **25.854** ft
Shear wall aspect ratio;	h / b = **0.313**

Segmented shear wall capacity

Maximum shear force under wind loading;	$V_{w\_max} = 0.6 \times W$ = **3.115** kips
Shear capacity for wind loading;	$V_w = v_w \times b / 2$ = **9.437** kips
	V_{w_max} / V_w = **0.33**
	PASS - Shear capacity for wind load exceeds maximum shear force
Maximum shear force under seismic loading;	$V_{s\_max} = 0.7 \times E_q$ = **0.685** kips
Shear capacity for seismic loading;	$V_s = v_s \times b / 2$ = **6.722** kips
	V_{s_max} / V_s = **0.102**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio;	h / b = **0.313**
Load combination 5	
Shear force for maximum tension;	$V = 0.6 \times W$ = **3.115** kips
Axial force for maximum tension;	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.078** kips
Maximum tensile force in chord;	$T = V \times h / b - P$ = **-0.103** kips
Maximum applied tensile stress;	$f_t = T / A_{en}$ = **-9** lb/in^2
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2
	f_t / F_t' = **-0.007**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression;	$V = 0.6 \times W$ = **3.115** kips
Axial force for maximum compression;	$P = ((D + S_{wt} \times h)) \times s / 2$ = **0.093** kips
Maximum compressive force in chord;	$C = V \times h / b + P$ = **1.067** kips
Maximum applied compressive stress;	$f_c = C / A_e$ = **68** lb/in^2
Design compressive stress;	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **584** lb/in^2
	f_c / F_c' = **0.116**
	PASS - Design compressive stress exceeds maximum applied compressive stress
Design bearing compr. stress, bottom plate;	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2
	f_c / F_{c_perp}' = **0.108**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W = \mathbf{5.191}$ kips

Deflection limit; $\Delta_{w\_allow} = h / 360 = \mathbf{0.27}$ in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b = \mathbf{200.78}$ lb/ft

Anchor tension force; $T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.546}$ kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a = \mathbf{0.011}$ in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.113}$ in

$\delta_{sww} / \Delta_{w\_allow} = \mathbf{0.419}$

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q = \mathbf{0.978}$ kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h = \mathbf{1.942}$ in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b = \mathbf{37.83}$ lb/ft

Anchor tension force; $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a = \mathbf{0.000}$ in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.021}$ in

Deflection ampification factor; $C_{d\delta} = \mathbf{4}$

Seismic importance factor; $I_e = \mathbf{1}$

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.083}$ in

$\delta_{sws} / \Delta_{s\_allow} = \mathbf{0.043}$

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9437	4364	0.462	PASS
Chord capacity	lb/in^2	432	176	0.407	PASS
Deflection	in	0.316	0.196	0.620	PASS

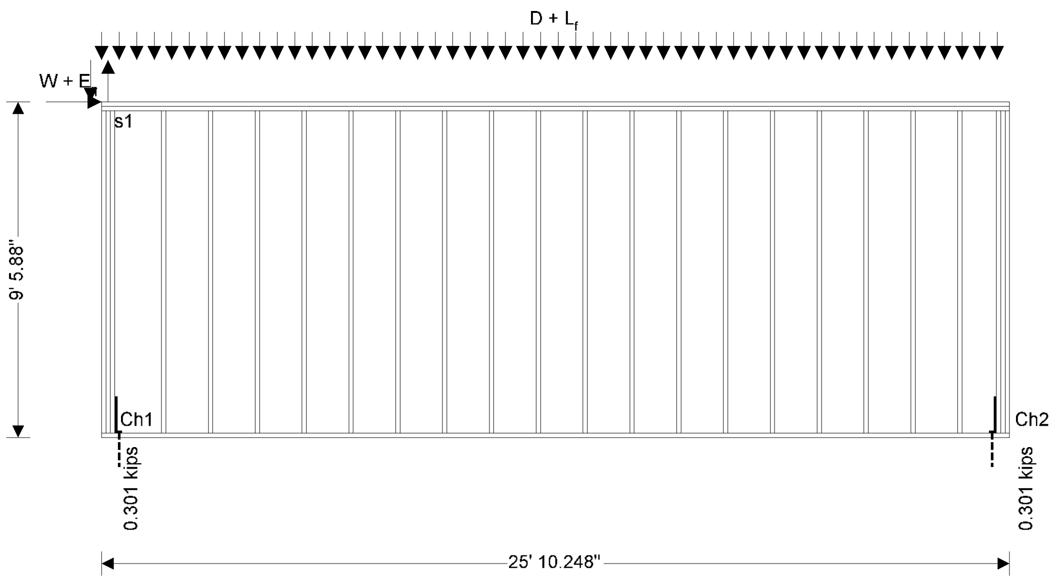
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **25.854** ft

Total area of wall; A = h × b = **245.354** 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; 3 x 2" x 4"

Dressed end post size; 3 x 1.5" x 3.5"

Cross-sectional area of end posts; A_e = **15.75** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **11.25** 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

Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

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

Floor live load acting on top of panel; L_f = **80** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **7273** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **1718** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-1624;	-306;	93;	1797;	0;	27;	0;	0;
Ch2;	1624;	306;	93;	1797;	0;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **25.854** ft

Shear wall aspect ratio; h / b = **0.367**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **4.364** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **9.437** kips

V_{w_max} / V_w = **0.462**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **1.203** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.722** kips

V_{s_max} / V_s = **0.179**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.367**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **4.364** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **1.301** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **0.301** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **27** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.019**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **4.364** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **1.17** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **2.772** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **176** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.407**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.282**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **4.364** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **1.301** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **0.301** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **27** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **0.019**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **4.364** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **1.17** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **2.772** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **176** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.407**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.282**

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

Hold down force

Chord 1; $T_1 =$ **0.301** kips

Chord 2; $T_2 =$ **0.301** kips

Chord reactions by load type

Chord	**$W_{ch[i]R}$ (lbs)**	**$E_{q\_ch[i]R}$ (lbs)**	**$D_{C\_ch[i]R}$ (lbs)**	**$D_{T\_ch[i]R}$ (lbs)**	**$L_{f\_ch[i]R}$ (lbs)**	**$L_{r\_ch[i]R}$ (lbs)**	**$S_{ch[i]R}$ (lbs)**	**$R_{ch[i]R}$ (lbs)**
Ch1;	-4294;	-937;	196;	3793;	53;	27;	0;	0;
Ch2;	4294;	937;	196;	3793;	53;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **7.273** kips

Deflection limit; Δ_{w_allow}= h / 360 = **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b$ = **281.31** lb/ft

Anchor tension force; $T_\delta = \max(0\text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2})))$ = **2.018** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.041** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b$ = **0.196** in

$\delta_{sww} / \Delta_{w\_allow}$ = **0.62**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q$ = **1.718** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b$ = **66.45** lb/ft

Anchor tension force; $T_\delta = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b$ = **0.043** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e$ = **0.171** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.075**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9437	5717	0.606	PASS
Chord capacity	lb/in^2	432	316	0.731	PASS
Deflection	in	0.316	0.269	0.851	PASS

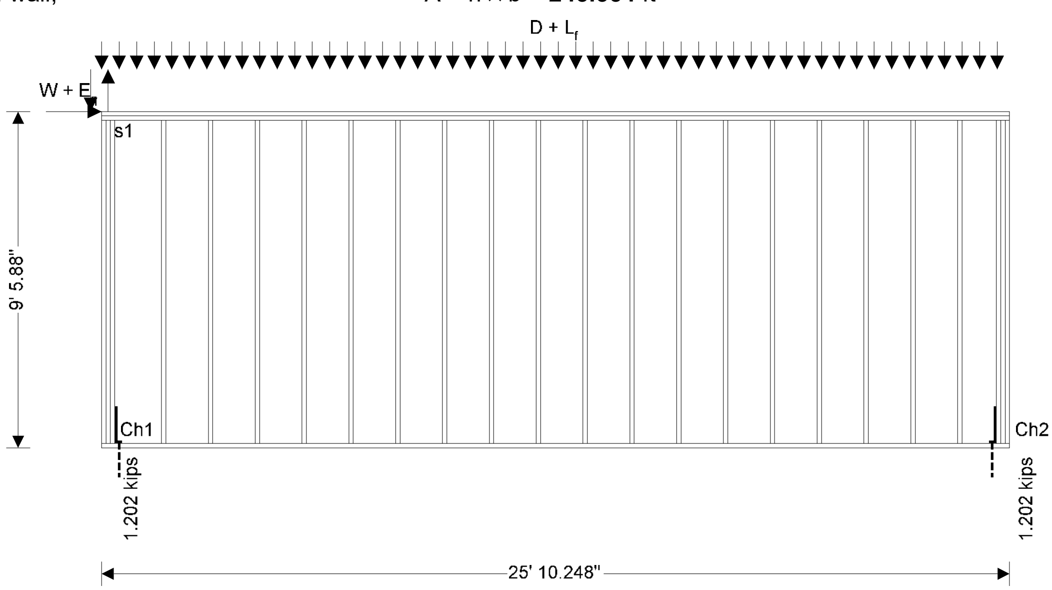
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **25.854** ft

Total area of wall; $A = h \times b =$ **245.354** 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; 3 x 2" x 4"

Dressed end post size; 3 x 1.5" x 3.5"

Cross-sectional area of end posts; A_e = **15.75** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **11.25** 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

Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

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

Floor live load acting on top of panel; L_f = **80** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **9528** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **2195** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-4294;	-937;	196;	3793;	53;	27;	0;	0;
Ch2;	4294;	937;	196;	3793;	53;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **25.854** ft

Shear wall aspect ratio; h / b = **0.367**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **5.717** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **9.437** kips

V_{w_max} / V_w = **0.606**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **1.537** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.722** kips

V_{s_max} / V_s = **0.229**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.367**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.717** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **0.897** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **1.202** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **107** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.077**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **5.717** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1}$ = **2.875** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **4.973** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **316** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.731**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.505**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.717** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **0.897** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **1.202** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **107** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **0.077**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **5.717** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **2.875** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **4.973** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **316** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.731**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.505**

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

Hold down force

Chord 1; $T_1 =$ **1.202** kips

Chord 2; $T_2 =$ **1.202** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-7791;	-1742;	299;	5788;	107;	27;	0;	0;
Ch2;	7791;	1742;	299;	5788;	107;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **9.528** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **368.53** lb/ft

Anchor tension force; $T_{\delta} = \max(0\text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **4.318** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a =$ **0.088** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.269** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.851**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **2.195** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **84.9** lb/ft

Anchor tension force; $T_{\delta} = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.055** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.218** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.096**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	13767	7070	0.514	PASS
Chord capacity	lb/in^2	432	231	0.534	PASS
Deflection	in	0.316	0.227	0.719	PASS

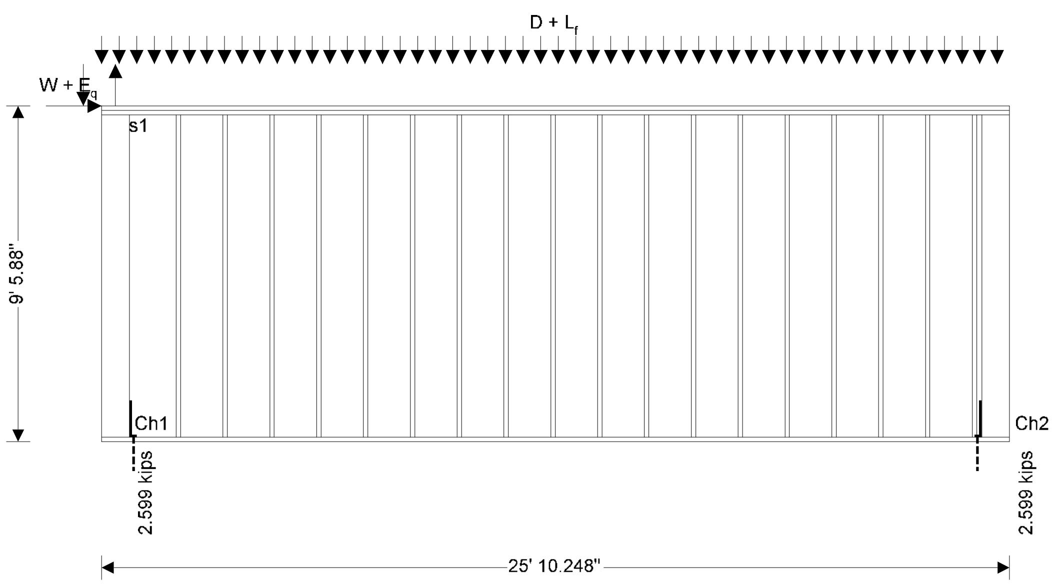
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **25.854** ft

Total area of wall; $A = h \times b$ = **245.354** 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; 10" x 4"

Dressed end post size; 9.5" x 3.5"

Cross-sectional area of end posts; A_e = **33.25** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **23.75** 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
Vertical anchor stiffness; k_a = **95741** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" 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 for seismic design; v_s = **760** lb/ft
Nominal unit shear capacity for wind design; v_w = **1065** lb/ft
Apparent shear wall shear stiffness; G_a = **22** kips/in

Loading details

Dead load acting on top of panel; D = **50** lb/ft
Floor live load acting on top of panel; L_f = **80** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **11783** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **2377** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-7791;	-1742;	299;	5788;	107;	27;	0;	0;
Ch2;	7791;	1742;	299;	5788;	107;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **25.854** ft

Shear wall aspect ratio; h / b = **0.367**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **7.07** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **13.767** kips

V_{w_max} / V_w = **0.514**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **1.664** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **9.825** kips

V_{s_max} / V_s = **0.169**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.367**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **7.07** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **-0.004** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **2.599** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **109** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.079**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **7.07** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1}$ = **5.076** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **7.671** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **231** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.534**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.369**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **7.07** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2}$ = **-0.004** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **2.599** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **109** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.079**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **7.07** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2}$ = **5.076** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **7.671** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **231** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.534**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.369**

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

Hold down force

Chord 1; T_1 = **2.599** kips

Chord 2; T_2 = **2.599** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-12116;	-2615;	401;	7784;	160;	27;	0;	0;
Ch2;	12116;	2615;	401;	7784;	160;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **11.783** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b$ = **455.75** lb/ft

Anchor tension force; $T_\delta = \max(0\text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **7.446** kips

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

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.227** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.719**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **2.377** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **91.94** lb/ft

Anchor tension force; $T_\delta = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.04** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.16** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.07**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8889	3176	0.357	PASS
Chord capacity	lb/in^2	584	109	0.187	PASS
Deflection	in	0.270	0.127	0.469	PASS

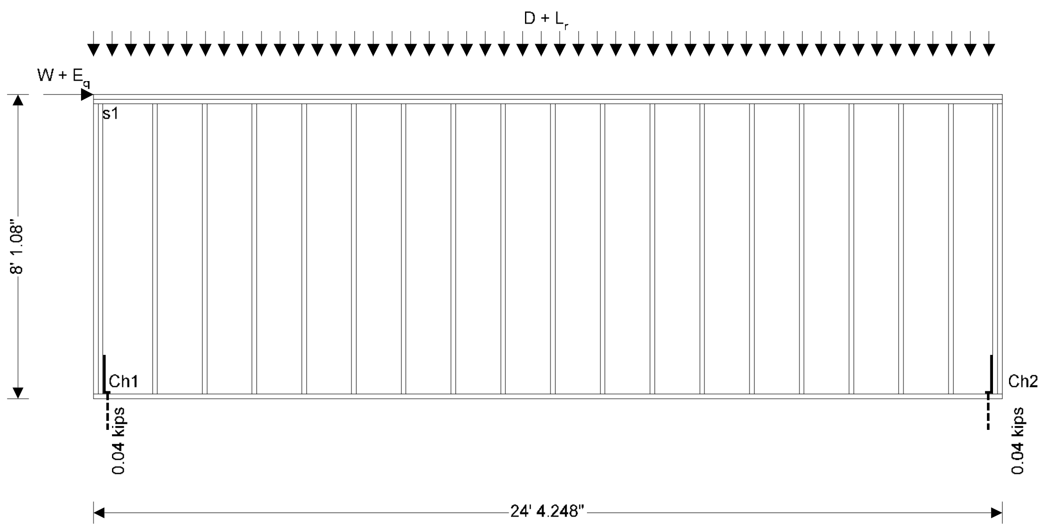
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **24.354** ft

Total area of wall; A = h × b = **197.024** 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

Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

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

Roof live load acting on top of panel; L_r = **40** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **5294** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **411** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **620** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.24**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.332**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **3.176** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.889** kips

V_{w_max} / V_w = **0.357**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.288** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.332** kips

V_{s_max} / V_s = **0.045**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b = **0.332**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **3.176** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.015** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **0.040** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **5** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.004**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **3.176** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.093** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **1.148** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **109** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **584** lb/in^2

f_c / F_c' = **0.187**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.175**

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

Hold down force

Chord 1; T_1 = **0.04** kips

Chord 2; T_2 = **0.04** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-1759;	-137;	93;	1692;	0;	27;	0;	0;
Ch2;	1759;	137;	93;	1692;	0;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **5.294** kips

Deflection limit; Δ_{w_allow}= h / 360 = **0.27** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b$ = **217.38** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.743** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.021** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b$ = **0.127** in

$\delta_{sww} / \Delta_{w\_allow}$ = **0.469**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q$ = **0.411** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **1.942** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b$ = **16.88** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b$ = **0.009** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e$ = **0.037** in

$\delta_{sws} / \Delta_{s\_allow}$ = **0.019**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8889	4456	0.501	PASS
Chord capacity	lb/in^2	432	284	0.658	PASS
Deflection	in	0.316	0.226	0.714	PASS

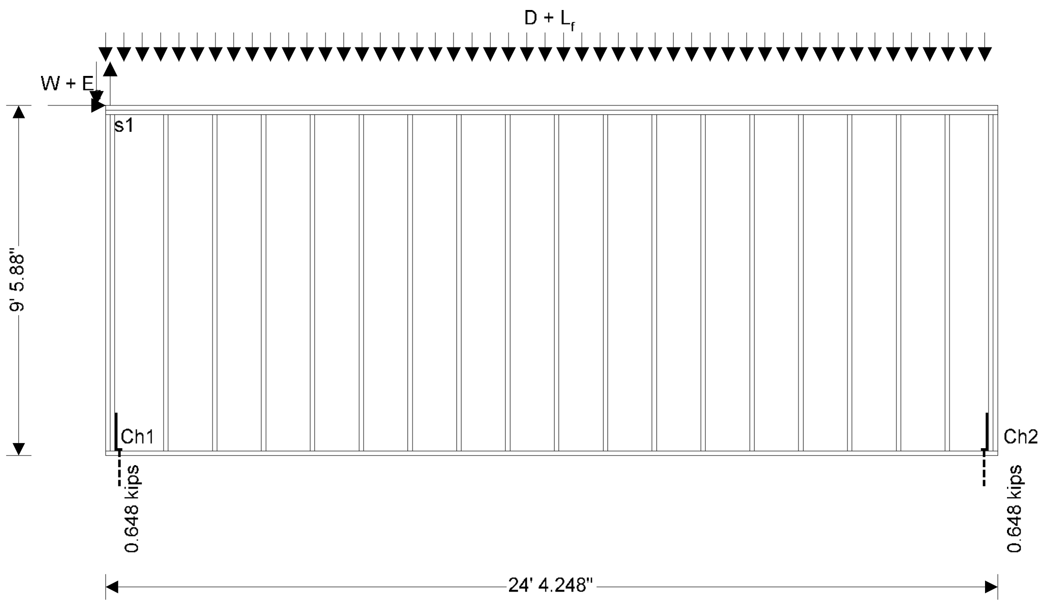
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **24.354** ft

Total area of wall; A = h × b = **231.119** 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

Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

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

Floor live load acting on top of panel; L_f = **80** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **7427** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **721** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-1759;	-137;	93;	1692;	0;	27;	0;	0;
Ch2;	1759;	137;	93;	1692;	0;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.39**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **4.456** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.889** kips

V_{w_max} / V_w = **0.501**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.505** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.332** kips

V_{s_max} / V_s = **0.08**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.39**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **4.456** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **1.088** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **0.648** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **86** lb/in²

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in²

f_t / F_t' = **0.063**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 × W = **4.456** kips

Axial force for maximum compression; P = ((D + S_{wt} × h)) × s / 2 + -1 × 0.6 × W_{ch1} + D_{C_ch1} = **1.251** kips

Maximum compressive force in chord; C = V × h / b + P = **2.987** kips

Maximum applied compressive stress; f_c = C / A_e = **284** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **432** lb/in^2

f_c / F_c' = **0.658**

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 = **625** lb/in^2

f_c / F_{c_perp}' = **0.455**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **4.456** kips

Axial force for maximum tension; P = (0.6 × (D + S_{wt} × h)) × b_1 / 2 + -1 × 0.6 × W_{ch2} + 0.6 × D_{T_ch2} = **1.088** kips

Maximum tensile force in chord; T = V × h / b - P = **0.648** kips

Maximum applied tensile stress; f_t = T / A_{en} = **86** lb/in^2

Design tensile stress; F_t' = F_t × C_D × C_{Mt} × C_{tt} × C_{Ft} × C_i = **1380** lb/in^2

f_t / F_t' = **0.063**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 × W = **4.456** kips

Axial force for maximum compression; P = ((D + S_{wt} × h)) × s / 2 + 0.6 × W_{ch2} + D_{C_ch2} = **1.251** kips

Maximum compressive force in chord; C = V × h / b + P = **2.987** kips

Maximum applied compressive stress; f_c = C / A_e = **284** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **432** lb/in^2

f_c / F_c' = **0.658**

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 = **625** lb/in^2

f_c / F_{c_perp}' = **0.455**

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

Hold down force

Chord 1; T_1 = **0.648** kips

Chord 2; T_2 = **0.648** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-4653;	-417;	196;	3572;	53;	27;	0;	0;
Ch2;	4653;	417;	196;	3572;	53;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **7.427** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **304.96** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **2.509** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.072** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.226** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.714**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.721** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **29.6** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.019** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.077** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.034**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8889	5837	0.657	PASS
Chord capacity	lb/in^2	432	341	0.788	PASS
Deflection	in	0.316	0.298	0.943	PASS

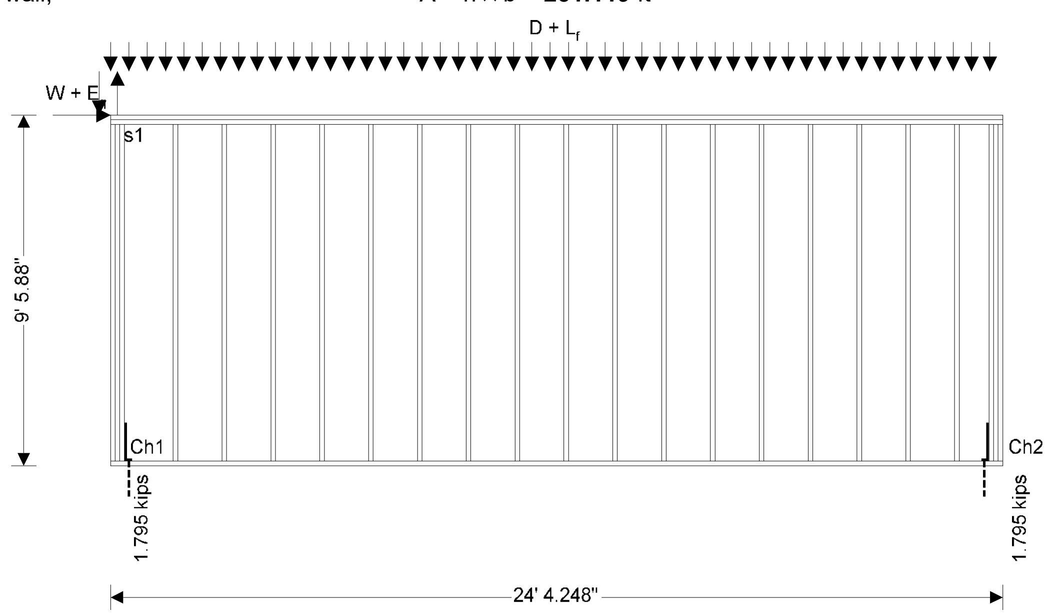
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **24.354** ft

Total area of wall; A = h × b = **231.119** 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; 3 x 2" x 4"

Dressed end post size; 3 x 1.5" x 3.5"

Cross-sectional area of end posts; A_e = **15.75** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **11.25** 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
Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **50** lb/ft
Floor live load acting on top of panel; L_f = **80** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **9729** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **921** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-4653;	-417;	196;	3572;	53;	27;	0;	0;
Ch2;	4653;	417;	196;	3572;	53;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.39**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **5.837** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.889** kips

V_{w_max} / V_w = **0.657**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.645** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.332** kips

V_{s_max} / V_s = **0.102**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.39**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.837** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **0.48** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **1.795** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **160** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** 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; $V = 0.6 \times W$ = **5.837** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **3.09** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **5.365** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **341** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.788**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.545**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.837** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **0.48** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **1.795** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **160** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** 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; $V = 0.6 \times W =$ **5.837** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **3.09** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **5.365** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **341** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.788**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.545**

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

Hold down force

Chord 1; $T_1 =$ **1.795** kips

Chord 2; $T_2 =$ **1.795** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-8444;	-776;	299;	5452;	107;	27;	0;	0;
Ch2;	8444;	776;	299;	5452;	107;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **9.729** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **399.48** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) = \mathbf{5.172}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.105}$ in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.298}$ in

$\delta_{sww} / \Delta_{w\_allow} = \mathbf{0.943}$

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q = \mathbf{0.921}$ kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h = \mathbf{2.278}$ in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b = \mathbf{37.82}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.024}$ in

Deflection ampification factor; $C_{d\delta} = \mathbf{4}$

Seismic importance factor; $I_e = \mathbf{1}$

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.097}$ in

$\delta_{sws} / \Delta_{s\_allow} = \mathbf{0.043}$

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	12969	7219	0.557	PASS
Chord capacity	lb/in^2	432	249	0.576	PASS
Deflection	in	0.316	0.251	0.794	PASS

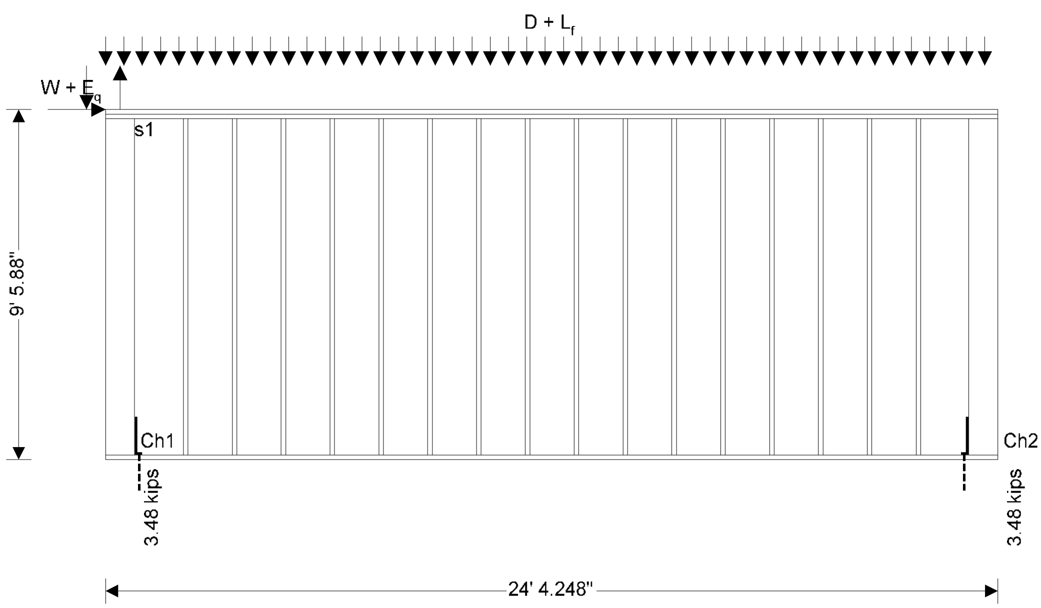
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **24.354** ft

Total area of wall; $A = h \times b =$ **231.119** 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; 10" x 4"

Dressed end post size; 9.5" x 3.5"

Cross-sectional area of end posts; A_e = **33.25** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **23.75** 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
Vertical anchor stiffness; k_a = **95741** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" 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 for seismic design; v_s = **760** lb/ft
Nominal unit shear capacity for wind design; v_w = **1065** lb/ft
Apparent shear wall shear stiffness; G_a = **22** kips/in

Loading details

Dead load acting on top of panel; D = **50** lb/ft
Floor live load acting on top of panel; L_f = **80** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **12032** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **998** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-8444;	-776;	299;	5452;	107;	27;	0;	0;
Ch2;	8444;	776;	299;	5452;	107;	27;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.354** ft

Shear wall aspect ratio; h / b = **0.39**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **7.219** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **12.969** kips

V_{w_max} / V_w = **0.557**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.699** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **9.255** kips

V_{s_max} / V_s = **0.075**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.39**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **7.219** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **-0.667** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **3.480** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **147** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.106**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **7.219** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **5.468** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **8.281** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **249** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.576**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.398**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **7.219** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **-0.667** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **3.480** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **147** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **0.106**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **7.219** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **5.468** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **8.281** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **249** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.576**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.398**

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

Hold down force

Chord 1; $T_1 =$ **3.48** kips

Chord 2; $T_2 =$ **3.48** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-13132;	-1165;	401;	7333;	160;	27;	0;	0;
Ch2;	13132;	1165;	401;	7333;	160;	27;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **12.032** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **494.05** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2}))) =$ **8.733** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.091** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.251** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.794**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.998** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **40.98** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.018** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.072** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.031**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8822	856	0.097	PASS
Chord capacity	lb/in^2	592	39	0.066	PASS
Deflection	in	0.268	0.032	0.120	PASS

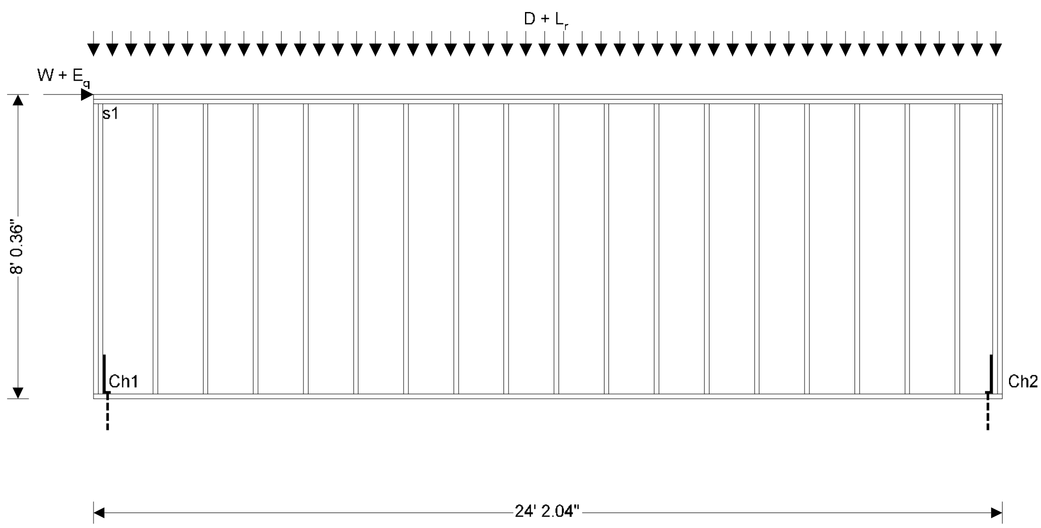
Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.03** ft

Panel length; b = **24.17** ft

Total area of wall; A = h × b = **194.085** 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

Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **97.14** lb/ft

Roof live load acting on top of panel; L_r = **77.708** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **1427** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **415** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **629** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.24**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.17** ft

Shear wall aspect ratio; h / b = **0.332**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **0.856** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.822** kips

V_{w_max} / V_w = **0.097**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.291** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.284** kips

V_{s_max} / V_s = **0.046**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b = **0.332**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **0.856** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.345** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-1.060** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-141** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.102**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **0.856** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.124** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **0.408** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **39** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **592** lb/in^2

f_c / F_c' = **0.066**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.062**

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

Wind load deflection

Design shear force; $V_{\delta w}$ = f_{Wserv} × W = **1.427** kips

Deflection limit; Δ_{w_allow}= h / 360 = **0.268** in

Induced unit shear; $v_{\delta w}$ = $V_{\delta w}$ / b = **59.04** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta w}$ × h - 0.6 × (D + S_{wt} × h) × b / 2) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** 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_a) + h × Δ_a / b = **0.032** in

δ_{sww} / Δ_{w_allow} = **0.12**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s}$ = E_q = **0.415** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **1.927** in

Induced unit shear; $v_{\delta s}$ = $V_{\delta s}$ / b = **17.17** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s}$ × h - (0.6 - 0.2 × S_{DS}) × (D + S_{wt} × h) × b / 2) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; δ_{swse} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b = **0.009** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; δ_{sws} = $C_{d\delta}$ × δ_{swse} / I_e = **0.037** in

δ_{sws} / Δ_{s_allow} = **0.019**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8822	1223	0.139	PASS
Chord capacity	lb/in^2	432	99	0.228	PASS
Deflection	in	0.316	0.055	0.173	PASS

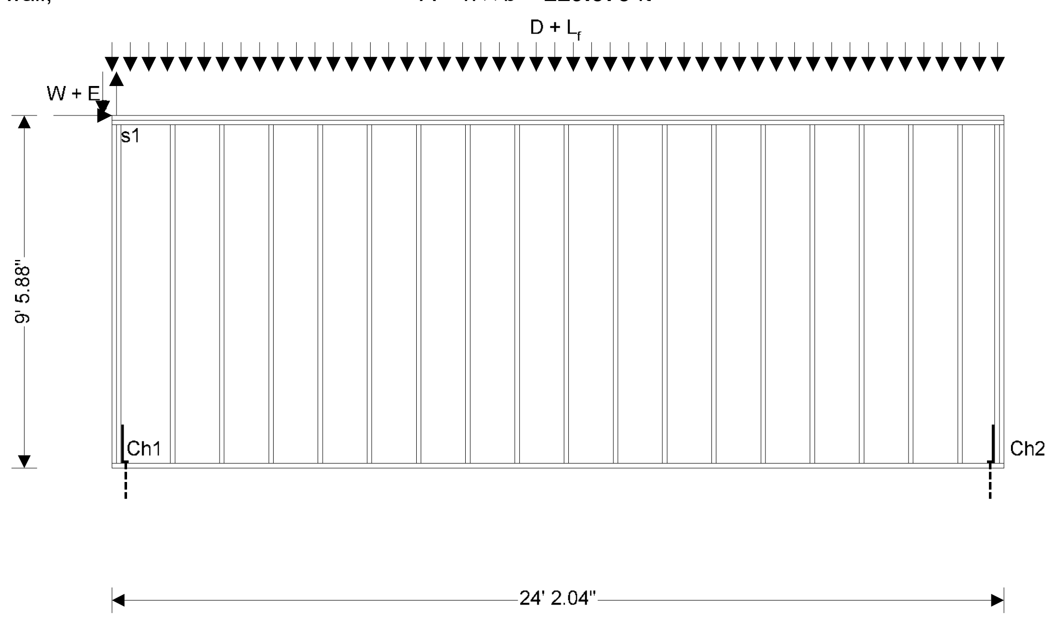
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **24.17** ft

Total area of wall; A = h × b = **229.373** 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

Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider

Specific gravity; G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **97.14** lb/ft

Floor live load acting on top of panel; L_f = **328.54** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **2038** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **729** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-474;	-138;	124;	2241;	0;	52;	0;	0;
Ch2;	474;	138;	124;	2241;	0;	52;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.17** ft

Shear wall aspect ratio; h / b = **0.393**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **1.223** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.822** kips

V_{w_max} / V_w = **0.139**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.51** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.284** kips

V_{s_max} / V_s = **0.081**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.393**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **1.223** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **2.522** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-2.042** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-272** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.197**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **0.917** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + -1 \times 0.45 \times W_{ch1} + D_{C\_ch1} + 0.75 \times L_{r\_ch1}$ = **0.674** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **1.035** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **99** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.228**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.158**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **1.223** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2}$ = **2.522** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-2.042** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-272** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.197**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **0.917** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + 0.45 \times W_{ch2} + D_{C\_ch2} + 0.75 \times L_{r\_ch2}$ = **0.674** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **1.035** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **99** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.228**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.158**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **2.038** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b$ = **84.32** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.055** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.173**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.729** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b =$ **30.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.02** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.078** in

$\delta_{sws} / \Delta_{s\_allow} =$ **0.034**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8822	1619	0.183	PASS
Chord capacity	lb/in^2	432	172	0.399	PASS
Deflection	in	0.316	0.073	0.229	PASS

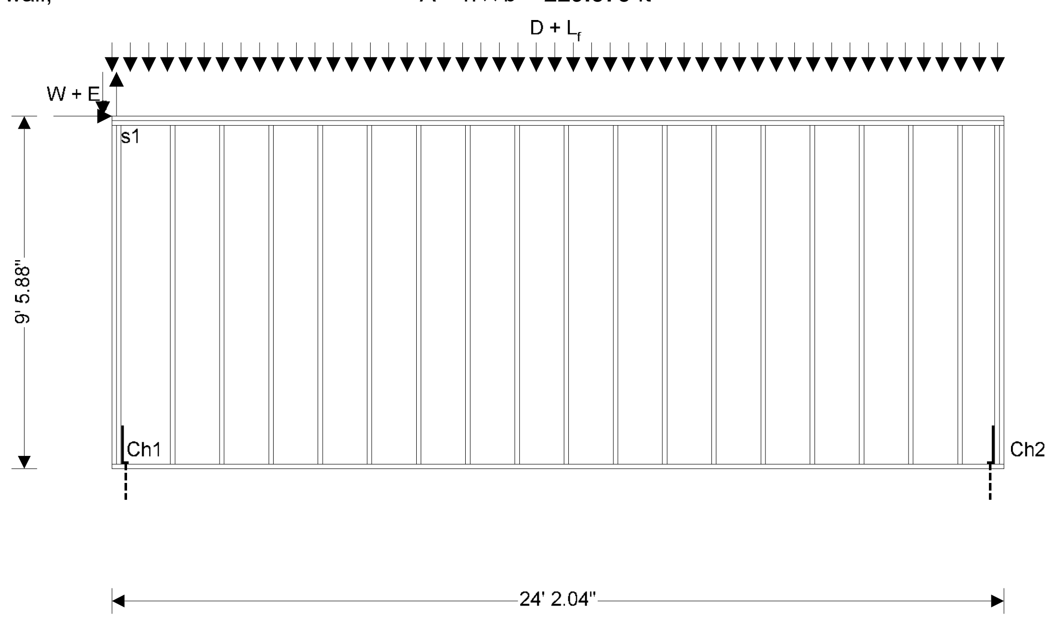
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **24.17** ft

Total area of wall; A = h × b = **229.373** 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
Vertical anchor stiffness; k_a = **34943** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **97.135** lb/ft
Floor live load acting on top of panel; L_f = **328.54** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **2698** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **931** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-1274;	-424;	258;	4677;	219;	52;	0;	0;
Ch2;	1274;	424;	258;	4677;	219;	52;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.17** ft

Shear wall aspect ratio; h / b = **0.393**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **1.619** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.822** kips

V_{w_max} / V_w = **0.183**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.652** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.284** kips

V_{s_max} / V_s = **0.104**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.393**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **1.619** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **3.503** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-2.867** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-382** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.277**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **1.214** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + -1 \times 0.45 \times W_{ch1} + D_{C\_ch1} + 0.75 \times L_{f\_ch1} + 0.75 \times L_{r\_ch1}$ = **1.333** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **1.810** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **172** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.399**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.276**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **1.619** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2}$ = **3.503** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-2.867** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-382** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.277**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **1.214** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + 0.45 \times W_{ch2} + D_{C\_ch2} + 0.75 \times L_{f\_ch2} + 0.75 \times L_{r\_ch2}$ = **1.333** kips

Maximum compressive force in chord; $C = V \times h / b + P$ = **1.810** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **172** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.399**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.276**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **2.698** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b$ = **111.63** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(W_{ch1}), \text{abs}(W_{ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b$ = **0.073** in

$\delta_{sww} / \Delta_{w\_allow}$ = **0.229**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q = \mathbf{0.931}$ kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h = \mathbf{2.278}$ in

Induced unit shear; $v_{\delta s} = V_{\delta s} / b = \mathbf{38.52}$ lb/ft

Anchor tension force; $T_{\delta} = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a = \mathbf{0.000}$ in

Shear wall elastic deflection – Eqn. 4.3-1; $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b = \mathbf{0.025}$ in

Deflection ampification factor; $C_{d\delta} = \mathbf{4}$

Seismic importance factor; $I_e = \mathbf{1}$

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e = \mathbf{0.1}$ in

$\delta_{sws} / \Delta_{s\_allow} = \mathbf{0.044}$

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8822	2015	0.228	PASS
Chord capacity	lb/in^2	427	173	0.405	PASS
Deflection	in	0.318	0.090	0.283	PASS

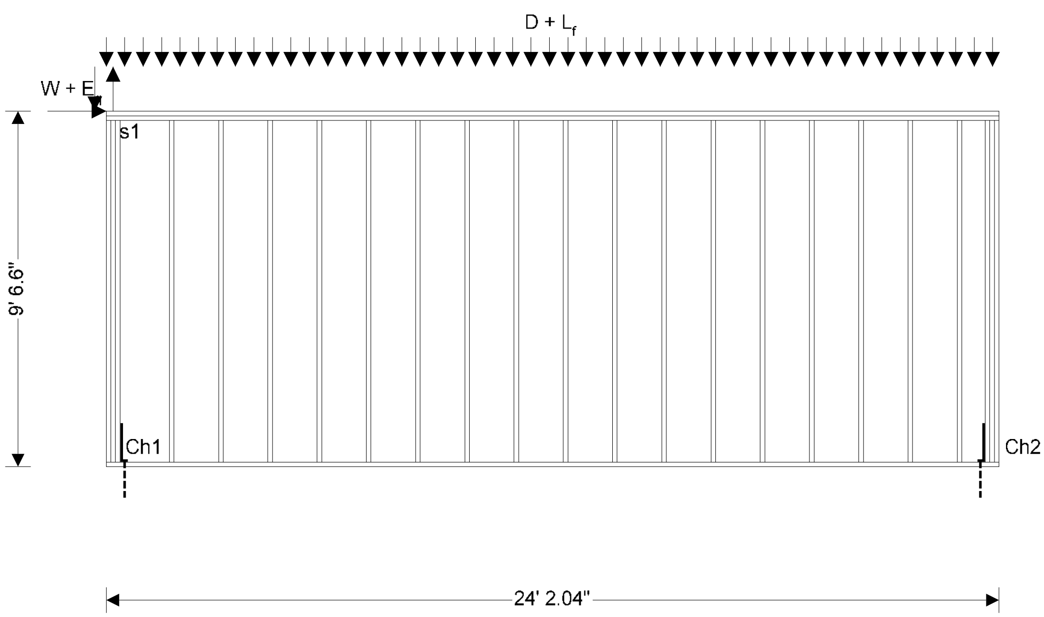
Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.55** ft

Panel length; b = **24.17** ft

Total area of wall; A = h × b = **230.824** 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; 3 x 2" x 4"

Dressed end post size; 3 x 1.5" x 3.5"

Cross-sectional area of end posts; A_e = **15.75** in^2

Hole diameter; Dia = **1** in

Net cross-sectional area of end posts; A_{en} = **11.25** 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
Vertical anchor stiffness; k_a = **40044** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **97.135** lb/ft
Floor live load acting on top of panel; L_f = **328.54** lb/ft
Self weight of panel; S_{wt} = **11** lb/ft^2
In plane wind load acting at head of panel; W = **3358** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **1009** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-2334;	-790;	392;	7112;	438;	52;	0;	0;
Ch2;	2334;	790;	392;	7112;	438;	52;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **445** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Shear wall length; b = **24.17** ft

Shear wall aspect ratio; h / b = **0.395**

Segmented shear wall capacity

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **2.015** kips

Shear capacity for wind loading; $V_w = v_w \times b / 2$ = **8.822** kips

V_{w_max} / V_w = **0.228**

PASS - Shear capacity for wind load exceeds maximum shear force

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **0.706** kips

Shear capacity for seismic loading; $V_s = v_s \times b / 2$ = **6.284** kips

V_{s_max} / V_s = **0.112**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b = **0.395**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.015** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **4.333** kips

Maximum tensile force in chord; $T = V \times h / b - P$ = **-3.537** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-314** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.228**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **2.015** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **1.927** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **2.723** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **173** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **427** lb/in^2

$f_c / F_c' =$ **0.405**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.277**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.015** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **4.333** kips

Maximum tensile force in chord; $T = V \times h / b - P =$ **-3.537** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-314** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.228**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **2.015** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **1.927** kips

Maximum compressive force in chord; $C = V \times h / b + P =$ **2.723** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **173** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **427** lb/in^2

$f_c / F_c' =$ **0.405**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.277**

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

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **3.358** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.318** in

Induced unit shear; $v_{\delta w} = V_{\delta w} / b =$ **138.93** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\mathrm{abs}(W_{ch1}), \mathrm{abs}(W_{ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Shear wall deflection – Eqn. 4.3-1; $\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.09** in

$\delta_{sww} / \Delta_{w\_allow} =$ **0.283**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s}$ = E_q = **1.009** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **2.292** in

Induced unit shear; $v_{\delta s}$ = $V_{\delta s}$ / b = **41.75** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s}$ × h - (0.6 - 0.2 × S_{DS}) × (D + S_{wt} × h) × b / 2 - (0.6 - 0.2 × S_{DS}) × min(D_{T_ch1},D_{T_ch2}) + max(abs(E_{q_ch1}),abs(E_{q_ch2}))) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** in

Shear wall elastic deflection – Eqn. 4.3-1; δ_{swse} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b = **0.027** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; δ_{sws} = $C_{d\delta}$ × δ_{swse} / I_e = **0.108** in

δ_{sws} / Δ_{s_allow} = **0.047**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

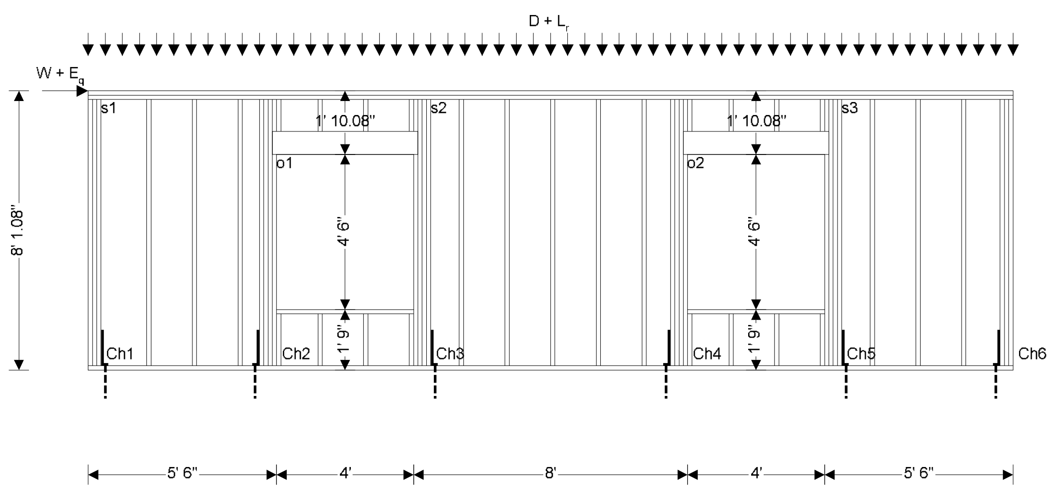
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	6525	1893	0.290	PASS
Chord capacity	lb/in^2	1252	46	0.074	PASS
Collector capacity	lb/in^2	1196	10	0.008	PASS
Deflection	in	0.270	0.092	0.343	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **27** ft



Panel opening details

Width of opening; w_{o1} = **4** ft

Height of opening; h_{o1} = **4.5** ft

Height to underside of lintel over opening; l_{o1} = **6.25** ft

Position of opening; P_{o1} = **5.5** ft

Width of opening; w_{o2} = **4** ft

Height of opening; h_{o2} = **4.5** ft

Height to underside of lintel over opening; l_{o2} = **6.25** ft

Position of opening; P_{o2} = **17.5** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **182.43** 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;	3 x 2" x 6"
Dressed end post size;	3 x 1.5" x 5.5"
Cross-sectional area of end posts;	A_e = **24.75** in^2
Hole diameter;	Dia = **1** in
Net cross-sectional area of end posts;	A_{en} = **20.25** 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
Vertical anchor stiffness;	k_a = **49087** lb/in

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

Species, grade and size classification;	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity;	G = **0.50**
Tension parallel to grain;	F_t = **575** lb/in^2
Compression parallel to grain;	F_c = **1350** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Modulus of elasticity;	E = **1600000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material;	7/16" wood panel oriented strandboard sheathing
Fastener type;	8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design;	v_s = **520** lb/ft
Nominal unit shear capacity for wind design;	v_w = **730** lb/ft
Apparent shear wall shear stiffness;	G_a = **15** kips/in

Loading details

Dead load acting on top of panel;	D = **304.425** lb/ft
Roof live load acting on top of panel;	L_r = **243.54** lb/ft
Self weight of panel;	S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel;	W = **3155** lbs
Wind load serviceability factor;	f_{Wserv} = **1.00**
In plane seismic load acting at head of panel;	E_q = **699.5** lbs
Design spectral response accel. par., short periods;	S_{DS} = **0.106**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi
Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1530** psi
Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi
For sawn lumber; c = **0.8**
Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.53**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios
Maximum shear wall aspect ratio; 3.5
Segment 1 wall length; b_1 = **5.5** ft
Shear wall aspect ratio; h / b_1 = **1.471**
Segment 2 wall length; b_2 = **8** ft
Shear wall aspect ratio; h / b_2 = **1.011**
Segment 3 wall length; b_3 = **5.5** ft
Shear wall aspect ratio; h / b_3 = **1.471**

Segmented shear wall capacity - Equal deflection method
Wind loading:
Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.021** in/kip
Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **11.121** kips/in
Unit shear capacity, widest segment; $v_{sww2} = v_w / 2$ = **365** plf
Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww2} / k_a$ = **0.060** in
Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sww2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2$ = **0.263** in
Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **0.030** in/kip
Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **6.865** kips/in
Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1$ = **327.72** plf
Segment 1 shear capacity; $v_{sww1} = v_w / 2$ = **365** plf
v_{dsww1} / v_{sww1} = **0.898**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.030** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **6.865** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 =$ **327.72** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2 =$ **365** plf

$v_{dsww3} / v_{sww3} =$ **0.898**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W =$ **1.893** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1}, v_{dsww1}) \times b_1 + v_{sww2} \times b_2 + \min(v_{sww3}, v_{dsww3}) \times b_3 =$ **6.525** kips

$V_{w\_max} / V_w =$ **0.29**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **0.021** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **11.121** kips/in

Unit shear capacity, widest segment; $v_{sws2} = v_s / 2 =$ **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws2} / k_a =$ **0.043** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sws2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2 =$ **0.187** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **0.030** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **6.865** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsws1} = \delta_{Cap} \times k_1 / b_1 =$ **233.44** plf

Segment 1 shear capacity; $v_{sws1} = v_s / 2 =$ **260** plf

$v_{dsws1} / v_{sws1} =$ **0.898**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.030** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **6.865** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3 =$ **233.44** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2 =$ **260** plf

$v_{dsws3} / v_{sws3} =$ **0.898**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **0.49** kips

Shear capacity for seismic loading; $V_s = \min(v_{sws1}, v_{dsws1}) \times b_1 + v_{sws2} \times b_2 + \min(v_{sws3}, v_{dsws3}) \times b_3 =$ **4.648** kips

$V_{s\_max} / V_s =$ **0.105**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; $h / b_1 =$ **1.471**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.893** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 =$ **0.836** kips

Maximum tensile force in chord; $T = V \times (k_1 / sum(k_1,k_2,k_3)) \times h / b_1 - P =$ **-0.033** kips

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

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.001**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **1.893** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.338** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_1,k_2,k_3)) \times h / b_1 + P =$ **1.141** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.037**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.074**

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

Chord capacity for chords 3 and 4

Shear wall aspect ratio; $h / b_2 =$ **1.011**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.893** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.216** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_1,k_2,k_3)) \times h / b_2 - P =$ **-0.406** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-20** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.017**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **1.893** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.338** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_1,k_2,k_3)) \times h / b_2 + P =$ **1.147** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.037**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.074**

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

Chord capacity for chords 5 and 6

Shear wall aspect ratio; $h / b_3 =$ **1.471**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.893** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **0.836** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_1,k_2,k_3)) \times h / b_3 - P =$ **-0.033** kips

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

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in^2

$f_t / F_t' = \mathbf{-0.001}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W = \mathbf{1.893}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 = \mathbf{0.338}$ kips

Maximum compressive force in chord; $C = V \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_3 + P = \mathbf{1.141}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{46}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{1252}$ lb/in^2

$f_c / F_c' = \mathbf{0.037}$

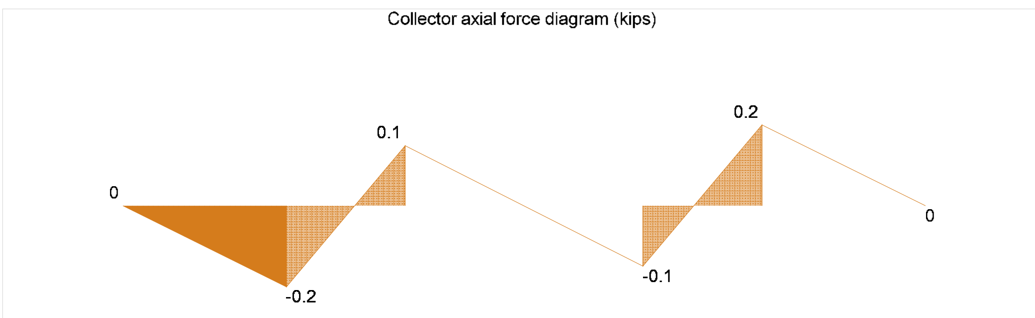
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.074}$

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

Collector capacity



Collector seismic design force factor; $F_{Coll} = \mathbf{1}$

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max}) = \mathbf{1.893}$ kips

Maximum force in collector; $P_{coll} = \mathbf{0.161}$ kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) = \mathbf{10}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in^2

$f_t / F_t' = \mathbf{0.008}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) = \mathbf{10}$ lb/in^2

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

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{2376}$ lb/in^2

$f_c / F_c' = \mathbf{0.004}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W = \mathbf{3.155}$ kips

Deflection limit; $\Delta_{w\_allow} = h / 360 = \mathbf{0.27}$ in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \text{sum}(k_1,k_2,k_3)) / b_1 =$ **165.52** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.092** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.343**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \text{sum}(k_1,k_2,k_3)) / b_2 =$ **166.79** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.092** in

$\delta_{sww2} / \Delta_{w\_allow} =$ **0.342**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \text{sum}(k_1,k_2,k_3)) / b_3 =$ **165.52** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.092** in

$\delta_{sww3} / \Delta_{w\_allow} =$ **0.343**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.7** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **1.942** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / \text{sum}(k_1,k_2,k_3)) / b_1 =$ **36.7** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.021** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e =$ **0.082** in

$\delta_{sws1} / \Delta_{s\_allow} =$ **0.042**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / \text{sum}(k_1,k_2,k_3)) / b_2 =$ **36.98** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.02** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.082** in

$\delta_{sws2} / \Delta_{s\_allow}$ = **0.042**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \text{sum}(k_1,k_2,k_3)) / b_3$ = **36.7** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.021** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e$ = **0.082** in

$\delta_{sws3} / \Delta_{s\_allow}$ = **0.042**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

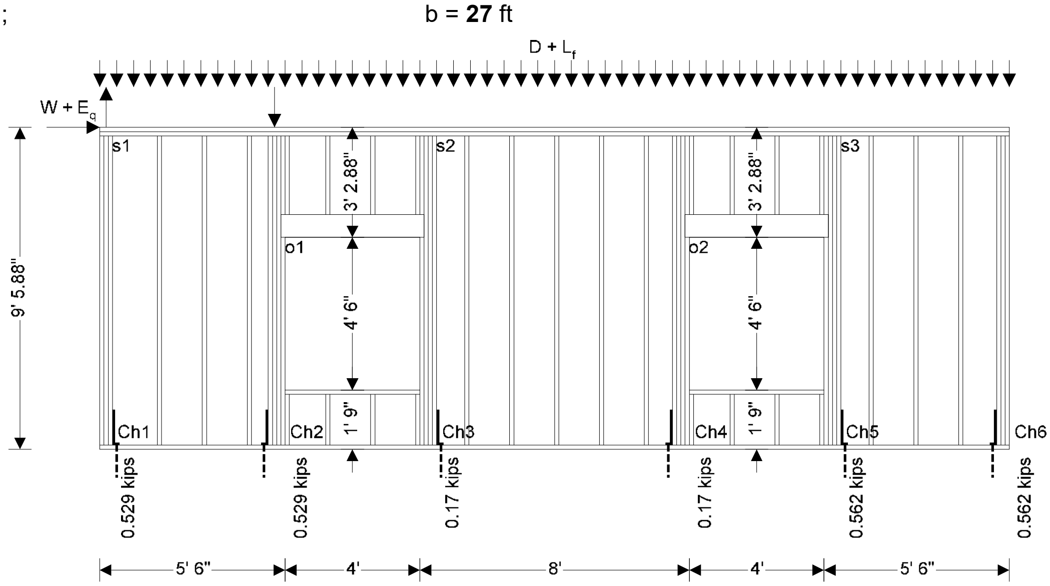
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	6476	2927	0.452	PASS
Chord capacity	lb/in^2	976	120	0.191	PASS
Collector capacity	lb/in^2	1196	15	0.013	PASS
Deflection	in	0.316	0.170	0.537	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **27** ft



Panel opening details

Width of opening;	w_{o1} = **4** ft
Height of opening;	h_{o1} = **4.5** ft
Height to underside of lintel over opening;	l_{o1} = **6.25** ft
Position of opening;	P_{o1} = **5.5** ft
Width of opening;	w_{o2} = **4** ft
Height of opening;	h_{o2} = **4.5** ft
Height to underside of lintel over opening;	l_{o2} = **6.25** ft
Position of opening;	P_{o2} = **17.5** ft
Total area of wall;	$A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **220.23** 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;	3 x 2" x 6"
Dressed end post size;	3 x 1.5" x 5.5"
Cross-sectional area of end posts;	A_e = **24.75** in^2
Hole diameter;	Dia = **1** in
Net cross-sectional area of end posts;	A_{en} = **20.25** 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
Vertical anchor stiffness;	k_a = **49087** lb/in

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

Species, grade and size classification;	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity;	G = **0.50**
Tension parallel to grain;	F_t = **575** lb/in^2
Compression parallel to grain;	F_c = **1350** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Modulus of elasticity;	E = **1600000** lb/in^2
Minimum modulus of elasticity;	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material;	7/16" wood panel oriented strandboard sheathing
Fastener type;	8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design;	v_s = **520** lb/ft
Nominal unit shear capacity for wind design;	v_w = **730** lb/ft
Apparent shear wall shear stiffness;	G_a = **15** kips/in

Loading details

Dead load acting on top of panel;	D = **304.425** lb/ft

Floor live load acting on top of panel; L_f = **487.08** lb/ft

Self weight of panel; S_{wt} = **25** lb/ft^2

In plane wind load acting at head of panel; W = **4878** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **1228.5** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-1339;	-297;	338;	1393;	0;	162;	0;	0;
Ch2;	1339;	297;	338;	1393;	0;	162;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1112** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **5.5** ft

Shear wall aspect ratio; h / b_1 = **1.725**

Segment 2 wall length; b_2 = **8** ft

Shear wall aspect ratio; h / b_2 = **1.186**

Segment 3 wall length; b_3 = **5.5** ft

Shear wall aspect ratio; h / b_3 = **1.725**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.024** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **9.054** kips/in

Unit shear capacity, widest segment; $v_{sww2} = v_w / 2$ = **365** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww2} / k_a$ = **0.071** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sww2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2$ = **0.323** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **0.035** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **5.513** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1$ = **323.27** plf

Segment 1 shear capacity; $v_{sww1} = v_w / 2$ = **365** plf

v_{dsww1} / v_{sww1} = **0.886**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **0.035** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **5.513** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3$ = **323.27** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2$ = **365** plf

v_{dsww3} / v_{sww3} = **0.886**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **2.927** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1},v_{dsww1}) \times b_1 + v_{sww2} \times b_2 + \min(v_{sww3},v_{dsww3}) \times b_3$ = **6.476** kips

V_{w_max} / V_w = **0.452**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.024** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **9.054** kips/in

Unit shear capacity, widest segment; $v_{sws2} = v_s / 2$ = **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws2} / k_a$ = **0.050** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sws2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2$ = **0.23** in

Segment 1 vertical unit deflection;	$\Delta_{a1} = h / (k_a \times b_1) =$ **0.035** in/kip
Segment 1 stiffness;	$k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **5.513** kips/in
Segment 1 unit shear at δ_{Cap};	$v_{dsws1} = \delta_{Cap} \times k_1 / b_1 =$ **230.28** plf
Segment 1 shear capacity;	$v_{sws1} = v_s / 2 =$ **260** plf
	$v_{dsws1} / v_{sws1} =$ **0.886**
	PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}
Segment 3 vertical unit deflection;	$\Delta_{a1} = h / (k_a \times b_3) =$ **0.035** in/kip
Segment 3 stiffness;	$k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **5.513** kips/in
Segment 3 unit shear at δ_{Cap};	$v_{dsws3} = \delta_{Cap} \times k_3 / b_3 =$ **230.28** plf
Segment 3 shear capacity;	$v_{sws3} = v_s / 2 =$ **260** plf
	$v_{dsws3} / v_{sws3} =$ **0.886**
	PASS - Segment shear capacity exceeds segment unit shear at δ_{Cap}
Maximum shear force under seismic loading;	$V_{s\_max} = 0.7 \times E_q =$ **0.86** kips
Shear capacity for seismic loading;	$V_s = \min(v_{sws1}, v_{dsws1}) \times b_1 + v_{sws2} \times b_2 + \min(v_{sws3}, v_{dsws3}) \times b_3 =$ **4.613** kips
	$V_{s\_max} / V_s =$ **0.186**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio;	$h / b_1 =$ **1.725**
Load combination 5	
Shear force for maximum tension;	$V = 0.6 \times W =$ **2.927** kips
Axial force for maximum tension;	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1} =$ **0.926** kips
Maximum tensile force in chord;	$T = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_1 - P =$ **0.529** kips
Maximum applied tensile stress;	$f_t = T / A_{en} =$ **26** lb/in^2
Design tensile stress;	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2
	$f_t / F_t' =$ **0.022**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression;	$V = 0.6 \times W =$ **2.927** kips
Axial force for maximum compression;	$P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **1.502** kips
Maximum compressive force in chord;	$C = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_1 + P =$ **2.958** kips
Maximum applied compressive stress;	$f_c = C / A_e =$ **120** lb/in^2
Design compressive stress;	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2
	$f_c / F_c' =$ **0.122**
	PASS - Design compressive stress exceeds maximum applied compressive stress
Design bearing compr. stress, bottom plate;	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2
	$f_c / F_{c\_perp}' =$ **0.191**
	PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Chord capacity for chord 2

Load combination 5	
Shear force for maximum tension;	$V = 0.6 \times W =$ **2.927** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2}$ = **0.926** kips

Maximum tensile force in chord; $T = V \times (k_1 / sum(k_1,k_2,k_3)) \times h / b_1 - P$ = **0.529** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **26** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.022**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **2.927** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2}$ = **1.502** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_1,k_2,k_3)) \times h / b_1 + P$ = **2.958** kips

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

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.122**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.191**

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

Chord capacity for chord 3

Shear wall aspect ratio; h / b_2 = **1.186**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.927** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.3** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_1,k_2,k_3)) \times h / b_2 - P$ = **0.170** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **8** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.007**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **2.927** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.361** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_1,k_2,k_3)) \times h / b_2 + P$ = **1.831** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.076**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.118**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.927** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.3** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_1,k_2,k_3)) \times h / b_2 - P =$ **0.170** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **8** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **0.007**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **2.927** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.361** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_1,k_2,k_3)) \times h / b_2 + P =$ **1.831** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.076**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.118**

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

Chord capacity for chord 5

Shear wall aspect ratio; $h / b_3 =$ **1.725**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.927** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **0.894** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_1,k_2,k_3)) \times h / b_3 - P =$ **0.562** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **28** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **0.023**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **2.927** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.361** kips

Maximum compressive force in chord; $C = V \times (k_3 / sum(k_1,k_2,k_3)) \times h / b_3 + P =$ **1.817** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **73** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.075**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.117**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.927** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **0.894** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_1,k_2,k_3)) \times h / b_3 - P =$ **0.562** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **28** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.023**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **2.927** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.361** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_1,k_2,k_3)) \times h / b_3 + P$ = **1.817** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **73** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.075**

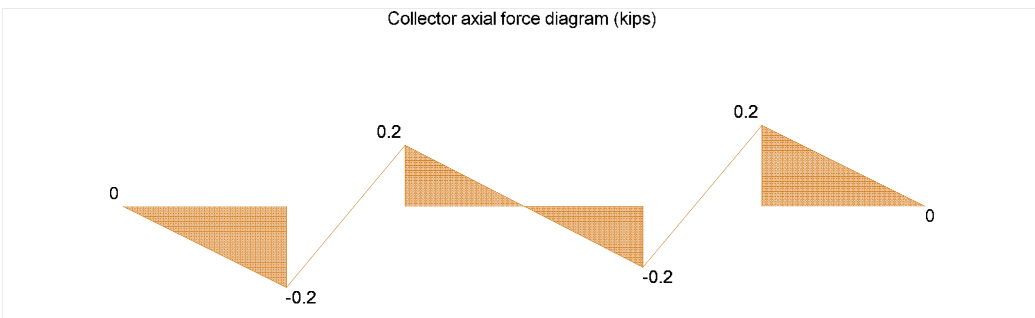
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.117**

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

Collector capacity



Collector seismic design force factor; F_{Coll} = **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **2.927** kips

Maximum force in collector; P_{coll} = **0.248** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s)$ = **15** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.013**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s)$ = **15** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2376** lb/in^2

f_c / F_c' = **0.006**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; T_1 = **0.529** kips

Chord 2; T_2 = **0.529** kips

Chord 3; T_3 = **0.17** kips

Chord 4; T_4 = **0.17** kips

Chord 5; T_5 = **0.562** kips

Chord 6; T_6 = **0.562** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-3765;	-908;	699;	2883;	325;	162;	0;	0;
Ch2;	3765;	908;	699;	2883;	325;	162;	0;	0;
Ch3;	-2450;	-617;	361;	2167;	325;	0;	0;	0;
Ch4;	2450;	617;	361;	2167;	325;	0;	0;	0;
Ch5;	-2426;	-611;	361;	1490;	325;	0;	0;	0;
Ch6;	2426;	611;	361;	1490;	325;	0;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **4.878** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / sum(k_1,k_2,k_3)) / b_1$ = **255.67** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times min(D_{T\_ch1},D_{T\_ch2}) + max(abs(W_{ch1}),abs(W_{ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.17** in

$\delta_{sww1} / \Delta_{w\_allow}$ = **0.537**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / sum(k_1,k_2,k_3)) / b_2$ = **258.21** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.169** in

$\delta_{sww2} / \Delta_{w\_allow}$ = **0.534**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / sum(k_1,k_2,k_3)) / b_3$ = **255.67** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.17** in

$\delta_{sww3} / \Delta_{w\_allow}$ = **0.537**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q$ = **1.229** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h$ = **2.278** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / \text{sum}(k_1,k_2,k_3)) / b_1 =$ **64.39** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1}, D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}), \text{abs}(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.043** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e =$ **0.171** in

$\delta_{sws1} / \Delta_{s\_allow} =$ **0.075**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / \text{sum}(k_1,k_2,k_3)) / b_2 =$ **65.03** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.043** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.17** in

$\delta_{sws2} / \Delta_{s\_allow} =$ **0.075**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \text{sum}(k_1,k_2,k_3)) / b_3 =$ **64.39** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.043** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e =$ **0.171** in

$\delta_{sws3} / \Delta_{s\_allow} =$ **0.075**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

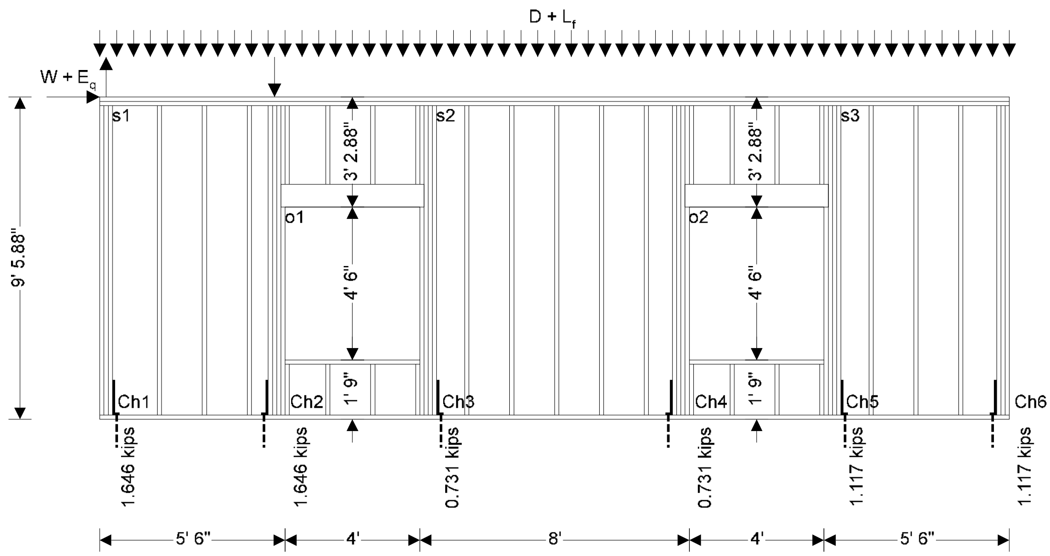
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	6476	4042	0.624	PASS
Chord capacity	lb/in^2	976	215	0.345	PASS
Collector capacity	lb/in^2	1196	21	0.017	PASS
Deflection	in	0.285	0.270	0.947	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **27** ft



Panel opening details

Width of opening; w_{o1} = **4** ft

Height of opening; h_{o1} = **4.5** ft

Height to underside of lintel over opening; l_{o1} = **6.25** ft

Position of opening; P_{o1} = **5.5** ft

Width of opening; w_{o2} = **4** ft

Height of opening; h_{o2} = **4.5** ft

Height to underside of lintel over opening; l_{o2} = **6.25** ft

Position of opening; P_{o2} = **17.5** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **220.23** 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; 3 x 2" x 6"
Dressed end post size; 3 x 1.5" x 5.5"
Cross-sectional area of end posts; A_e = **24.75** in^2
Hole diameter; Dia = **1** in
Net cross-sectional area of end posts; A_{en} = **20.25** 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
Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **304.425** lb/ft
Floor live load acting on top of panel; L_f = **487.08** lb/ft
Self weight of panel; S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel; W = **6737** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **1569** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-3765;	-908;	699;	2883;	325;	162;	0;	0;
Ch2;	3765;	908;	699;	2883;	325;	162;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1112** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **5.5** ft

Shear wall aspect ratio; h / b_1 = **1.725**

Segment 2 wall length; b_2 = **8** ft

Shear wall aspect ratio; h / b_2 = **1.186**

Segment 3 wall length; b_3 = **5.5** ft

Shear wall aspect ratio; h / b_3 = **1.725**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.024** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **9.054** kips/in

Unit shear capacity, widest segment; $v_{sww2} = v_w / 2$ = **365** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww2} / k_a =$ **0.071** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sww2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2 =$ **0.323** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **0.035** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **5.513** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1 =$ **323.27** plf

Segment 1 shear capacity; $v_{sww1} = v_w / 2 =$ **365** plf

$v_{dsww1} / v_{sww1} =$ **0.886**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.035** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **5.513** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 =$ **323.27** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2 =$ **365** plf

$v_{dsww3} / v_{sww3} =$ **0.886**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W =$ **4.042** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1},v_{dsww1}) \times b_1 + v_{sww2} \times b_2 + \min(v_{sww3},v_{dsww3}) \times b_3 =$ **6.476** kips

$V_{w\_max} / V_w =$ **0.624**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **0.024** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **9.054** kips/in

Unit shear capacity, widest segment; $v_{sws2} = v_s / 2 =$ **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws2} / k_a =$ **0.050** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sws2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2 =$ **0.23** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **0.035** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **5.513** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsws1} = \delta_{Cap} \times k_1 / b_1 =$ **230.28** plf

Segment 1 shear capacity; $v_{sws1} = v_s / 2 =$ **260** plf

$v_{dsws1} / v_{sws1} =$ **0.886**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.035** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **5.513** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3 =$ **230.28** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2 =$ **260** plf

$v_{dsws3} / v_{sws3} =$ **0.886**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **1.098** kips

Shear capacity for seismic loading; $V_s = \min(v_{sws1},v_{dsws1}) \times b_1 + v_{sws2} \times b_2 + \min(v_{sws3},v_{dsws3}) \times b_3 =$ **4.613** kips

$V_{s\_max} / V_s =$ **0.238**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; $h / b_1 =$ **1.725**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **4.042** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1} =$ **0.364** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_1,k_2,k_3)) \times h / b_1 - P =$ **1.646** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **81** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **0.068**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **4.042** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **3.319** kips

Maximum compressive force in chord; $C = V \times (k_1 / \text{sum}(k_1,k_2,k_3)) \times h / b_1 + P =$ **5.330** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **215** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.345**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **4.042** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **0.364** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_1,k_2,k_3)) \times h / b_1 - P =$ **1.646** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **81** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **0.068**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **4.042** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **3.319** kips

Maximum compressive force in chord; $C = V \times (k_1 / \text{sum}(k_1,k_2,k_3)) \times h / b_1 + P =$ **5.330** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **215** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.345**

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

Chord capacity for chord 3

Shear wall aspect ratio; h / b_2 = **1.186**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **4.042** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.3** kips

Maximum tensile force in chord; $T = V \times (k_2 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_2 - P$ = **0.731** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **36** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.030**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **4.042** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.361** kips

Maximum compressive force in chord; $C = V \times (k_2 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_2 + P$ = **2.392** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **97** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.099**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.155**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **4.042** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.3** kips

Maximum tensile force in chord; $T = V \times (k_2 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_2 - P$ = **0.731** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **36** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.030**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **4.042** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.361** kips

Maximum compressive force in chord; $C = V \times (k_2 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_2 + P$ = **2.392** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **97** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.099**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.155**

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

Chord capacity for chord 5

Shear wall aspect ratio; $h / b_3 = \mathbf{1.725}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{4.042}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 = \mathbf{0.894}$ kips

Maximum tensile force in chord; $T = V \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_3 - P = \mathbf{1.117}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{55}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in^2

$f_t / F_t' = \mathbf{0.046}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W = \mathbf{4.042}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 = \mathbf{0.361}$ kips

Maximum compressive force in chord; $C = V \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_3 + P = \mathbf{2.372}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{96}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{976}$ lb/in^2

$f_c / F_c' = \mathbf{0.098}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.153}$

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{4.042}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 = \mathbf{0.894}$ kips

Maximum tensile force in chord; $T = V \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_3 - P = \mathbf{1.117}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{55}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in^2

$f_t / F_t' = \mathbf{0.046}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W = \mathbf{4.042}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 = \mathbf{0.361}$ kips

Maximum compressive force in chord; $C = V \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_3 + P = \mathbf{2.372}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{96}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{976}$ lb/in^2

$f_c / F_c' = \mathbf{0.098}$

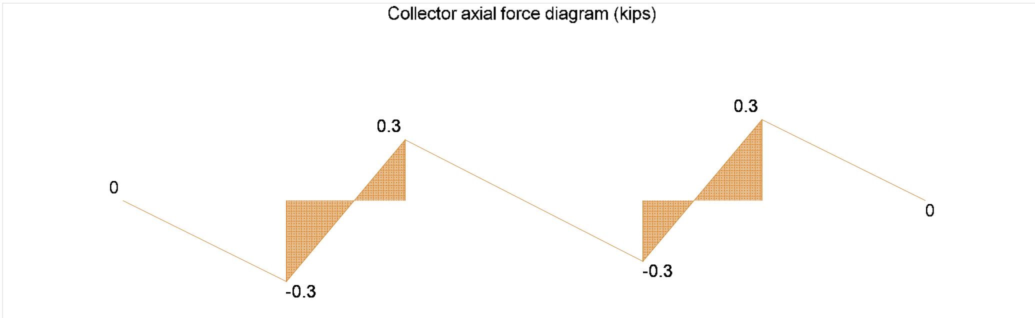
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.153}$

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

Collector capacity



Collector seismic design force factor; F_{Coll} = **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **4.042** kips

Maximum force in collector; P_{coll} = **0.342** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s)$ = **21** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.017**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s)$ = **21** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2376** lb/in^2

f_c / F_c' = **0.009**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; T_1 = **1.646** kips

Chord 2; T_2 = **1.646** kips

Chord 3; T_3 = **0.731** kips

Chord 4; T_4 = **0.731** kips

Chord 5; T_5 = **1.117** kips

Chord 6; T_6 = **1.117** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-7116;	-1688;	1060;	4373;	649;	162;	0;	0;
Ch2;	7116;	1688;	1060;	4373;	649;	162;	0;	0;
Ch3;	-3384;	-788;	361;	2167;	325;	0;	0;	0;
Ch4;	3384;	788;	361;	2167;	325;	0;	0;	0;
Ch5;	-3351;	-780;	361;	1490;	325;	0;	0;	0;
Ch6;	3351;	780;	361;	1490;	325;	0;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **6.737** kips

Deflection limit; Δ_{w_allow}= h / 400 = **0.285** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / sum(k_1,k_2,k_3)) / b_1$ = **353.1** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times min(D_{T\_ch1}, D_{T\_ch2}) + max(abs(W_{ch1}), abs(W_{ch2})))$ = **0.999** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.020** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.27** in

$\delta_{sww1} / \Delta_{w\_allow}$ = **0.947**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / sum(k_1,k_2,k_3)) / b_2$ = **356.61** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.233** in

$\delta_{sww2} / \Delta_{w\_allow}$ = **0.819**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / sum(k_1,k_2,k_3)) / b_3$ = **353.1** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.234** in

$\delta_{sww3} / \Delta_{w\_allow}$ = **0.824**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q$ = **1.569** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **2.278** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / sum(k_1,k_2,k_3)) / b_1$ = **82.24** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times min(D_{T\_ch1}, D_{T\_ch2}) + max(abs(E_{q\_ch1}), abs(E_{q\_ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.055** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e$ = **0.218** in

$\delta_{sws1} / \Delta_{s\_allow}$ = **0.096**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / sum(k_1,k_2,k_3)) / b_2$ = **83.05** lb/ft

Anchor tension force; $T_\delta = max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.054** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.217** in

$\delta_{sws2} / \Delta_{s\_allow} =$ **0.095**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / sum(k_1,k_2,k_3)) / b_3 =$ **82.24** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.055** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e =$ **0.218** in

$\delta_{sws3} / \Delta_{s\_allow} =$ **0.096**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

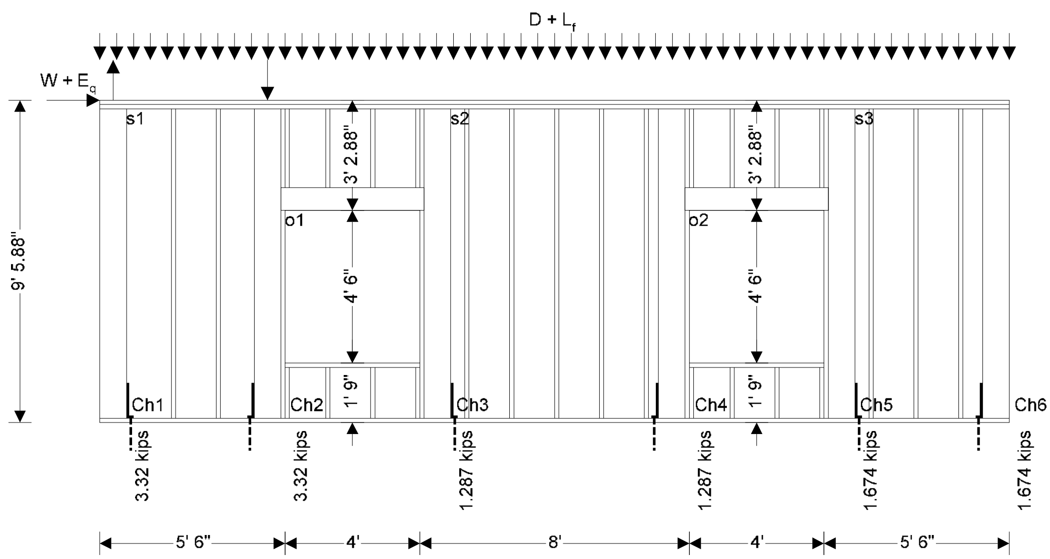
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	9566	5158	0.539	PASS
Chord capacity	lb/in^2	976	158	0.253	PASS
Collector capacity	lb/in^2	1196	27	0.022	PASS
Deflection	in	0.316	0.280	0.886	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **27** ft



Panel opening details

Width of opening; w_{o1} = **4** ft

Height of opening; h_{o1} = **4.5** ft

Height to underside of lintel over opening; l_{o1} = **6.25** ft

Position of opening; P_{o1} = **5.5** ft

Width of opening; w_{o2} = **4** ft

Height of opening; h_{o2} = **4.5** ft

Height to underside of lintel over opening; l_{o2} = **6.25** ft

Position of opening; P_{o2} = **17.5** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **220.23** 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; 10" x 6"
Dressed end post size; 9.5" x 5.5"
Cross-sectional area of end posts; A_e = **52.25** in^2
Hole diameter; Dia = **1** in
Net cross-sectional area of end posts; A_{en} = **42.75** 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
Vertical anchor stiffness; k_a = **95741** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" 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 for seismic design; v_s = **760** lb/ft
Nominal unit shear capacity for wind design; v_w = **1065** lb/ft
Apparent shear wall shear stiffness; G_a = **22** kips/in

Loading details

Dead load acting on top of panel; D = **304.425** lb/ft
Floor live load acting on top of panel; L_f = **487.08** lb/ft
Self weight of panel; S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel; W = **8596** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **1699** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-7116;	-1688;	1060;	4373;	649;	162;	0;	0;
Ch2;	7116;	1688;	1060;	4373;	649;	162;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;

Ch6;	0;	0;	0;	0;	0;	0;	0;	0;

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} = **1.30**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1112** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **5.5** ft

Shear wall aspect ratio; h / b_1 = **1.725**

Segment 2 wall length; b_2 = **8** ft

Shear wall aspect ratio; h / b_2 = **1.186**

Segment 3 wall length; b_3 = **5.5** ft

Shear wall aspect ratio; h / b_3 = **1.725**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.012** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **14.307** kips/in

Unit shear capacity, widest segment; $v_{sww2} = v_w / 2 =$ **532.5** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww2} / k_a =$ **0.053** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sww2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2 =$ **0.298** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **0.018** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **8.91** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1 =$ **482.38** plf

Segment 1 shear capacity; $v_{sww1} = v_w / 2 =$ **532.5** plf

$v_{dsww1} / v_{sww1} =$ **0.906**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.018** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **8.91** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 =$ **482.38** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2 =$ **532.5** plf

$v_{dsww3} / v_{sww3} =$ **0.906**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W =$ **5.158** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1}, v_{dsww1}) \times b_1 + v_{sww2} \times b_2 + \min(v_{sww3}, v_{dsww3}) \times b_3 =$ **9.566** kips

$V_{w\_max} / V_w =$ **0.539**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **0.012** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **14.307** kips/in

Unit shear capacity, widest segment; $v_{sws2} = v_s / 2 =$ **380** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws2} / k_a =$ **0.038** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sws2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2 =$ **0.212** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **0.018** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **8.91** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsws1} = \delta_{Cap} \times k_1 / b_1 =$ **344.24** plf

Segment 1 shear capacity; $v_{sws1} = v_s / 2 =$ **380** plf

$v_{dsws1} / v_{sws1} =$ **0.906**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.018** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **8.91** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3 =$ **344.24** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2 =$ **380** plf

$v_{dsws3} / v_{sws3} =$ **0.906**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **1.189** kips

Shear capacity for seismic loading; $V_s = \min(v_{sws1}, v_{dsws1}) \times b_1 + v_{sws2} \times b_2 + \min(v_{sws3}, v_{dsws3}) \times b_3 =$ **6.827** kips

$V_{s\_max} / V_s =$ **0.174**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; $h / b_1 =$ **1.725**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.158** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1} =$ **-0.752** kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_1 - P =$ **3.320** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **78** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **0.065**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **5.158** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + -1 \times 0.6 \times W_{ch1} + D_{C\_ch1} =$ **5.691** kips

Maximum compressive force in chord; $C = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_1 + P =$ **8.259** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **158** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.162**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.253**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.158** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **-0.752** kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_1 - P =$ **3.320** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **78** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **0.065**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W =$ **5.158** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2 + 0.6 \times W_{ch2} + D_{C\_ch2} =$ **5.691** kips

Maximum compressive force in chord; $C = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3)) \times h / b_1 + P =$ **8.259** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **158** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.162**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.253**

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

Chord capacity for chord 3

Shear wall aspect ratio; h / b_2 = **1.186**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.158** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.3** kips

Maximum tensile force in chord; $T = V \times (k_2 / \text{sum}(k_1,k_2,k_3)) \times h / b_2 - P$ = **1.287** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **30** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.025**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **5.158** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.361** kips

Maximum compressive force in chord; $C = V \times (k_2 / \text{sum}(k_1,k_2,k_3)) \times h / b_2 + P$ = **2.948** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **56** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.058**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.090**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.158** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.3** kips

Maximum tensile force in chord; $T = V \times (k_2 / \text{sum}(k_1,k_2,k_3)) \times h / b_2 - P$ = **1.287** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **30** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.025**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; $V = 0.6 \times W$ = **5.158** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h)) \times s / 2$ = **0.361** kips

Maximum compressive force in chord; $C = V \times (k_2 / \text{sum}(k_1,k_2,k_3)) \times h / b_2 + P$ = **2.948** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **56** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.058**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.090**

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

Chord capacity for chord 5

Shear wall aspect ratio; h / b_3 = **1.725**

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **5.158** kips

Axial force for maximum tension; P = (0.6 × (D + S_{wt} × h)) × b_3 / 2 = **0.894** kips

Maximum tensile force in chord; T = V × (k_3 / sum(k_1,k_2,k_3)) × h / b_3 - P = **1.674** kips

Maximum applied tensile stress; f_t = T / A_{en} = **39** lb/in^2

Design tensile stress; F_t' = F_t × C_D × C_{Mt} × C_{tt} × C_{Ft} × C_i = **1196** lb/in^2

f_t / F_t' = **0.033**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 × W = **5.158** kips

Axial force for maximum compression; P = ((D + S_{wt} × h)) × s / 2 = **0.361** kips

Maximum compressive force in chord; C = V × (k_3 / sum(k_1,k_2,k_3)) × h / b_3 + P = **2.929** kips

Maximum applied compressive stress; f_c = C / A_e = **56** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **976** lb/in^2

f_c / F_c' = **0.057**

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 = **625** lb/in^2

f_c / F_{c_perp}' = **0.090**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **5.158** kips

Axial force for maximum tension; P = (0.6 × (D + S_{wt} × h)) × b_3 / 2 = **0.894** kips

Maximum tensile force in chord; T = V × (k_3 / sum(k_1,k_2,k_3)) × h / b_3 - P = **1.674** kips

Maximum applied tensile stress; f_t = T / A_{en} = **39** lb/in^2

Design tensile stress; F_t' = F_t × C_D × C_{Mt} × C_{tt} × C_{Ft} × C_i = **1196** lb/in^2

f_t / F_t' = **0.033**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 × W = **5.158** kips

Axial force for maximum compression; P = ((D + S_{wt} × h)) × s / 2 = **0.361** kips

Maximum compressive force in chord; C = V × (k_3 / sum(k_1,k_2,k_3)) × h / b_3 + P = **2.929** kips

Maximum applied compressive stress; f_c = C / A_e = **56** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **976** lb/in^2

f_c / F_c' = **0.057**

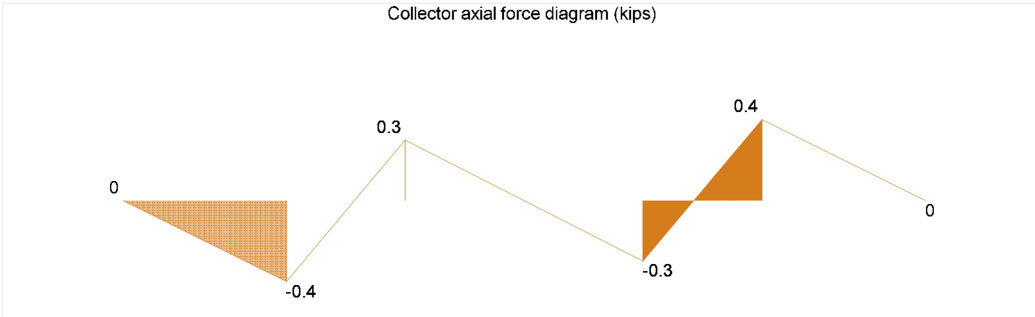
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; F_{c_perp}' = F_{c_perp} × C_{Mc} × C_{tc} × C_i × C_b = **625** lb/in^2

f_c / F_{c_perp}' = **0.090**

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

Collector capacity



Collector seismic design force factor; F_{Coll} = **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **5.158** kips

Maximum force in collector; P_{coll} = **0.438** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s)$ = **27** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.022**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s)$ = **27** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2376** lb/in^2

f_c / F_c' = **0.011**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1; T_1 = **3.32** kips

Chord 2; T_2 = **3.32** kips

Chord 3; T_3 = **1.287** kips

Chord 4; T_4 = **1.287** kips

Chord 5; T_5 = **1.674** kips

Chord 6; T_6 = **1.674** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	-11396;	-2534;	1421;	5862;	974;	162;	0;	0;
Ch2;	11396;	2534;	1421;	5862;	974;	162;	0;	0;
Ch3;	-4312;	-852;	361;	2167;	325;	0;	0;	0;
Ch4;	4312;	852;	361;	2167;	325;	0;	0;	0;
Ch5;	-4280;	-846;	361;	1490;	325;	0;	0;	0;
Ch6;	4280;	846;	361;	1490;	325;	0;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **8.596** kips

Deflection limit; Δ_{w_allow}= h / 360 = **0.316** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / sum(k_1,k_2,k_3)) / b_1$ = **450.98** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times min(D_{T\_ch1},D_{T\_ch2}) + max(abs(W_{ch1}),abs(W_{ch2})))$ = **4.385** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.046** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.28** in

$\delta_{sww1} / \Delta_{w\_allow}$ = **0.886**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / sum(k_1,k_2,k_3)) / b_2$ = **454.4** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.201** in

$\delta_{sww2} / \Delta_{w\_allow}$ = **0.634**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / sum(k_1,k_2,k_3)) / b_3$ = **450.98** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.201** in

$\delta_{sww3} / \Delta_{w\_allow}$ = **0.636**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q$ = **1.699** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **2.278** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / sum(k_1,k_2,k_3)) / b_1$ = **89.14** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times min(D_{T\_ch1},D_{T\_ch2}) + max(abs(E_{q\_ch1}),abs(E_{q\_ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.04** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e$ = **0.159** in

$\delta_{sws1} / \Delta_{s\_allow}$ = **0.07**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / sum(k_1,k_2,k_3)) / b_2$ = **89.81** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.04** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e$ = **0.159** in

$\delta_{sws2} / \Delta_{s\_allow}$ = **0.07**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \mathrm{sum}(k_1,k_2,k_3)) / b_3$ = **89.14** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.04** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e$ = **0.159** in

$\delta_{sws3} / \Delta_{s\_allow}$ = **0.07**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

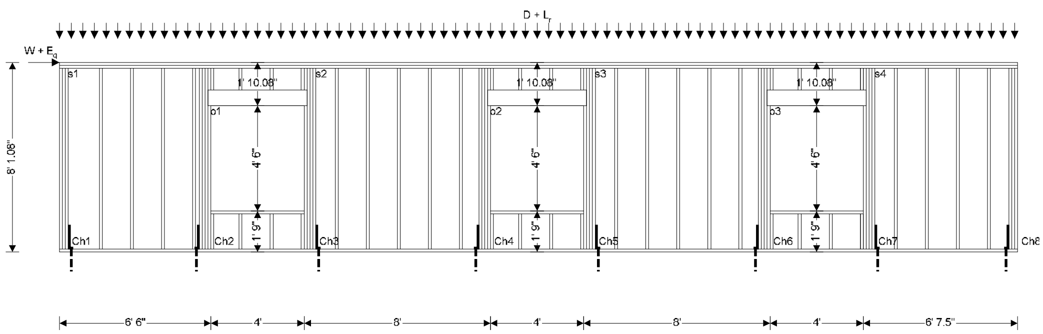
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	10382	963	0.093	PASS
Chord capacity	lb/in^2	1252	27	0.043	PASS
Collector capacity	lb/in^2	1196	4	0.003	PASS
Deflection	in	0.270	0.031	0.116	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **41.125** ft



Panel opening details

Width of opening; w_{o1} = **4** ft

Height of opening; h_{o1} = **4.5** ft

Height to underside of lintel over opening; l_{o1} = **6.25** ft

Position of opening; P_{o1} = **6.5** ft

Width of opening; w_{o2} = **4** ft

Height of opening; h_{o2} = **4.5** ft

Height to underside of lintel over opening; l_{o2} = **6.25** ft

Position of opening; P_{o2} = **18.5** ft

Width of opening; w_{o3} = **4** ft

Height of opening; h_{o3} = **4.5** ft

Height to underside of lintel over opening; l_{o3} = **6.25** ft

Position of opening; P_{o3} = **30.5** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3}$ = **278.701** 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; 3 x 2" x 6"
Dressed end post size; 3 x 1.5" x 5.5"
Cross-sectional area of end posts; A_e = **24.75** in^2
Hole diameter; Dia = **1** in
Net cross-sectional area of end posts; A_{en} = **20.25** 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
Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **304.425** lb/ft
Roof live load acting on top of panel; L_r = **243.54** lb/ft
Self weight of panel; S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel; W = **1605** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **925** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi
Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1530** psi
Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi
For sawn lumber; c = **0.8**
Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.53**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5
Segment 1 wall length; b_1 = **6.5** ft
Shear wall aspect ratio; h / b_1 = **1.245**
Segment 2 wall length; b_2 = **8** ft
Shear wall aspect ratio; h / b_2 = **1.011**
Segment 3 wall length; b_3 = **8** ft
Shear wall aspect ratio; h / b_3 = **1.011**
Segment 4 wall length; b_4 = **6.625** ft
Shear wall aspect ratio; h / b_4 = **1.221**

Segmented shear wall capacity - Equal deflection method

Wind loading:
Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **0.021** in/kip
Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **11.121** kips/in
Unit shear capacity, widest segment; $v_{sww3} = v_w / 2$ = **365** plf
Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww3} / k_a$ = **0.060** in
Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww3} \times h^3 / (3 \times E \times A_e \times b_3) + v_{sww3} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_3$ = **0.263** in
Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **0.025** in/kip
Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **8.542** kips/in
Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1$ = **345.07** plf

Segment 1 shear capacity; $v_{sww1} = v_w / 2 =$ **365** plf

$v_{dsww1} / v_{sww1} =$ **0.945**

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

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **0.021** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **11.121** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2 =$ **365** plf

Segment 2 shear capacity; $v_{sww2} = v_w / 2 =$ **365** plf

$v_{dsww2} / v_{sww2} =$ **1.000**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) =$ **0.025** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) =$ **8.755** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsww4} = \delta_{Cap} \times k_4 / b_4 =$ **346.98** plf

Segment 4 shear capacity; $v_{sww4} = v_w / 2 =$ **365** plf

$v_{dsww4} / v_{sww4} =$ **0.951**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W =$ **0.963** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1},v_{dsww1}) \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + v_{sww3} \times b_3 + \min(v_{sww4},v_{dsww4}) \times b_4 =$ **10.382** kips

$V_{w\_max} / V_w =$ **0.093**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.021** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **11.121** kips/in

Unit shear capacity, widest segment; $v_{sws3} = v_s / 2 =$ **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws3} / k_a =$ **0.043** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws3} \times h^3 / (3 \times E \times A_e \times b_3) + v_{sws3} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_3 =$ **0.187** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **0.025** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **8.542** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsws1} = \delta_{Cap} \times k_1 / b_1 =$ **245.8** plf

Segment 1 shear capacity; $v_{sws1} = v_s / 2 =$ **260** plf

$v_{dsws1} / v_{sws1} =$ **0.945**

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

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **0.021** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **11.121** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsws2} = \delta_{Cap} \times k_2 / b_2 =$ **260** plf

Segment 2 shear capacity; $v_{sws2} = v_s / 2 =$ **260** plf

$v_{dsws2} / v_{sws2} =$ **1.000**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) =$ **0.025** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) =$ **8.755** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsws4} = \delta_{Cap} \times k_4 / b_4 =$ **247.16** plf

Segment 4 shear capacity; $v_{sws4} = v_s / 2 =$ **260** plf

$v_{dsws4} / v_{sws4} =$ **0.951**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **0.648** kips

Shear capacity for seismic loading; $V_s = \min(v_{sws1},v_{dsws1}) \times b_1 + \min(v_{sws2},v_{dsws2}) \times b_2 + v_{sws3} \times b_3 + \min(v_{sws4},v_{dsws4}) \times b_4 =$ **7.395** kips

$V_{s\_max} / V_s =$ **0.088**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; $h / b_1 =$ **1.245**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **0.963** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 =$ **0.988** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 - P =$ **-0.729** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-36** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.030**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **0.722** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + + 0.75 \times L_r) \times s / 2 =$ **0.46** kips

Maximum compressive force in chord; $C = V \times (k_1 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 + P =$ **0.654** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **26** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1252** lb/in^2

$f_c / F_c' =$ **0.021**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.042**

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

Chord capacity for chords 3 and 4

Shear wall aspect ratio; $h / b_2 =$ **1.011**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **0.963** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.216** kips

Maximum tensile force in chord; $T = V \times (k_2 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_2 - P =$ **-0.942** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-47** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.039**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{0.722}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 = \mathbf{0.46}$ kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_1,k_2,k_3,k_4)) \times h / b_2 + P = \mathbf{0.665}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{27}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{1252}$ lb/in^2

$f_c / F_c' = \mathbf{0.021}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.043}$

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

Chord capacity for chords 5 and 6

Shear wall aspect ratio; $h / b_3 = \mathbf{1.011}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{0.963}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 = \mathbf{1.216}$ kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 - P = \mathbf{-0.942}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-47}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in^2

$f_t / F_t' = \mathbf{-0.039}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{0.722}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 = \mathbf{0.46}$ kips

Maximum compressive force in chord; $C = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 + P = \mathbf{0.665}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{27}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{1252}$ lb/in^2

$f_c / F_c' = \mathbf{0.021}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.043}$

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

Chord capacity for chords 7 and 8

Shear wall aspect ratio; $h / b_4 = \mathbf{1.221}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{0.963}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 = \mathbf{1.007}$ kips

Maximum tensile force in chord; $T = V \times (k_4 / sum(k_1,k_2,k_3,k_4)) \times h / b_4 - P = \mathbf{-0.747}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-37}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in^2

$f_t / F_t' = \mathbf{-0.031}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{0.722}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 = \mathbf{0.46}$ kips

Maximum compressive force in chord; $C = V \times (k_4 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_4 + P = \mathbf{0.655}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{26}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{1252}$ lb/in^2

$f_c / F_c' = \mathbf{0.021}$

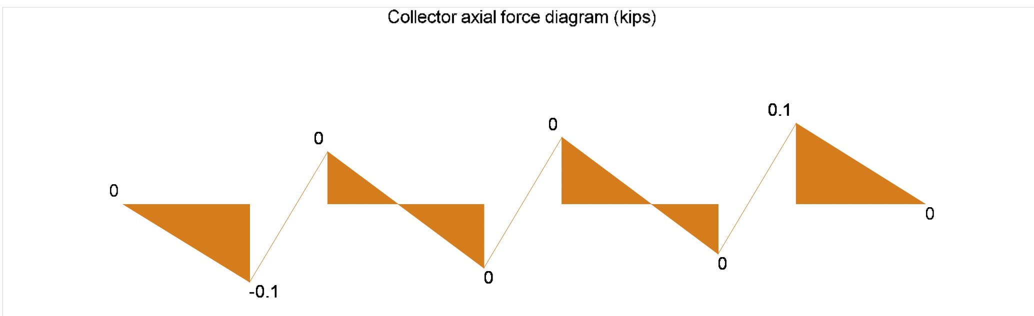
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.042}$

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

Collector capacity



Collector seismic design force factor; $F_{Coll} = \mathbf{1}$

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max}) = \mathbf{0.963}$ kips

Maximum force in collector; $P_{coll} = \mathbf{0.058}$ kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) = \mathbf{4}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1196}$ lb/in^2

$f_t / F_t' = \mathbf{0.003}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) = \mathbf{4}$ lb/in^2

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

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{2376}$ lb/in^2

$f_c / F_c' = \mathbf{0.001}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W = \mathbf{1.605}$ kips

Deflection limit; $\Delta_{w\_allow} = h / 360 = \mathbf{0.27}$ in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \mathrm{sum}(k_1,k_2,k_3,k_4)) / b_1 = \mathbf{53.35}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.03** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.11**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / sum(k_1,k_2,k_3,k_4)) / b_2 =$ **56.43** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.031** in

$\delta_{sww2} / \Delta_{w\_allow} =$ **0.116**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / sum(k_1,k_2,k_3,k_4)) / b_3 =$ **56.43** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.031** in

$\delta_{sww3} / \Delta_{w\_allow} =$ **0.116**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_4 / sum(k_1,k_2,k_3,k_4)) / b_4 =$ **53.64** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{sww4} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_4 =$ **0.03** in

$\delta_{sww4} / \Delta_{w\_allow} =$ **0.11**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **0.925** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **1.942** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / sum(k_1,k_2,k_3,k_4)) / b_1 =$ **30.75** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.017** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e =$ **0.068** in

$\delta_{sws1} / \Delta_{s\_allow} =$ **0.035**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / sum(k_1,k_2,k_3,k_4)) / b_2 =$ **32.52** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.018** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.072** in

$\delta_{sws2} / \Delta_{s\_allow} =$ **0.037**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / sum(k_1,k_2,k_3,k_4)) / b_3 =$ **32.52** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.018** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e =$ **0.072** in

$\delta_{sws3} / \Delta_{s\_allow} =$ **0.037**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_4 / sum(k_1,k_2,k_3,k_4)) / b_4 =$ **30.92** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{swse4} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_4 =$ **0.017** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws4} = C_{d\delta} \times \delta_{swse4} / I_e =$ **0.069** in

$\delta_{sws4} / \Delta_{s\_allow} =$ **0.035**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

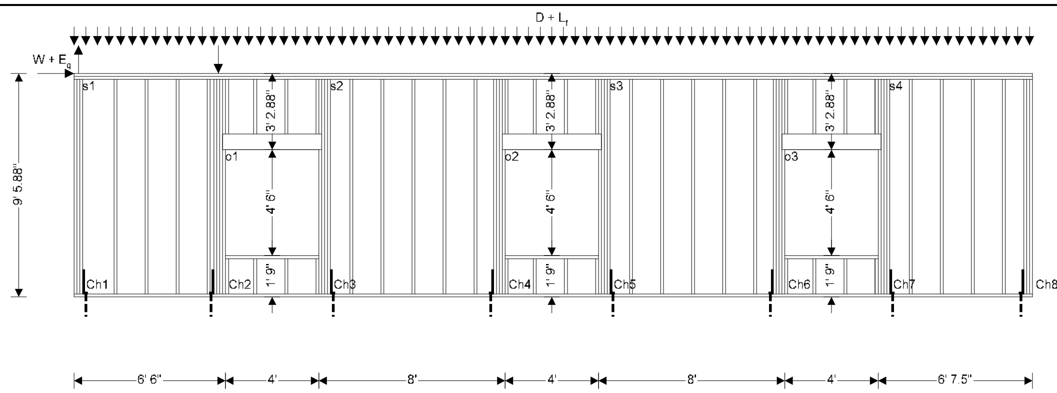
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	7373	1137	0.154	PASS
Chord capacity	lb/in^2	976	67	0.106	PASS
Collector capacity	lb/in^2	1196	6	0.005	PASS
Deflection	in	0.316	0.059	0.186	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **41.125** ft



Panel opening details

Width of opening;	w_{o1} = **4** ft
Height of opening;	h_{o1} = **4.5** ft
Height to underside of lintel over opening;	l_{o1} = **6.25** ft
Position of opening;	P_{o1} = **6.5** ft
Width of opening;	w_{o2} = **4** ft
Height of opening;	h_{o2} = **4.5** ft
Height to underside of lintel over opening;	l_{o2} = **6.25** ft
Position of opening;	P_{o2} = **18.5** ft
Width of opening;	w_{o3} = **4** ft
Height of opening;	h_{o3} = **4.5** ft
Height to underside of lintel over opening;	l_{o3} = **6.25** ft
Position of opening;	P_{o3} = **30.5** ft
Total area of wall;	$A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3}$ = **336.276** 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;	3 x 2" x 6"
Dressed end post size;	3 x 1.5" x 5.5"
Cross-sectional area of end posts;	A_e = **24.75** in^2
Hole diameter;	Dia = **1** in
Net cross-sectional area of end posts;	A_{en} = **20.25** 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
Vertical anchor stiffness;	k_a = **49087** lb/in

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

Species, grade and size classification;	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity;	G = **0.50**

Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **323.175** lb/ft
Floor live load acting on top of panel; L_f = **517.08** lb/ft
Self weight of panel; S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel; W = **2553** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **1624** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-432;	-249;	338;	1647;	0;	162;	0;	0;
Ch2;	432;	249;	338;	1647;	0;	162;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;
Ch7;	0;	0;	0;	0;	0;	0;	0;	0;
Ch8;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1112** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **6.5** ft

Shear wall aspect ratio; h / b_1 = **1.46**

Segment 2 wall length; b_2 = **8** ft

Shear wall aspect ratio; h / b_2 = **1.186**

Segment 3 wall length; b_3 = **8** ft

Shear wall aspect ratio; h / b_3 = **1.186**

Segment 4 wall length; b_4 = **6.625** ft

Shear wall aspect ratio; h / b_4 = **1.432**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **0.024** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **9.054** kips/in

Unit shear capacity, widest segment; $v_{sww3} = v_w / 2$ = **365** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww3} / k_a$ = **0.071** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww3} \times h^3 / (3 \times E \times A_e \times b_3) + v_{sww3} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_3$ = **0.323** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **0.030** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **6.904** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1$ = **342.55** plf

Segment 1 shear capacity; $v_{sww1} = v_w / 2$ = **365** plf

v_{dsww1} / v_{sww1} = **0.938**

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

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.024** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **9.054** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2$ = **365** plf

Segment 2 shear capacity; $v_{sww2} = v_w / 2$ = **365** plf

v_{dsww2} / v_{sww2} = **1.000**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4)$ = **0.029** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4)$ = **7.081** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsww4} = \delta_{Cap} \times k_4 / b_4$ = **344.68** plf

Segment 4 shear capacity; $v_{sww4} = v_w / 2$ = **365** plf

v_{dsww4} / v_{sww4} = **0.944**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **1.532** kips

Shear capacity for wind loading; $V_w = \min(v_{sww1},v_{dsww1}) \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + v_{sww3} \times b_3 + \min(v_{sww4},v_{dsww4}) \times b_4$ = **10.35** kips

V_{w_max} / V_w = **0.148**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **0.024** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **9.054** kips/in

Unit shear capacity, widest segment; $v_{sws3} = v_s / 2$ = **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws3} / k_a$ = **0.050** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws3} \times h^3 / (3 \times E \times A_e \times b_3) + v_{sws3} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_3$ = **0.23** in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **0.030** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **6.904** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsws1} = \delta_{Cap} \times k_1 / b_1$ = **244.01** plf

Segment 1 shear capacity; $v_{sws1} = v_s / 2$ = **260** plf

v_{dsws1} / v_{sws1} = **0.938**

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

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.024** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **9.054** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsws2} = \delta_{Cap} \times k_2 / b_2$ = **260** plf

Segment 2 shear capacity; $v_{sws2} = v_s / 2$ = **260** plf

v_{dsws2} / v_{sws2} = **1.000**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4)$ = **0.029** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4)$ = **7.081** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsws4} = \delta_{Cap} \times k_4 / b_4 =$ **245.53** plf

Segment 4 shear capacity; $v_{sws4} = v_s / 2 =$ **260** plf

$v_{dsws4} / v_{sws4} =$ **0.944**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **1.137** kips

Shear capacity for seismic loading; $V_s = \min(v_{sws1},v_{dsws1}) \times b_1 + \min(v_{sws2},v_{dsws2}) \times b_2 + v_{sws3} \times b_3 + \min(v_{sws4},v_{dsws4}) \times b_4 =$ **7.373** kips

$V_{s\_max} / V_s =$ **0.154**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; $h / b_1 =$ **1.46**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1} =$ **1.822** kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 - P =$ **-1.341** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-66** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.055**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + -1 \times 0.45 \times W_{ch1} + D_{C\_ch1} + 0.75 \times L_{r\_ch1} =$ **1.286** kips

Maximum compressive force in chord; $C = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 + P =$ **1.647** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **67** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.068**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.106**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **1.822** kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 - P =$ **-1.341** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-66** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.055**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + 0.45 \times W_{ch2} + D_{C\_ch2} + 0.75 \times L_{r\_ch2} =$ **1.286** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_1,k_2,k_3,k_4)) \times h / b_1 + P =$ **1.647** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **67** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.068**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.106**

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

Chord capacity for chord 3

Shear wall aspect ratio; $h / b_2 =$ **1.186**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.345** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_1,k_2,k_3,k_4)) \times h / b_2 - P =$ **-0.832** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-41** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.034**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_1,k_2,k_3,k_4)) \times h / b_2 + P =$ **1.017** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **41** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.042**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.066**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.345** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_1,k_2,k_3,k_4)) \times h / b_2 - P =$ **-0.832** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-41** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.034**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_1,k_2,k_3,k_4)) \times h / b_2 + P =$ **1.017** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **41** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.042**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.066**

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

Chord capacity for chord 5

Shear wall aspect ratio; $h / b_3 =$ **1.186**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.345** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 - P =$ **-0.832** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-41** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.034**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 + P =$ **1.017** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **41** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.042**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.066**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.345** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 - P =$ **-0.832** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-41** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.034**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 + P =$ **1.017** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **41** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.042**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.066**

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

Chord capacity for chord 7

Shear wall aspect ratio; h / b_4 = **1.432**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2$ = **1.114** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_4 - P$ = **-0.630** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-31** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.026**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.632** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_4 + P$ = **0.995** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **40** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.041**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.064**

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

Chord capacity for chord 8

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **1.532** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2$ = **1.114** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_4 - P$ = **-0.630** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-31** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.026**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **1.149** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.632** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_4 + P$ = **0.995** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **40** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.041**

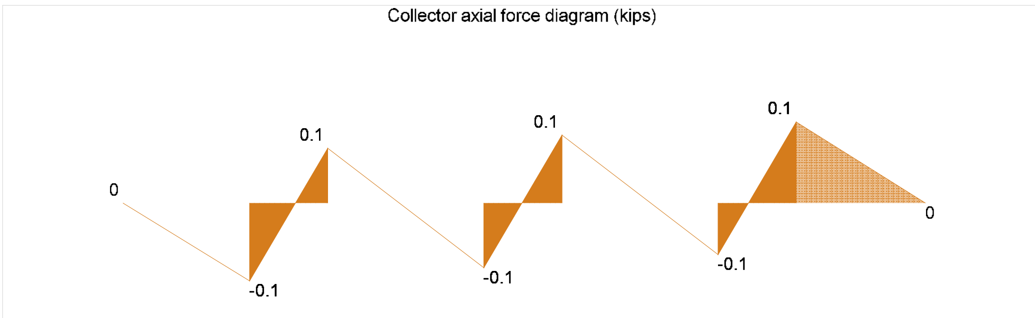
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.064**

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

Collector capacity



Collector seismic design force factor; F_{Coll} = **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **1.532** kips

Maximum force in collector; P_{coll} = **0.091** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s)$ = **6** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.005**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s)$ = **6** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2376** lb/in^2

f_c / F_c' = **0.002**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **2.553** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \mathrm{sum}(k_1,k_2,k_3,k_4)) / b_1$ = **84.5** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1}, D_{T\_ch2})$ $+ \max(\mathrm{abs}(W_{ch1}), \mathrm{abs}(W_{ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.056** in

$\delta_{sww1} / \Delta_{w\_allow}$ = **0.176**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \mathrm{sum}(k_1,k_2,k_3,k_4)) / b_2$ = **90.03** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.059** in

$\delta_{sww2} / \Delta_{w\_allow} =$ **0.186**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / sum(k_1,k_2,k_3,k_4)) / b_3 =$ **90.03** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.059** in

$\delta_{sww3} / \Delta_{w\_allow} =$ **0.186**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_4 / sum(k_1,k_2,k_3,k_4)) / b_4 =$ **85.02** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{sww4} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_4 =$ **0.056** in

$\delta_{sww4} / \Delta_{w\_allow} =$ **0.177**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **1.624** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / sum(k_1,k_2,k_3,k_4)) / b_1 =$ **53.75** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times min(D_{T\_ch1},D_{T\_ch2}) + max(abs(E_{q\_ch1}),abs(E_{q\_ch2}))) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.035** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e =$ **0.142** in

$\delta_{sws1} / \Delta_{s\_allow} =$ **0.062**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / sum(k_1,k_2,k_3,k_4)) / b_2 =$ **57.27** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.037** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.15** in

$\delta_{sws2} / \Delta_{s\_allow} =$ **0.066**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \text{sum}(k_1,k_2,k_3,k_4)) / b_3$ = **57.27** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.037** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e$ = **0.15** in

$\delta_{sws3} / \Delta_{s\_allow}$ = **0.066**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_4 / \text{sum}(k_1,k_2,k_3,k_4)) / b_4$ = **54.08** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{swse4} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_4$ = **0.036** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws4} = C_{d\delta} \times \delta_{swse4} / I_e$ = **0.143** in

$\delta_{sws4} / \Delta_{s\_allow}$ = **0.063**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

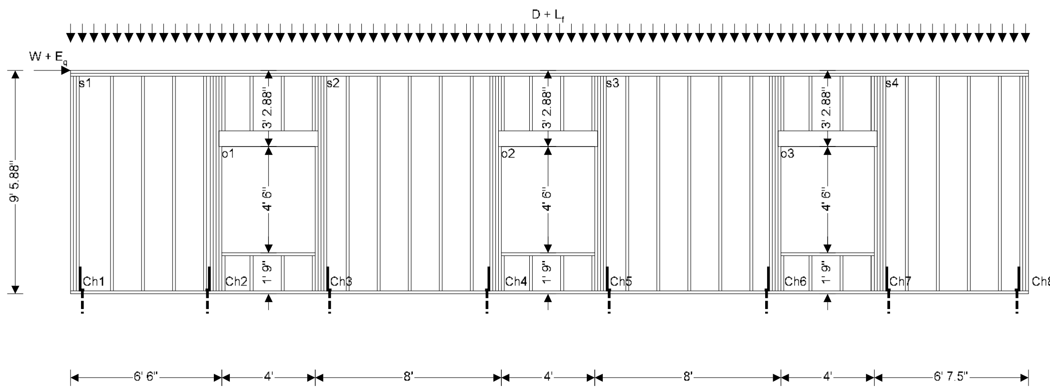
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	10350	2146	0.207	PASS
Chord capacity	lb/in^2	976	47	0.076	PASS
Collector capacity	lb/in^2	1196	8	0.006	PASS
Deflection	in	0.285	0.083	0.290	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; $h = \mathbf{9.49}$ ft

Panel length; $b = \mathbf{41.125}$ ft



Panel opening details

Width of opening; $w_{o1} = \mathbf{4}$ ft

Height of opening; $h_{o1} = \mathbf{4.5}$ ft

Height to underside of lintel over opening; $l_{o1} = \mathbf{6.25}$ ft

Position of opening; $P_{o1} = \mathbf{6.5}$ ft

Width of opening; $w_{o2} = \mathbf{4}$ ft

Height of opening; $h_{o2} = \mathbf{4.5}$ ft

Height to underside of lintel over opening; $l_{o2} = \mathbf{6.25}$ ft

Position of opening; $P_{o2} = \mathbf{18.5}$ ft

Width of opening; $w_{o3} = \mathbf{4}$ ft

Height of opening; $h_{o3} = \mathbf{4.5}$ ft

Height to underside of lintel over opening; $l_{o3} = \mathbf{6.25}$ ft

Position of opening; $P_{o3} = \mathbf{30.5}$ ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3} = \mathbf{336.276}$ 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; 3 x 2" x 6"
Dressed end post size; 3 x 1.5" x 5.5"
Cross-sectional area of end posts; A_e = **24.75** in^2
Hole diameter; Dia = **1** in
Net cross-sectional area of end posts; A_{en} = **20.25** 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
Vertical anchor stiffness; k_a = **49087** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **323.175** lb/ft
Floor live load acting on top of panel; L_f = **517.08** lb/ft
Self weight of panel; S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel; W = **3577** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **2075** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	0;	0;	0;	0;	0;	0;	0;	0;
Ch2;	0;	0;	0;	0;	0;	0;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;

Ch7;	0;	0;	0;	0;	0;	0;	0;	0;
Ch8;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1112** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **6.5** ft

Shear wall aspect ratio; h / b_1 = **1.46**

Segment 2 wall length; b_2 = **8** ft

Shear wall aspect ratio; h / b_2 = **1.186**

Segment 3 wall length; b_3 = **8** ft

Shear wall aspect ratio; h / b_3 = **1.186**

Segment 4 wall length; b_4 = **6.625** ft

Shear wall aspect ratio; h / b_4 = **1.432**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) = \mathbf{0.024}$ in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) = \mathbf{9.054}$ kips/in

Unit shear capacity, widest segment; $v_{sww3} = v_w / 2 = \mathbf{365}$ plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww3} / k_a = \mathbf{0.071}$ in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww3} \times h^3 / (3 \times E \times A_e \times b_3) + v_{sww3} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_3 = \mathbf{0.323}$ in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) = \mathbf{0.030}$ in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) = \mathbf{6.904}$ kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsww1} = \delta_{Cap} \times k_1 / b_1 = \mathbf{342.55}$ plf

Segment 1 shear capacity; $v_{sww1} = v_w / 2 = \mathbf{365}$ plf

$v_{dsww1} / v_{sww1} = \mathbf{0.938}$

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

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) = \mathbf{0.024}$ in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) = \mathbf{9.054}$ kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2 = \mathbf{365}$ plf

Segment 2 shear capacity; $v_{sww2} = v_w / 2 = \mathbf{365}$ plf

$v_{dsww2} / v_{sww2} = \mathbf{1.000}$

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) = \mathbf{0.029}$ in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) = \mathbf{7.081}$ kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsww4} = \delta_{Cap} \times k_4 / b_4 = \mathbf{344.68}$ plf

Segment 4 shear capacity; $v_{sww4} = v_w / 2 = \mathbf{365}$ plf

$v_{dsww4} / v_{sww4} = \mathbf{0.944}$

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W = \mathbf{2.146}$ kips

Shear capacity for wind loading; $V_w = \min(v_{sww1},v_{dsww1}) \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + v_{sww3} \times b_3 + \min(v_{sww4},v_{dsww4}) \times b_4 = \mathbf{10.35}$ kips

$V_{w\_max} / V_w = \mathbf{0.207}$

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) = \mathbf{0.024}$ in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) = \mathbf{9.054}$ kips/in

Unit shear capacity, widest segment; $v_{sws3} = v_s / 2 = \mathbf{260}$ plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws3} / k_a = \mathbf{0.050}$ in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws3} \times h^3 / (3 \times E \times A_e \times b_3) + v_{sws3} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_3 = \mathbf{0.23}$ in

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) = \mathbf{0.030}$ in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **6.904** kips/in

Segment 1 unit shear at δ_{Cap}; $v_{dsws1} = \delta_{Cap} \times k_1 / b_1$ = **244.01** plf

Segment 1 shear capacity; $v_{sws1} = v_s / 2$ = **260** plf

v_{dsws1} / v_{sws1} = **0.938**

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

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.024** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **9.054** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsws2} = \delta_{Cap} \times k_2 / b_2$ = **260** plf

Segment 2 shear capacity; $v_{sws2} = v_s / 2$ = **260** plf

v_{dsws2} / v_{sws2} = **1.000**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4)$ = **0.029** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4)$ = **7.081** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsws4} = \delta_{Cap} \times k_4 / b_4$ = **245.53** plf

Segment 4 shear capacity; $v_{sws4} = v_s / 2$ = **260** plf

v_{dsws4} / v_{sws4} = **0.944**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **1.453** kips

Shear capacity for seismic loading; $V_s = \min(v_{sws1}, v_{dsws1}) \times b_1 + \min(v_{sws2}, v_{dsws2}) \times b_2 + v_{sws3} \times b_3 + \min(v_{sws4}, v_{dsws4}) \times b_4$ = **7.373** kips

V_{s_max} / V_s = **0.197**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b_1 = **1.46**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.093** kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 - P$ = **-0.419** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-21** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.017**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.632** kips

Maximum compressive force in chord; $C = V \times (k_1 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 + P$ = **1.138** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.047**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.074**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.093** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 - P$ = **-0.419** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-21** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.017**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.632** kips

Maximum compressive force in chord; $C = V \times (k_1 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_1 + P$ = **1.138** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.047**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.074**

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

Chord capacity for chord 3

Shear wall aspect ratio; h / b_2 = **1.186**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.345** kips

Maximum tensile force in chord; $T = V \times (k_2 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_2 - P$ = **-0.627** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-31** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **-0.026**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.632** kips

Maximum compressive force in chord; $C = V \times (k_2 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_2 + P$ = **1.171** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **47** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **976** lb/in^2

f_c / F_c' = **0.048**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.076**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.345** kips

Maximum tensile force in chord; $T = V \times (k_2 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_2 - P =$ **-0.627** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-31** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.026**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_2 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_2 + P =$ **1.171** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **47** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.048**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.076**

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

Chord capacity for chord 5

Shear wall aspect ratio; $h / b_3 =$ **1.186**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.345** kips

Maximum tensile force in chord; $T = V \times (k_3 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_3 - P =$ **-0.627** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-31** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.026**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_1,k_2,k_3,k_4)) \times h / b_3 + P =$ **1.171** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **47** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.048**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.076**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.345** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 - P =$ **-0.627** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-31** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.026**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_3 / sum(k_1,k_2,k_3,k_4)) \times h / b_3 + P =$ **1.171** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **47** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.048**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.076**

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

Chord capacity for chord 7

Shear wall aspect ratio; $h / b_4 =$ **1.432**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 =$ **1.114** kips

Maximum tensile force in chord; $T = V \times (k_4 / sum(k_1,k_2,k_3,k_4)) \times h / b_4 - P =$ **-0.436** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-22** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.018**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_4 / sum(k_1,k_2,k_3,k_4)) \times h / b_4 + P =$ **1.141** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.047**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.074**

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

Chord capacity for chord 8

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.146** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 =$ **1.114** kips

Maximum tensile force in chord; $T = V \times (k_4 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_4 - P =$ **-0.436** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-22** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.018**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **1.61** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.632** kips

Maximum compressive force in chord; $C = V \times (k_4 / \mathrm{sum}(k_1,k_2,k_3,k_4)) \times h / b_4 + P =$ **1.141** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.047**

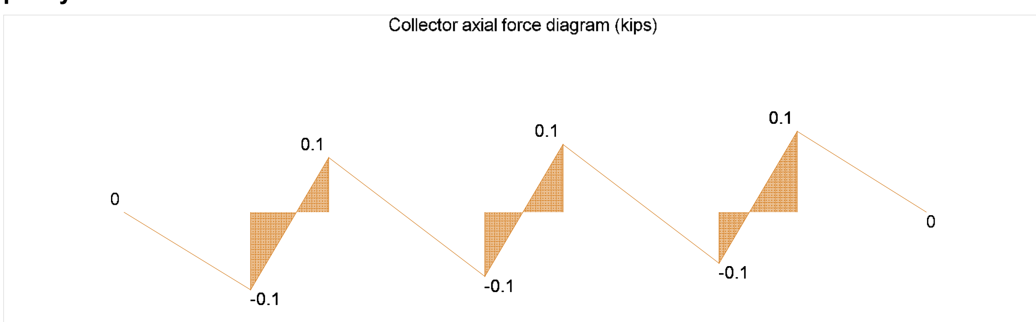
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.074**

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

Collector capacity



Collector seismic design force factor; $F_{Coll} =$ **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max}) =$ **2.146** kips

Maximum force in collector; $P_{coll} =$ **0.128** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) =$ **8** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **0.006**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) =$ **8** lb/in^2

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

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2376** lb/in^2

$f_c / F_c' =$ **0.003**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **3.577** kips

Deflection limit; $\Delta_{w\_allow} = h / 400 =$ **0.285** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \text{sum}(k_1,k_2,k_3,k_4)) / b_1 =$ **118.39** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.078** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.274**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \text{sum}(k_1,k_2,k_3,k_4)) / b_2 =$ **126.14** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.083** in

$\delta_{sww2} / \Delta_{w\_allow} =$ **0.29**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \text{sum}(k_1,k_2,k_3,k_4)) / b_3 =$ **126.14** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.083** in

$\delta_{sww3} / \Delta_{w\_allow} =$ **0.29**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_4 / \text{sum}(k_1,k_2,k_3,k_4)) / b_4 =$ **119.12** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{sww4} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_4 =$ **0.078** in

$\delta_{sww4} / \Delta_{w\_allow} =$ **0.276**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **2.075** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / \text{sum}(k_1,k_2,k_3,k_4)) / b_1 =$ **68.67** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.045** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e =$ **0.181** in

$\delta_{sws1} / \Delta_{s\_allow} =$ **0.08**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / sum(k_1,k_2,k_3,k_4)) / b_2 =$ **73.18** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.048** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.192** in

$\delta_{sws2} / \Delta_{s\_allow} =$ **0.084**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / sum(k_1,k_2,k_3,k_4)) / b_3 =$ **73.18** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.048** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e =$ **0.192** in

$\delta_{sws3} / \Delta_{s\_allow} =$ **0.084**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_4 / sum(k_1,k_2,k_3,k_4)) / b_4 =$ **69.1** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{swse4} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_4 =$ **0.046** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws4} = C_{d\delta} \times \delta_{swse4} / I_e =$ **0.182** in

$\delta_{sws4} / \Delta_{s\_allow} =$ **0.08**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

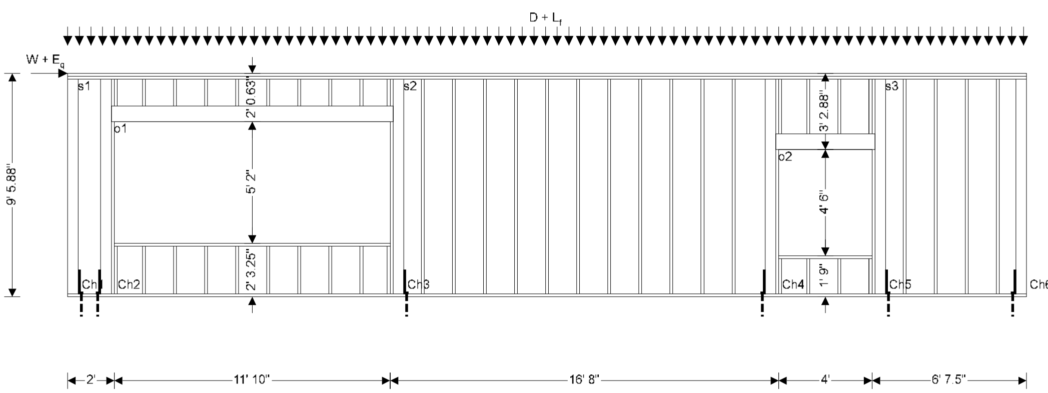
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	8130	2761	0.340	PASS
Chord capacity	lb/in^2	976	52	0.083	PASS
Collector capacity	lb/in^2	1196	56	0.047	PASS
Deflection	in	0.316	0.132	0.419	PASS
DESIGN WARNING - External chord forces are applied to a segment that exceeds the maximum aspect ratio permitted. The segment chords are not designed.					

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **41.125** ft



Panel opening details

Width of opening; w_{o1} = **11.833** ft

Height of opening; h_{o1} = **5.167** ft

Height to underside of lintel over opening; l_{o1} = **7.438** ft

Position of opening; P_{o1} = **2** ft

Width of opening; w_{o2} = **4** ft

Height of opening; h_{o2} = **4.5** ft

Height to underside of lintel over opening; l_{o2} = **6.25** ft

Position of opening; P_{o2} = **30.5** ft

Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2}$ = **311.137** 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; 6" x 6"
Dressed end post size; 5.5" x 5.5"
Cross-sectional area of end posts; A_e = **30.25** in^2
Hole diameter; Dia = **1** in
Net cross-sectional area of end posts; A_{en} = **24.75** 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
Vertical anchor stiffness; k_a = **69646** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2
Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing
Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft
Nominal unit shear capacity for wind design; v_w = **730** lb/ft
Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **353.175** lb/ft
Floor live load acting on top of panel; L_f = **517.08** lb/ft
Self weight of panel; S_{wt} = **25** lb/ft^2
In plane wind load acting at head of panel; W = **4601** lbs
Wind load serviceability factor; f_{Wserv} = **1.00**
In plane seismic load acting at head of panel; E_q = **2247** lbs
Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	0;	0;	0;	0;	0;	0;	0;	0;
Ch2;	0;	0;	0;	0;	0;	0;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1112** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.41**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **2** ft

Shear wall aspect ratio; h / b_1 = **4.745**

Segment 2 wall length; b_2 = **16.667** ft

Shear wall aspect ratio; h / b_2 = **0.569**

Segment 3 wall length; b_3 = **6.625** ft

Shear wall aspect ratio; h / b_3 = **1.432**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **0.008** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **23.189** kips/in

Unit shear capacity, widest segment; $v_{sww2} = v_w / 2$ = **365** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww2} / k_a$ = **0.050** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sww2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2 =$ **0.262** in

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.021** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **7.802** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 =$ **308.93** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2 =$ **365** plf

$v_{dsww3} / v_{sww3} =$ **0.846**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W =$ **2.761** kips

Shear capacity for wind loading; $V_w = v_{sww2} \times b_2 + \min(v_{sww3}, v_{dsww3}) \times b_3 =$ **8.13** kips

$V_{w\_max} / V_w =$ **0.34**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **0.008** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **23.189** kips/in

Unit shear capacity, widest segment; $v_{sws2} = v_s / 2 =$ **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws2} / k_a =$ **0.035** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws2} \times h^3 / (3 \times E \times A_e \times b_2) + v_{sws2} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_2 =$ **0.187** in

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **0.021** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **7.802** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3 =$ **220.06** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2 =$ **260** plf

$v_{dsws3} / v_{sws3} =$ **0.846**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **1.573** kips

Shear capacity for seismic loading; $V_s = v_{sws2} \times b_2 + \min(v_{sws3}, v_{dsws3}) \times b_3 =$ **5.791** kips

$V_{s\_max} / V_s =$ **0.272**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; $h / b_1 =$ **4.745**

Segment not considered, shear wall aspect ratio exceeds maximum allowable.

Chord capacity for chord 3

Shear wall aspect ratio; $h / b_2 =$ **0.569**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.761** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **2.952** kips

Maximum tensile force in chord; $T = V \times (k_2 / \text{sum}(k_2,k_3)) \times h / b_2 - P =$ **-1.776** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-72** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

f_t / F_t' = **-0.060**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 × W = **2.761** kips

Axial force for maximum compression; P = ((D + S_{wt} × h)) × s / 2 = **0.394** kips

Maximum compressive force in chord; C = V × (k_2 / sum(k_2,k_3)) × h / b_2 + P = **1.570** kips

Maximum applied compressive stress; f_c = C / A_e = **52** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **976** lb/in^2

f_c / F_c' = **0.053**

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 = **625** lb/in^2

f_c / F_{c_perp}' = **0.083**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **2.761** kips

Axial force for maximum tension; P = (0.6 × (D + S_{wt} × h)) × b_2 / 2 = **2.952** kips

Maximum tensile force in chord; T = V × (k_2 / sum(k_2,k_3)) × h / b_2 - P = **-1.776** kips

Maximum applied tensile stress; f_t = T / A_{en} = **-72** lb/in^2

Design tensile stress; F_t' = F_t × C_D × C_{Mt} × C_{tt} × C_{Ft} × C_i = **1196** lb/in^2

f_t / F_t' = **-0.060**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 1

Shear force for maximum compression; V = 0.6 × W = **2.761** kips

Axial force for maximum compression; P = ((D + S_{wt} × h)) × s / 2 = **0.394** kips

Maximum compressive force in chord; C = V × (k_2 / sum(k_2,k_3)) × h / b_2 + P = **1.570** kips

Maximum applied compressive stress; f_c = C / A_e = **52** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **976** lb/in^2

f_c / F_c' = **0.053**

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 = **625** lb/in^2

f_c / F_{c_perp}' = **0.083**

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

Chord capacity for chord 5

Shear wall aspect ratio; h / b_3 = **1.432**

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **2.761** kips

Axial force for maximum tension; P = (0.6 × (D + S_{wt} × h)) × b_3 / 2 = **1.173** kips

Maximum tensile force in chord; T = V × (k_3 / sum(k_2,k_3)) × h / b_3 - P = **-0.178** kips

Maximum applied tensile stress; f_t = T / A_{en} = **-7** lb/in^2

Design tensile stress; F_t' = F_t × C_D × C_{Mt} × C_{tt} × C_{Ft} × C_i = **1196** lb/in^2

f_t / F_t' = **-0.006**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.07** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.652** kips

Maximum compressive force in chord; $C = V \times (k_3 / sum(k_2,k_3)) \times h / b_3 + P =$ **1.399** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.047**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.074**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **2.761** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.173** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_2,k_3)) \times h / b_3 - P =$ **-0.178** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-7** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1196** lb/in^2

$f_t / F_t' =$ **-0.006**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.07** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.652** kips

Maximum compressive force in chord; $C = V \times (k_3 / sum(k_2,k_3)) \times h / b_3 + P =$ **1.399** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **46** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **976** lb/in^2

$f_c / F_c' =$ **0.047**

PASS - Design compressive stress exceeds maximum applied compressive stress

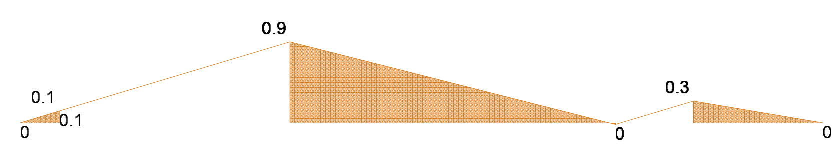
Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.074**

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

Collector capacity

Collector axial force diagram (kips)



Collector seismic design force factor; F_{Coll} = **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **2.761** kips

Maximum force in collector; P_{coll} = **0.929** kips

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

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1196** lb/in^2

f_t / F_t' = **0.047**

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; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2376** lb/in^2

f_c / F_c' = **0.024**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 3; T_3 = **-1.776** kips

Chord 4; T_4 = **-1.776** kips

Chord 5; T_5 = **-0.178** kips

Chord 6; T_6 = **-0.178** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1;	0;	0;	394;	590;	345;	0;	0;	0;
Ch2;	0;	0;	394;	590;	345;	0;	0;	0;
Ch3;	-1960;	-957;	394;	4920;	345;	0;	0;	0;
Ch4;	1960;	957;	394;	4920;	345;	0;	0;	0;
Ch5;	-1659;	-810;	394;	1956;	345;	0;	0;	0;
Ch6;	1659;	810;	394;	1956;	345;	0;	0;	0;

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **4.601** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \mathrm{sum}(k_2,k_3)) / b_2$ = **206.56** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.132** in

$\delta_{sww2} / \Delta_{w\_allow}$ = **0.419**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \mathrm{sum}(k_2,k_3)) / b_3$ = **174.83** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.114** in

$\delta_{sww3} / \Delta_{w\_allow}$ = **0.361**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **2.247** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **2.278** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / \text{sum}(k_2,k_3)) / b_2 =$ **100.88** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.065** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.259** in

$\delta_{sws2} / \Delta_{s\_allow} =$ **0.114**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \text{sum}(k_2,k_3)) / b_3 =$ **85.38** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.056** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e =$ **0.223** in

$\delta_{sws3} / \Delta_{s\_allow} =$ **0.098**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

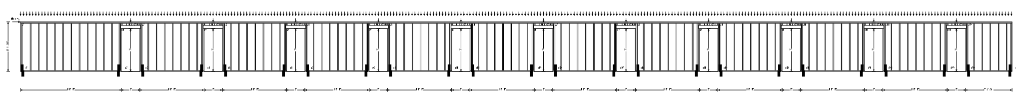
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	31692	3650	0.115	PASS
Chord capacity	lb/in^2	584	70	0.120	PASS
Collector capacity	lb/in^2	1380	31	0.022	PASS
Deflection	in	0.270	0.039	0.144	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **8.09** ft

Panel length; b = **162.104** ft



Panel opening details

Width of opening; w_{o1} = **3** ft

Height of opening; h_{o1} = **7** ft

Height to underside of lintel over opening; l_{o1} = **7** ft

Position of opening; P_{o1} = **16.5** ft

Width of opening; w_{o2} = **3** ft

Height of opening; h_{o2} = **7** ft

Height to underside of lintel over opening; l_{o2} = **7** ft

Position of opening; P_{o2} = **30** ft

Width of opening; w_{o3} = **3** ft

Height of opening; h_{o3} = **7** ft

Height to underside of lintel over opening; l_{o3} = **7** ft

Position of opening; P_{o3} = **43.5** ft

Width of opening; w_{o4} = **3** ft

Height of opening; h_{o4} = **7** ft

Height to underside of lintel over opening; l_{o4} = **7** ft

Position of opening; P_{o4} = **57** ft

Width of opening; w_{o5} = **3** ft

Height of opening; h_{o5} = **7** ft

Height to underside of lintel over opening; l_{o5} = **7** ft

Position of opening; P_{o5} = **70.5** ft

Width of opening; w_{o6} = **3** ft

Height of opening; h_{o6} = **7** ft

Height to underside of lintel over opening; l_{o6} = **7** ft

Position of opening; P_{o6} = **84** ft

Width of opening;	w_{o7} = **3** ft
Height of opening;	h_{o7} = **7** ft
Height to underside of lintel over opening;	l_{o7} = **7** ft
Position of opening;	P_{o7} = **97.5** ft
Width of opening;	w_{o8} = **3** ft
Height of opening;	h_{o8} = **7** ft
Height to underside of lintel over opening;	l_{o8} = **7** ft
Position of opening;	P_{o8} = **111** ft
Width of opening;	w_{o9} = **3** ft
Height of opening;	h_{o9} = **7** ft
Height to underside of lintel over opening;	l_{o9} = **7** ft
Position of opening;	P_{o9} = **124.5** ft
Width of opening;	w_{o10} = **3** ft
Height of opening;	h_{o10} = **7** ft
Height to underside of lintel over opening;	l_{o10} = **7** ft
Position of opening;	P_{o10} = **138** ft
Width of opening;	w_{o11} = **3** ft
Height of opening;	h_{o11} = **7** ft
Height to underside of lintel over opening;	l_{o11} = **7** ft
Position of opening;	P_{o11} = **151.5** ft
Total area of wall;	$A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3} - w_{o4} \times h_{o4} - w_{o5} \times h_{o5} - w_{o6} \times h_{o6} -$

$w_{o7} \times h_{o7} - w_{o8} \times h_{o8} - w_{o9} \times h_{o9} - w_{o10} \times h_{o10} - w_{o11} \times h_{o11}$ = **1080.423** 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
Vertical anchor stiffness;	k_a = **1** lb/in

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

Species, grade and size classification;	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity;	G = **0.50**
Tension parallel to grain;	F_t = **575** lb/in^2
Compression parallel to grain;	F_c = **1350** lb/in^2
Compression perpendicular to grain;	F_{c_perp} = **625** lb/in^2
Modulus of elasticity;	E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **395.31** lb/ft

Roof live load acting on top of panel; L_r = **316.248** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **6083** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **2842** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **620** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.24**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **16.5** ft

Shear wall aspect ratio; h / b_1 = **0.49**

Segment 2 wall length; b_2 = **10.5** ft

Shear wall aspect ratio; h / b_2 = **0.77**

Segment 3 wall length; b_3 = **10.5** ft

Shear wall aspect ratio; h / b_3 = **0.77**

Segment 4 wall length; b_4 = **10.5** ft

Shear wall aspect ratio; h / b_4 = **0.77**

Segment 5 wall length; b_5 = **10.5** ft

Shear wall aspect ratio; h / b_5 = **0.77**

Segment 6 wall length; b_6 = **10.5** ft

Shear wall aspect ratio; h / b_6 = **0.77**

Segment 7 wall length; b_7 = **10.5** ft

Shear wall aspect ratio; h / b_7 = **0.77**

Segment 8 wall length; b_8 = **10.5** ft

Shear wall aspect ratio; h / b_8 = **0.77**

Segment 9 wall length; b_9 = **10.5** ft

Shear wall aspect ratio; h / b_9 = **0.77**

Segment 10 wall length; b_{10} = **10.5** ft

Shear wall aspect ratio; h / b_{10} = **0.77**

Segment 11 wall length; b_{11} = **10.5** ft

Shear wall aspect ratio; h / b_{11} = **0.77**

Segment 12 wall length; b_{12} = **7.604** ft

Shear wall aspect ratio; h / b_{12} = **1.064**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **490.303** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **0.004** kips/in

Unit shear capacity, widest segment; $v_{sww1} = v_w / 2$ = **365** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww1} / k_a$ = **2952.850** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sww1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1$ = **1447.994** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **770.476** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **0.002** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2$ = **232.28** plf

Segment 2 shear capacity; $v_{sww2} = v_w / 2$ = **365** plf

v_{dsww2} / v_{sww2} = **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **770.476** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **0.002** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 =$ **232.28** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2 =$ **365** plf

$v_{dsww3} / v_{sww3} =$ **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) =$ **770.476** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) =$ **0.002** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsww4} = \delta_{Cap} \times k_4 / b_4 =$ **232.28** plf

Segment 4 shear capacity; $v_{sww4} = v_w / 2 =$ **365** plf

$v_{dsww4} / v_{sww4} =$ **0.636**

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

Segment 5 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_5) =$ **770.476** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5) =$ **0.002** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsww5} = \delta_{Cap} \times k_5 / b_5 =$ **232.28** plf

Segment 5 shear capacity; $v_{sww5} = v_w / 2 =$ **365** plf

$v_{dsww5} / v_{sww5} =$ **0.636**

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

Segment 6 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_6) =$ **770.476** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6) =$ **0.002** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsww6} = \delta_{Cap} \times k_6 / b_6 =$ **232.28** plf

Segment 6 shear capacity; $v_{sww6} = v_w / 2 =$ **365** plf

$v_{dsww6} / v_{sww6} =$ **0.636**

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

Segment 7 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_7) =$ **770.476** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7) =$ **0.002** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsww7} = \delta_{Cap} \times k_7 / b_7 =$ **232.28** plf

Segment 7 shear capacity; $v_{sww7} = v_w / 2 =$ **365** plf

$v_{dsww7} / v_{sww7} =$ **0.636**

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

Segment 8 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_8) =$ **770.476** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8) =$ **0.002** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsww8} = \delta_{Cap} \times k_8 / b_8 =$ **232.28** plf

Segment 8 shear capacity; $v_{sww8} = v_w / 2 =$ **365** plf

$v_{dsww8} / v_{sww8} =$ **0.636**

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

Segment 9 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_9) =$ **770.476** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9) =$ **0.002** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsww9} = \delta_{Cap} \times k_9 / b_9 =$ **232.28** plf

Segment 9 shear capacity; $v_{sww9} = v_w / 2 =$ **365** plf

$v_{dsww9} / v_{sww9} =$ **0.636**

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

Segment 10 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{10}) =$ **770.476** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10}) =$ **0.002** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsww10} = \delta_{Cap} \times k_{10} / b_{10} =$ **232.28** plf

Segment 10 shear capacity; $v_{sww10} = v_w / 2 =$ **365** plf

$v_{dsww10} / v_{sww10} =$ **0.636**

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

Segment 11 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{11}) =$ **770.476** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11}) =$ **0.002** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsww11} = \delta_{Cap} \times k_{11} / b_{11} =$ **232.28** plf

Segment 11 shear capacity; $v_{sww11} = v_w / 2 =$ **365** plf

$v_{dsww11} / v_{sww11} =$ **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12}) =$ **1063.886** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12}) =$ **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsww12} = \delta_{Cap} \times k_{12} / b_{12} =$ **168.23** plf

Segment 12 shear capacity; $v_{sww12} = v_w / 2 =$ **365** plf

$v_{dsww12} / v_{sww12} =$ **0.461**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W =$ **3.65** kips

Shear capacity for wind loading; $V_w = v_{sww1} \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + \min(v_{sww3},v_{dsww3}) \times b_3 + \min(v_{sww4},v_{dsww4}) \times b_4 + \min(v_{sww5},v_{dsww5}) \times b_5 + \min(v_{sww6},v_{dsww6}) \times b_6 + \min(v_{sww7},v_{dsww7}) \times b_7 + \min(v_{sww8},v_{dsww8}) \times b_8 + \min(v_{sww9},v_{dsww9}) \times b_9 + \min(v_{sww10},v_{dsww10}) \times b_{10} + \min(v_{sww11},v_{dsww11}) \times b_{11} + \min(v_{sww12},v_{dsww12}) \times b_{12} =$ **31.692** kips

$V_{w\_max} / V_w =$ **0.115**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **490.303** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **0.004** kips/in

Unit shear capacity, widest segment; $v_{sws1} = v_s / 2 =$ **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws1} / k_a =$ **2103.400** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sws1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1$ = **1031.448** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **770.476** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **0.002** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsws2} = \delta_{Cap} \times k_2 / b_2$ = **165.46** plf

Segment 2 shear capacity; $v_{sws2} = v_s / 2$ = **260** plf

v_{dsws2} / v_{sws2} = **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **770.476** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **0.002** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3$ = **165.46** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2$ = **260** plf

v_{dsws3} / v_{sws3} = **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4)$ = **770.476** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4)$ = **0.002** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsws4} = \delta_{Cap} \times k_4 / b_4$ = **165.46** plf

Segment 4 shear capacity; $v_{sws4} = v_s / 2$ = **260** plf

v_{dsws4} / v_{sws4} = **0.636**

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

Segment 5 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_5)$ = **770.476** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5)$ = **0.002** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsws5} = \delta_{Cap} \times k_5 / b_5$ = **165.46** plf

Segment 5 shear capacity; $v_{sws5} = v_s / 2$ = **260** plf

v_{dsws5} / v_{sws5} = **0.636**

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

Segment 6 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_6)$ = **770.476** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6)$ = **0.002** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsws6} = \delta_{Cap} \times k_6 / b_6$ = **165.46** plf

Segment 6 shear capacity; $v_{sws6} = v_s / 2$ = **260** plf

v_{dsws6} / v_{sws6} = **0.636**

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

Segment 7 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_7)$ = **770.476** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7)$ = **0.002** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsws7} = \delta_{Cap} \times k_7 / b_7$ = **165.46** plf

Segment 7 shear capacity; $v_{sws7} = v_s / 2$ = **260** plf

v_{dsws7} / v_{sws7} = **0.636**

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

Segment 8 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_8) =$ **770.476** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8) =$ **0.002** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsws8} = \delta_{Cap} \times k_8 / b_8 =$ **165.46** plf

Segment 8 shear capacity; $v_{sws8} = v_s / 2 =$ **260** plf

$v_{dsws8} / v_{sws8} =$ **0.636**

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

Segment 9 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_9) =$ **770.476** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9) =$ **0.002** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsws9} = \delta_{Cap} \times k_9 / b_9 =$ **165.46** plf

Segment 9 shear capacity; $v_{sws9} = v_s / 2 =$ **260** plf

$v_{dsws9} / v_{sws9} =$ **0.636**

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

Segment 10 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{10}) =$ **770.476** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10}) =$ **0.002** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsws10} = \delta_{Cap} \times k_{10} / b_{10} =$ **165.46** plf

Segment 10 shear capacity; $v_{sws10} = v_s / 2 =$ **260** plf

$v_{dsws10} / v_{sws10} =$ **0.636**

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

Segment 11 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{11}) =$ **770.476** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11}) =$ **0.002** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsws11} = \delta_{Cap} \times k_{11} / b_{11} =$ **165.46** plf

Segment 11 shear capacity; $v_{sws11} = v_s / 2 =$ **260** plf

$v_{dsws11} / v_{sws11} =$ **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12}) =$ **1063.886** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12}) =$ **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsws12} = \delta_{Cap} \times k_{12} / b_{12} =$ **119.83** plf

Segment 12 shear capacity; $v_{sws12} = v_s / 2 =$ **260** plf

$v_{dsws12} / v_{sws12} =$ **0.461**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **1.989** kips

Shear capacity for seismic loading; $V_s = v_{sws1} \times b_1 + \min(v_{sws2},v_{dsws2}) \times b_2 + \min(v_{sws3},v_{dsws3}) \times b_3 + \min(v_{sws4},v_{dsws4}) \times b_4 + \min(v_{sws5},v_{dsws5}) \times b_5 + \min(v_{sws6},v_{dsws6}) \times b_6 + \min(v_{sws7},v_{dsws7}) \times b_7 + \min(v_{sws8},v_{dsws8}) \times b_8 + \min(v_{sws9},v_{dsws9}) \times b_9 + \min(v_{sws10},v_{dsws10}) \times b_{10} + \min(v_{sws11},v_{dsws11}) \times b_{11} + \min(v_{sws12},v_{dsws12}) \times b_{12} =$ **22.575** kips

$V_{s\_max} / V_s =$ **0.088**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio; h / b_1 = **0.49**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **2.397** kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 - P$ = **-2.057** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-274** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.199**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2$ = **0.481** kips

Maximum compressive force in chord; $C = V \times (k_1 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 + P$ = **0.736** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **70** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **584** lb/in^2

f_c / F_c' = **0.120**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.112**

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 = **0.77**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.526** kips

Maximum tensile force in chord; $T = V \times (k_2 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 - P$ = **-1.309** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-175** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2$ = **0.481** kips

Maximum compressive force in chord; $C = V \times (k_2 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 + P$ = **0.643** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **61** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **584** lb/in^2

f_c / F_c' = **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 5 and 6

Shear wall aspect ratio; $h / b_3 =$ **0.77**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.526** kips

Maximum tensile force in chord; $T = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 - P =$ **-1.309** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-175** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 =$ **0.481** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 + P =$ **0.643** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **61** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2

$f_c / F_c' =$ **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 7 and 8

Shear wall aspect ratio; $h / b_4 =$ **0.77**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 =$ **1.526** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 - P =$ **-1.309** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-175** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 =$ **0.481** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 + P =$ **0.643** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **61** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2

$f_c / F_c' =$ **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 9 and 10

Shear wall aspect ratio; $h / b_5 =$ **0.77**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_5 / 2 =$ **1.526** kips

Maximum tensile force in chord; $T = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 - P =$ **-1.309** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-175** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \; + 0.75 \times L_r) \times s / 2 =$ **0.481** kips

Maximum compressive force in chord; $C = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 + P =$ **0.643** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **61** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2

$f_c / F_c' =$ **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 11 and 12

Shear wall aspect ratio; $h / b_6 =$ **0.77**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_6 / 2 =$ **1.526** kips

Maximum tensile force in chord; $T = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 - P =$ **-1.309** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-175** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 =$ **0.481** kips
Maximum compressive force in chord; $C = V \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 + P =$ **0.643** kips
Maximum applied compressive stress; $f_c = C / A_e =$ **61** lb/in^2
Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2
$f_c / F_c' =$ **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2
$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 13 and 14

Shear wall aspect ratio; $h / b_7 =$ **0.77**
Load combination 5
Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips
Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_7 / 2 =$ **1.526** kips
Maximum tensile force in chord; $T = V \times (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 - P =$ **-1.309** kips
Maximum applied tensile stress; $f_t = T / A_{en} =$ **-175** lb/in^2
Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2
$f_t / F_t' =$ **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3
Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips
Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 =$ **0.481** kips
Maximum compressive force in chord; $C = V \times (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 + P =$ **0.643** kips
Maximum applied compressive stress; $f_c = C / A_e =$ **61** lb/in^2
Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2
$f_c / F_c' =$ **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2
$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 15 and 16

Shear wall aspect ratio; $h / b_8 =$ **0.77**
Load combination 5
Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips
Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_8 / 2 =$ **1.526** kips
Maximum tensile force in chord; $T = V \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 - P =$ **-1.309** kips
Maximum applied tensile stress; $f_t = T / A_{en} =$ **-175** lb/in^2
Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2
$f_t / F_t' =$ **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + + 0.75 \times L_r) \times s / 2 =$ **0.481** kips

Maximum compressive force in chord; $C = V \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 + P =$ **0.643** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **61** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2

$f_c / F_c' =$ **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 17 and 18

Shear wall aspect ratio; $h / b_9 =$ **0.77**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_9 / 2 =$ **1.526** kips

Maximum tensile force in chord; $T = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 - P =$ **-1.309** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-175** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.126**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + + 0.75 \times L_r) \times s / 2 =$ **0.481** kips

Maximum compressive force in chord; $C = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 + P =$ **0.643** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **61** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2

$f_c / F_c' =$ **0.105**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.098**

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

Chord capacity for chords 19 and 20

Shear wall aspect ratio; $h / b_{10} =$ **0.77**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **3.65** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{10} / 2 =$ **1.526** kips

Maximum tensile force in chord; $T = V \times (k_{10} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} - P =$ **-1.309** kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-175}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.126}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{2.737}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + + 0.75 \times L_r) \times s / 2 = \mathbf{0.481}$ kips

Maximum compressive force in chord; $C = V \times (k_{10} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} + P = \mathbf{0.643}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{61}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{584}$ lb/in^2

$f_c / F_c' = \mathbf{0.105}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.098}$

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

Chord capacity for chords 21 and 22

Shear wall aspect ratio; $h / b_{11} = \mathbf{0.77}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{3.65}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{11} / 2 = \mathbf{1.526}$ kips

Maximum tensile force in chord; $T = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} - P = \mathbf{-1.309}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-175}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.126}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{2.737}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + + 0.75 \times L_r) \times s / 2 = \mathbf{0.481}$ kips

Maximum compressive force in chord; $C = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} + P = \mathbf{0.643}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{61}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{584}$ lb/in^2

$f_c / F_c' = \mathbf{0.105}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.098}$

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

Chord capacity for chords 23 and 24

Shear wall aspect ratio; $h / b_{12} = \mathbf{1.064}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{3.65}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{12} / 2 =$ **1.105** kips

Maximum tensile force in chord; $T = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} - P =$ **-0.948** kips

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

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.092**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **2.737** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + \ + 0.75 \times L_r) \times s / 2 =$ **0.481** kips

Maximum compressive force in chord; $C = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} + P =$ **0.599** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **57** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **584** lb/in^2

$f_c / F_c' =$ **0.098**

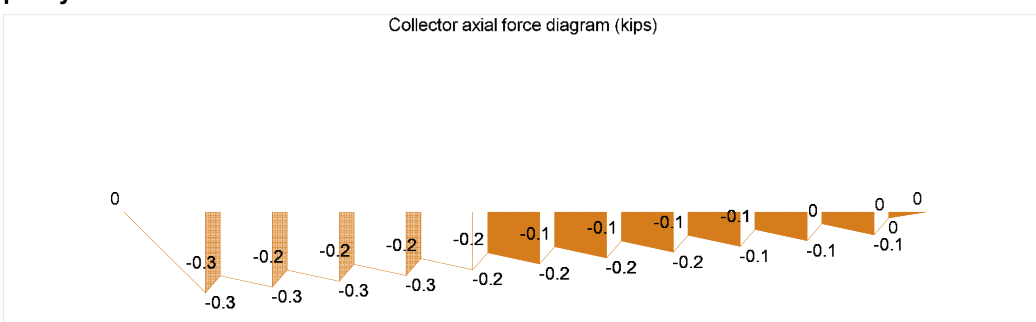
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.091**

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

Collector capacity



Collector seismic design force factor; $F_{Coll} =$ **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max}) =$ **3.65** kips

Maximum force in collector; $P_{coll} =$ **0.322** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) =$ **31** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **0.022**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) =$ **31** lb/in^2

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

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2484** lb/in^2

$f_c / F_c' =$ **0.012**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **6.083** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.27** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1 =$ **70.06** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.039** in

$\delta_{sww1} / \Delta_{w\_allow} =$ **0.144**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2 =$ **44.59** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.025** in

$\delta_{sww2} / \Delta_{w\_allow} =$ **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3 =$ **44.59** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.025** in

$\delta_{sww3} / \Delta_{w\_allow} =$ **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4 =$ **44.59** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{sww4} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_4 =$ **0.025** in

$\delta_{sww4} / \Delta_{w\_allow} =$ **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5 =$ **44.59** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{sww5} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_5 =$ **0.025** in

$\delta_{sww5} / \Delta_{w\_allow} =$ **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6 =$ **44.59** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{sww6} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_6$ = **0.025** in

$\delta_{sww6} / \Delta_{w\_allow}$ = **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7$ = **44.59** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{sww7} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_7$ = **0.025** in

$\delta_{sww7} / \Delta_{w\_allow}$ = **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8$ = **44.59** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{sww8} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_8$ = **0.025** in

$\delta_{sww8} / \Delta_{w\_allow}$ = **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9$ = **44.59** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{sww9} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_9$ = **0.025** in

$\delta_{sww9} / \Delta_{w\_allow}$ = **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{10} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10}$ = **44.59** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{sww10} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{10}$ = **0.025** in

$\delta_{sww10} / \Delta_{w\_allow}$ = **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{11} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11}$ = **44.59** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{sww11} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{11}$ = **0.025** in

$\delta_{sww11} / \Delta_{w\_allow}$ = **0.093**

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{12} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12}$ = **32.29** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{sww12} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{12} =$ **0.018** in

$\delta_{sww12} / \Delta_{w\_allow} =$ **0.069**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q =$ **2.842** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h =$ **1.942** in

Deflection ampification factor; $C_{d\delta} =$ **4**

Seismic importance factor; $I_e =$ **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1 =$ **32.73** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1 =$ **0.018** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e =$ **0.073** in

$\delta_{sws1} / \Delta_{s\_allow} =$ **0.037**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2 =$ **20.83** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2 =$ **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e =$ **0.047** in

$\delta_{sws2} / \Delta_{s\_allow} =$ **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3 =$ **20.83** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3 =$ **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e =$ **0.047** in

$\delta_{sws3} / \Delta_{s\_allow} =$ **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4 =$ **20.83** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{swse4} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_4$ = **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws4} = C_{d\delta} \times \delta_{swse4} / I_e$ = **0.047** in

$\delta_{sws4} / \Delta_{s\_allow}$ = **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_5 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5$ = **20.83** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{swse5} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_5$ = **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws5} = C_{d\delta} \times \delta_{swse5} / I_e$ = **0.047** in

$\delta_{sws5} / \Delta_{s\_allow}$ = **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6$ = **20.83** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{swse6} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_6$ = **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws6} = C_{d\delta} \times \delta_{swse6} / I_e$ = **0.047** in

$\delta_{sws6} / \Delta_{s\_allow}$ = **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7$ = **20.83** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{swse7} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_7$ = **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws7} = C_{d\delta} \times \delta_{swse7} / I_e$ = **0.047** in

$\delta_{sws7} / \Delta_{s\_allow}$ = **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8$ = **20.83** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{swse8} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_8$ = **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws8} = C_{d\delta} \times \delta_{swse8} / I_e$ = **0.047** in

$\delta_{sws8} / \Delta_{s\_allow}$ = **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9$ = **20.83** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{swse9} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_9 =$ **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws9} = C_{d\delta} \times \delta_{swse9} / I_e =$ **0.047** in

$\delta_{sws9} / \Delta_{s\_allow} =$ **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10} =$ **20.83** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{swse10} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{10} =$ **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws10} = C_{d\delta} \times \delta_{swse10} / I_e =$ **0.047** in

$\delta_{sws10} / \Delta_{s\_allow} =$ **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{11} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11} =$ **20.83** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{swse11} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{11} =$ **0.012** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws11} = C_{d\delta} \times \delta_{swse11} / I_e =$ **0.047** in

$\delta_{sws11} / \Delta_{s\_allow} =$ **0.024**

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12} =$ **15.09** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{swse12} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{12} =$ **0.009** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws12} = C_{d\delta} \times \delta_{swse12} / I_e =$ **0.035** in

$\delta_{sws12} / \Delta_{s\_allow} =$ **0.018**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

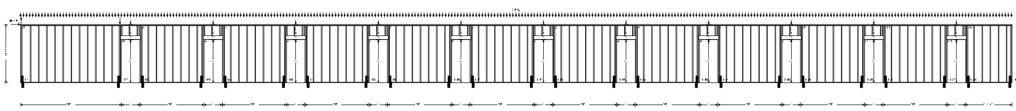
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	31691	5129	0.162	PASS
Chord capacity	lb/in^2	432	180	0.417	PASS
Collector capacity	lb/in^2	1380	43	0.031	PASS
Deflection	in	0.316	0.065	0.205	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **162.104** ft



Panel opening details

Width of opening; w_{o1} = **3** ft

Height of opening; h_{o1} = **7** ft

Height to underside of lintel over opening; l_{o1} = **7** ft

Position of opening; P_{o1} = **16.5** ft

Width of opening; w_{o2} = **3** ft

Height of opening; h_{o2} = **7** ft

Height to underside of lintel over opening; l_{o2} = **7** ft

Position of opening; P_{o2} = **30** ft

Width of opening; w_{o3} = **3** ft

Height of opening; h_{o3} = **7** ft

Height to underside of lintel over opening; l_{o3} = **7** ft

Position of opening; P_{o3} = **43.5** ft

Width of opening; w_{o4} = **3** ft

Height of opening; h_{o4} = **7** ft

Height to underside of lintel over opening; l_{o4} = **7** ft

Position of opening; P_{o4} = **57** ft

Width of opening; w_{o5} = **3** ft

Height of opening; h_{o5} = **7** ft

Height to underside of lintel over opening; l_{o5} = **7** ft

Position of opening; P_{o5} = **70.5** ft

Width of opening; w_{o6} = **3** ft

Height of opening; h_{o6} = **7** ft
Height to underside of lintel over opening; l_{o6} = **7** ft
Position of opening; P_{o6} = **84** ft
Width of opening; w_{o7} = **3** ft
Height of opening; h_{o7} = **7** ft
Height to underside of lintel over opening; l_{o7} = **7** ft
Position of opening; P_{o7} = **97.5** ft
Width of opening; w_{o8} = **3** ft
Height of opening; h_{o8} = **7** ft
Height to underside of lintel over opening; l_{o8} = **7** ft
Position of opening; P_{o8} = **111** ft
Width of opening; w_{o9} = **3** ft
Height of opening; h_{o9} = **7** ft
Height to underside of lintel over opening; l_{o9} = **7** ft
Position of opening; P_{o9} = **124.5** ft
Width of opening; w_{o10} = **3** ft
Height of opening; h_{o10} = **7** ft
Height to underside of lintel over opening; l_{o10} = **7** ft
Position of opening; P_{o10} = **138** ft
Width of opening; w_{o11} = **3** ft
Height of opening; h_{o11} = **7** ft
Height to underside of lintel over opening; l_{o11} = **7** ft
Position of opening; P_{o11} = **151.5** ft
Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3} - w_{o4} \times h_{o4} - w_{o5} \times h_{o5} - w_{o6} \times h_{o6} - w_{o7} \times h_{o7} - w_{o8} \times h_{o8} - w_{o9} \times h_{o9} - w_{o10} \times h_{o10} - w_{o11} \times h_{o11}$ = **1307.369** 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
Vertical anchor stiffness; k_a = **1** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2

Compression parallel to grain; F_c = **1350** lb/in^2

Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **395.31** lb/ft

Floor live load acting on top of panel; L_f = **805.622** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **8548** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **4992** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-567;	-265;	323;	3995;	0;	211;	0;	0;
Ch2;	567;	265;	323;	3995;	0;	211;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;
Ch7;	0;	0;	0;	0;	0;	0;	0;	0;
Ch8;	0;	0;	0;	0;	0;	0;	0;	0;
Ch9;	0;	0;	0;	0;	0;	0;	0;	0;
Ch10;	0;	0;	0;	0;	0;	0;	0;	0;
Ch11;	0;	0;	0;	0;	0;	0;	0;	0;
Ch12;	0;	0;	0;	0;	0;	0;	0;	0;
Ch13;	0;	0;	0;	0;	0;	0;	0;	0;
Ch14;	0;	0;	0;	0;	0;	0;	0;	0;
Ch15;	0;	0;	0;	0;	0;	0;	0;	0;
Ch16;	0;	0;	0;	0;	0;	0;	0;	0;
Ch17;	0;	0;	0;	0;	0;	0;	0;	0;
Ch18;	0;	0;	0;	0;	0;	0;	0;	0;
Ch19;	0;	0;	0;	0;	0;	0;	0;	0;
Ch20;	0;	0;	0;	0;	0;	0;	0;	0;
Ch21;	0;	0;	0;	0;	0;	0;	0;	0;

Ch22;	0;	0;	0;	0;	0;	0;	0;	0;
Ch23;	0;	0;	0;	0;	0;	0;	0;	0;
Ch24;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **16.5** ft

Shear wall aspect ratio; h / b_1 = **0.575**

Segment 2 wall length; b_2 = **10.5** ft

Shear wall aspect ratio; h / b_2 = **0.904**

Segment 3 wall length; b_3 = **10.5** ft

Shear wall aspect ratio; h / b_3 = **0.904**

Segment 4 wall length; b_4 = **10.5** ft

Shear wall aspect ratio; h / b_4 = **0.904**

Segment 5 wall length; b_5 = **10.5** ft

Shear wall aspect ratio; h / b_5 = **0.904**
Segment 6 wall length; b_6 = **10.5** ft
Shear wall aspect ratio; h / b_6 = **0.904**
Segment 7 wall length; b_7 = **10.5** ft
Shear wall aspect ratio; h / b_7 = **0.904**
Segment 8 wall length; b_8 = **10.5** ft
Shear wall aspect ratio; h / b_8 = **0.904**
Segment 9 wall length; b_9 = **10.5** ft
Shear wall aspect ratio; h / b_9 = **0.904**
Segment 10 wall length; b_{10} = **10.5** ft
Shear wall aspect ratio; h / b_{10} = **0.904**
Segment 11 wall length; b_{11} = **10.5** ft
Shear wall aspect ratio; h / b_{11} = **0.904**
Segment 12 wall length; b_{12} = **7.604** ft
Shear wall aspect ratio; h / b_{12} = **1.248**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **575.152** in/kip
Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **0.003** kips/in
Unit shear capacity, widest segment; $v_{sww1} = v_w / 2$ = **365** plf
Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww1} / k_a$ = **3463.850** in
Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sww1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1$ = **1992.479** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **903.810** in/kip
Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **0.001** kips/in
Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2$ = **232.28** plf
Segment 2 shear capacity; $v_{sww2} = v_w / 2$ = **365** plf
v_{dsww2} / v_{sww2} = **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **903.810** in/kip
Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **0.001** kips/in
Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3$ = **232.28** plf
Segment 3 shear capacity; $v_{sww3} = v_w / 2$ = **365** plf
v_{dsww3} / v_{sww3} = **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4)$ = **903.810** in/kip
Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4)$ = **0.001** kips/in
Segment 4 unit shear at δ_{Cap}; $v_{dsww4} = \delta_{Cap} \times k_4 / b_4$ = **232.28** plf
Segment 4 shear capacity; $v_{sww4} = v_w / 2$ = **365** plf

v_{dsww4} / v_{sww4} = **0.636**

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

Segment 5 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_5)$ = **903.810** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5)$ = **0.001** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsww5} = \delta_{Cap} \times k_5 / b_5$ = **232.28** plf

Segment 5 shear capacity; $v_{sww5} = v_w / 2$ = **365** plf

v_{dsww5} / v_{sww5} = **0.636**

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

Segment 6 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_6)$ = **903.810** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6)$ = **0.001** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsww6} = \delta_{Cap} \times k_6 / b_6$ = **232.28** plf

Segment 6 shear capacity; $v_{sww6} = v_w / 2$ = **365** plf

v_{dsww6} / v_{sww6} = **0.636**

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

Segment 7 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_7)$ = **903.810** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7)$ = **0.001** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsww7} = \delta_{Cap} \times k_7 / b_7$ = **232.28** plf

Segment 7 shear capacity; $v_{sww7} = v_w / 2$ = **365** plf

v_{dsww7} / v_{sww7} = **0.636**

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

Segment 8 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_8)$ = **903.810** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8)$ = **0.001** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsww8} = \delta_{Cap} \times k_8 / b_8$ = **232.28** plf

Segment 8 shear capacity; $v_{sww8} = v_w / 2$ = **365** plf

v_{dsww8} / v_{sww8} = **0.636**

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

Segment 9 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_9)$ = **903.810** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9)$ = **0.001** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsww9} = \delta_{Cap} \times k_9 / b_9$ = **232.28** plf

Segment 9 shear capacity; $v_{sww9} = v_w / 2$ = **365** plf

v_{dsww9} / v_{sww9} = **0.636**

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

Segment 10 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{10})$ = **903.810** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10})$ = **0.001** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsww10} = \delta_{Cap} \times k_{10} / b_{10}$ = **232.28** plf

Segment 10 shear capacity; $v_{sww10} = v_w / 2$ = **365** plf

v_{dsww10} / v_{sww10} = **0.636**

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

Segment 11 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{11})$ = **903.810** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11})$ = **0.001** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsww11} = \delta_{Cap} \times k_{11} / b_{11}$ = **232.28** plf

Segment 11 shear capacity; $v_{sww11} = v_w / 2$ = **365** plf

v_{dsww11} / v_{sww11} = **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12})$ = **1247.995** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12})$ = **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsww12} = \delta_{Cap} \times k_{12} / b_{12}$ = **168.22** plf

Segment 12 shear capacity; $v_{sww12} = v_w / 2$ = **365** plf

v_{dsww12} / v_{sww12} = **0.461**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **5.129** kips

Shear capacity for wind loading; $V_w = v_{sww1} \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + \min(v_{sww3},v_{dsww3}) \times b_3 + \min(v_{sww4},v_{dsww4}) \times b_4 + \min(v_{sww5},v_{dsww5}) \times b_5 + \min(v_{sww6},v_{dsww6}) \times b_6 + \min(v_{sww7},v_{dsww7}) \times b_7 + \min(v_{sww8},v_{dsww8}) \times b_8 + \min(v_{sww9},v_{dsww9}) \times b_9 + \min(v_{sww10},v_{dsww10}) \times b_{10} + \min(v_{sww11},v_{dsww11}) \times b_{11} + \min(v_{sww12},v_{dsww12}) \times b_{12}$ = **31.691** kips

V_{w_max} / V_w = **0.162**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **575.152** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **0.003** kips/in

Unit shear capacity, widest segment; $v_{sws1} = v_s / 2$ = **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws1} / k_a$ = **2467.400** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sws1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1$ = **1419.3** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **903.810** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **0.001** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsws2} = \delta_{Cap} \times k_2 / b_2$ = **165.46** plf

Segment 2 shear capacity; $v_{sws2} = v_s / 2$ = **260** plf

v_{dsws2} / v_{sws2} = **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **903.810** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **0.001** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3$ = **165.46** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2$ = **260** plf

v_{dsws3} / v_{sws3} = **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4)$ = **903.810** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4)$ = **0.001** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsws4} = \delta_{Cap} \times k_4 / b_4$ = **165.46** plf

Segment 4 shear capacity; $v_{sws4} = v_s / 2$ = **260** plf

v_{dsws4} / v_{sws4} = **0.636**

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

Segment 5 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_5)$ = **903.810** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5)$ = **0.001** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsws5} = \delta_{Cap} \times k_5 / b_5$ = **165.46** plf

Segment 5 shear capacity; $v_{sws5} = v_s / 2$ = **260** plf

v_{dsws5} / v_{sws5} = **0.636**

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

Segment 6 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_6)$ = **903.810** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6)$ = **0.001** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsws6} = \delta_{Cap} \times k_6 / b_6$ = **165.46** plf

Segment 6 shear capacity; $v_{sws6} = v_s / 2$ = **260** plf

v_{dsws6} / v_{sws6} = **0.636**

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

Segment 7 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_7)$ = **903.810** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7)$ = **0.001** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsws7} = \delta_{Cap} \times k_7 / b_7$ = **165.46** plf

Segment 7 shear capacity; $v_{sws7} = v_s / 2$ = **260** plf

v_{dsws7} / v_{sws7} = **0.636**

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

Segment 8 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_8)$ = **903.810** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8)$ = **0.001** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsws8} = \delta_{Cap} \times k_8 / b_8$ = **165.46** plf

Segment 8 shear capacity; $v_{sws8} = v_s / 2$ = **260** plf

v_{dsws8} / v_{sws8} = **0.636**

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

Segment 9 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_9)$ = **903.810** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9)$ = **0.001** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsws9} = \delta_{Cap} \times k_9 / b_9$ = **165.46** plf

Segment 9 shear capacity; $v_{sws9} = v_s / 2$ = **260** plf

v_{dsws9} / v_{sws9} = **0.636**

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

Segment 10 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{10})$ = **903.810** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10})$ = **0.001** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsws10} = \delta_{Cap} \times k_{10} / b_{10}$ = **165.46** plf

Segment 10 shear capacity; $v_{sws10} = v_s / 2$ = **260** plf

v_{dsws10} / v_{sws10} = **0.636**

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

Segment 11 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{11})$ = **903.810** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11})$ = **0.001** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsws11} = \delta_{Cap} \times k_{11} / b_{11}$ = **165.46** plf

Segment 11 shear capacity; $v_{sws11} = v_s / 2$ = **260** plf

v_{dsws11} / v_{sws11} = **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12})$ = **1247.995** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12})$ = **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsws12} = \delta_{Cap} \times k_{12} / b_{12}$ = **119.83** plf

Segment 12 shear capacity; $v_{sws12} = v_s / 2$ = **260** plf

v_{dsws12} / v_{sws12} = **0.461**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q$ = **3.494** kips

Shear capacity for seismic loading; $V_s = v_{sws1} \times b_1 + \min(v_{sws2},v_{dsws2}) \times b_2 + \min(v_{sws3},v_{dsws3}) \times b_3 + \min(v_{sws4},v_{dsws4}) \times b_4 + \min(v_{sws5},v_{dsws5}) \times b_5 + \min(v_{sws6},v_{dsws6}) \times b_6 + \min(v_{sws7},v_{dsws7}) \times b_7 + \min(v_{sws8},v_{dsws8}) \times b_8 + \min(v_{sws9},v_{dsws9}) \times b_9 + \min(v_{sws10},v_{dsws10}) \times b_{10} + \min(v_{sws11},v_{dsws11}) \times b_{11} + \min(v_{sws12},v_{dsws12}) \times b_{12}$ = **22.575** kips

V_{s_max} / V_s = **0.155**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; h / b_1 = **0.575**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1}$ = **4.531** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 - P$ = **-3.970** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-529** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.384**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + -1 \times 0.45 \times W_{ch1} + D_{C\_ch1} + 0.75 \times L_{r\_ch1}$ = **1.472** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 + P$ = **1.892** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **180** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.417**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.288**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2}$ = **4.531** kips

Maximum tensile force in chord; $T = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 - P$ = **-3.970** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-529** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.384**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + 0.45 \times W_{ch2} + D_{C\_ch2} + 0.75 \times L_{r\_ch2}$ = **1.472** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 + P$ = **1.892** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **180** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.417**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.288**

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

Chord capacity for chord 3

Shear wall aspect ratio; h / b_2 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; V = 0.45 × W = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_2 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_2 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; V = 0.45 × W = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_2 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 5

Shear wall aspect ratio; h / b_3 = **0.904**

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 - P =$ **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 + P =$ **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.153**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 - P =$ **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 + P =$ **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.153**

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

Chord capacity for chord 7

Shear wall aspect ratio; $h / b_4 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 - P =$ **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 + P =$ **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.153**

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

Chord capacity for chord 8

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 - P =$ **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 + P =$ **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.153**

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

Chord capacity for chord 9

Shear wall aspect ratio; h / b_5 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_5 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 10

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_5 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.153}$

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

Chord capacity for chord 11

Shear wall aspect ratio; $h / b_6 = \mathbf{0.904}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{5.129}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_6 / 2 = \mathbf{1.574}$ kips

Maximum tensile force in chord; $T = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 - P = \mathbf{-1.217}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-162}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.118}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{3.847}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 = \mathbf{0.736}$ kips

Maximum compressive force in chord; $C = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 + P = \mathbf{1.004}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{96}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{432}$ lb/in^2

$f_c / F_c' = \mathbf{0.221}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.153}$

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

Chord capacity for chord 12

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{5.129}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_6 / 2 = \mathbf{1.574}$ kips

Maximum tensile force in chord; $T = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 - P = \mathbf{-1.217}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-162}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.118}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{3.847}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 = \mathbf{0.736}$ kips

Maximum compressive force in chord; $C = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 + P = \mathbf{1.004}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{96}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{432}$ lb/in^2

$f_c / F_c' = \mathbf{0.221}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 13

Shear wall aspect ratio; h / b_7 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_7 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 14

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_7 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 15

Shear wall aspect ratio; h / b_8 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_8 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 16

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_8 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 17

Shear wall aspect ratio; h / b_9 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_9 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 + P$ = **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.153**

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

Chord capacity for chord 18

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_9 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 - P$ = **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 + P =$ **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.153**

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

Chord capacity for chord 19

Shear wall aspect ratio; $h / b_{10} =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{10} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} - P =$ **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} + P =$ **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.153**

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

Chord capacity for chord 20

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{10} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} - P =$ **-1.217** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-162** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{3.847}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 = \mathbf{0.736}$ kips

Maximum compressive force in chord; $C = V \times (k_{10} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} + P = \mathbf{1.004}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{96}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{432}$ lb/in^2

$f_c / F_c' = \mathbf{0.221}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.153}$

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

Chord capacity for chord 21

Shear wall aspect ratio; $h / b_{11} = \mathbf{0.904}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{5.129}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{11} / 2 = \mathbf{1.574}$ kips

Maximum tensile force in chord; $T = V \times (k_{11} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} - P = \mathbf{-1.217}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-162}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.118}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{3.847}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 = \mathbf{0.736}$ kips

Maximum compressive force in chord; $C = V \times (k_{11} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} + P = \mathbf{1.004}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{96}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{432}$ lb/in^2

$f_c / F_c' = \mathbf{0.221}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.153}$

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

Chord capacity for chord 22

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{5.129}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{11} / 2 = \mathbf{1.574}$ kips

Maximum tensile force in chord; $T = V \times (k_{11} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} - P = \mathbf{-1.217}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-162}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.118**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{11} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} + P =$ **1.004** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **96** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.221**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.153**

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

Chord capacity for chord 23

Shear wall aspect ratio; $h / b_{12} =$ **1.248**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{12} / 2 =$ **1.14** kips

Maximum tensile force in chord; $T = V \times (k_{12} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} - P =$ **-0.882** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-118** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.085**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{12} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} + P =$ **0.930** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **89** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.205**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.142**

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

Chord capacity for chord 24

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **5.129** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{12} / 2 =$ **1.14** kips

Maximum tensile force in chord; $T = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} - P$ = **-0.882** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-118** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.085**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **3.847** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} + P$ = **0.930** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **89** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.205**

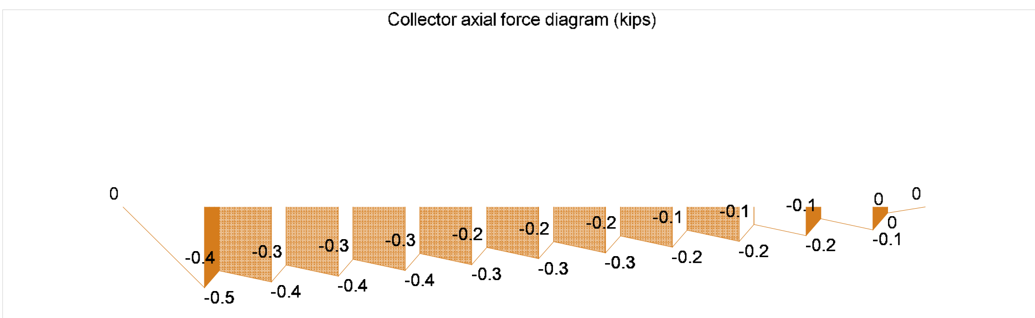
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.142**

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

Collector capacity



Collector seismic design force factor; F_{Coll} = **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **5.129** kips

Maximum force in collector; P_{coll} = **0.453** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s)$ = **43** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.031**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s)$ = **43** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2484** lb/in^2

f_c / F_c' = **0.017**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **8.548** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1$ = **98.45** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1},D_{T\_ch2}) + \max(\text{abs}(W_{ch1}),\text{abs}(W_{ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.065** in

$\delta_{sww1} / \Delta_{w\_allow}$ = **0.205**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2$ = **62.65** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.042** in

$\delta_{sww2} / \Delta_{w\_allow}$ = **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3$ = **62.65** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.042** in

$\delta_{sww3} / \Delta_{w\_allow}$ = **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4$ = **62.65** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{sww4} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_4$ = **0.042** in

$\delta_{sww4} / \Delta_{w\_allow}$ = **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5$ = **62.65** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{sww5} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_5$ = **0.042** in

$\delta_{sww5} / \Delta_{w\_allow}$ = **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6$ = **62.65** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{sww6} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_6 =$ **0.042** in

$\delta_{sww6} / \Delta_{w\_allow} =$ **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7 =$ **62.65** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{sww7} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_7 =$ **0.042** in

$\delta_{sww7} / \Delta_{w\_allow} =$ **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8 =$ **62.65** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{sww8} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_8 =$ **0.042** in

$\delta_{sww8} / \Delta_{w\_allow} =$ **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9 =$ **62.65** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{sww9} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_9 =$ **0.042** in

$\delta_{sww9} / \Delta_{w\_allow} =$ **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{10} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10} =$ **62.65** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{sww10} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{10} =$ **0.042** in

$\delta_{sww10} / \Delta_{w\_allow} =$ **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{11} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11} =$ **62.65** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{sww11} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{11} =$ **0.042** in

$\delta_{sww11} / \Delta_{w\_allow} =$ **0.133**

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{12} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12} =$ **45.37** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta w} \times h$ - 0.6 × (D + $S_{wt} \times h$) × b / 2) = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{sww12} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{12}$ = **0.031** in

$\delta_{sww12} / \Delta_{w\_allow}$ = **0.098**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q$ = **4.992** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **2.278** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1$ = **57.49** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s} \times h$ - (0.6 - 0.2 × S_{DS}) × (D + $S_{wt} \times h$) × b / 2 - (0.6 - 0.2 × S_{DS}) × min(D_{T_ch1},D_{T_ch2}) + max(abs(E_{q_ch1}),abs(E_{q_ch2}))) = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.038** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e$ = **0.151** in

$\delta_{sws1} / \Delta_{s\_allow}$ = **0.066**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2$ = **36.59** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s} \times h$ - (0.6 - 0.2 × S_{DS}) × (D + $S_{wt} \times h$) × b / 2) = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e$ = **0.098** in

$\delta_{sws2} / \Delta_{s\_allow}$ = **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3$ = **36.59** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s} \times h$ - (0.6 - 0.2 × S_{DS}) × (D + $S_{wt} \times h$) × b / 2) = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e$ = **0.098** in

$\delta_{sws3} / \Delta_{s\_allow}$ = **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4$ = **36.59** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s} \times h$ - (0.6 - 0.2 × S_{DS}) × (D + $S_{wt} \times h$) × b / 2) = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{swse4} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_4 =$ **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws4} = C_{d\delta} \times \delta_{swse4} / I_e =$ **0.098** in

$\delta_{sws4} / \Delta_{s\_allow} =$ **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_5 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5 =$ **36.59** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{swse5} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_5 =$ **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws5} = C_{d\delta} \times \delta_{swse5} / I_e =$ **0.098** in

$\delta_{sws5} / \Delta_{s\_allow} =$ **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6 =$ **36.59** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{swse6} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_6 =$ **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws6} = C_{d\delta} \times \delta_{swse6} / I_e =$ **0.098** in

$\delta_{sws6} / \Delta_{s\_allow} =$ **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7 =$ **36.59** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{swse7} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_7 =$ **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws7} = C_{d\delta} \times \delta_{swse7} / I_e =$ **0.098** in

$\delta_{sws7} / \Delta_{s\_allow} =$ **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8 =$ **36.59** lb/ft

Anchor tension force; $T_\delta = max(0 kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a =$ **0.000** in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{swse8} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_8 =$ **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws8} = C_{d\delta} \times \delta_{swse8} / I_e =$ **0.098** in

$\delta_{sws8} / \Delta_{s\_allow} =$ **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9 =$ **36.59** lb/ft

Anchor tension force; $T_{\delta} = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a$ = **0.000** in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{swse9} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_9$ = **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws9} = C_{d\delta} \times \delta_{swse9} / I_e$ = **0.098** in

$\delta_{sws9} / \Delta_{s\_allow}$ = **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{10} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10}$ = **36.59** lb/ft

Anchor tension force; $T_{\delta} = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a$ = **0.000** in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{swse10} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{10}$ = **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws10} = C_{d\delta} \times \delta_{swse10} / I_e$ = **0.098** in

$\delta_{sws10} / \Delta_{s\_allow}$ = **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11}$ = **36.59** lb/ft

Anchor tension force; $T_{\delta} = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a$ = **0.000** in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{swse11} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{11}$ = **0.025** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws11} = C_{d\delta} \times \delta_{swse11} / I_e$ = **0.098** in

$\delta_{sws11} / \Delta_{s\_allow}$ = **0.043**

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{12} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12}$ = **26.5** lb/ft

Anchor tension force; $T_{\delta} = \max(0\text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_{\delta} / k_a$ = **0.000** in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{swse12} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{12}$ = **0.018** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws12} = C_{d\delta} \times \delta_{swse12} / I_e$ = **0.073** in

$\delta_{sws12} / \Delta_{s\_allow}$ = **0.032**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

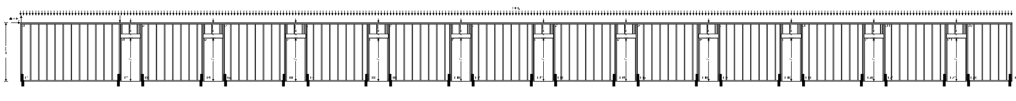
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	31691	6725	0.212	PASS
Chord capacity	lb/in^2	432	303	0.701	PASS
Collector capacity	lb/in^2	1380	57	0.041	PASS
Deflection	in	0.316	0.085	0.268	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **162.104** ft



Panel opening details

Width of opening; w_{o1} = **3** ft

Height of opening; h_{o1} = **7** ft

Height to underside of lintel over opening; l_{o1} = **7** ft

Position of opening; P_{o1} = **16.5** ft

Width of opening; w_{o2} = **3** ft

Height of opening; h_{o2} = **7** ft

Height to underside of lintel over opening; l_{o2} = **7** ft

Position of opening; P_{o2} = **30** ft

Width of opening; w_{o3} = **3** ft

Height of opening; h_{o3} = **7** ft

Height to underside of lintel over opening; l_{o3} = **7** ft

Position of opening; P_{o3} = **43.5** ft

Width of opening; w_{o4} = **3** ft

Height of opening; h_{o4} = **7** ft

Height to underside of lintel over opening; l_{o4} = **7** ft

Position of opening; P_{o4} = **57** ft

Width of opening; w_{o5} = **3** ft

Height of opening; h_{o5} = **7** ft

Height to underside of lintel over opening; l_{o5} = **7** ft

Position of opening; P_{o5} = **70.5** ft

Width of opening; w_{o6} = **3** ft

Height of opening; h_{o6} = **7** ft

Height to underside of lintel over opening; l_{o6} = **7** ft

Position of opening; P_{o6} = **84** ft

Width of opening; w_{o7} = **3** ft
Height of opening; h_{o7} = **7** ft
Height to underside of lintel over opening; l_{o7} = **7** ft
Position of opening; P_{o7} = **97.5** ft
Width of opening; w_{o8} = **3** ft
Height of opening; h_{o8} = **7** ft
Height to underside of lintel over opening; l_{o8} = **7** ft
Position of opening; P_{o8} = **111** ft
Width of opening; w_{o9} = **3** ft
Height of opening; h_{o9} = **7** ft
Height to underside of lintel over opening; l_{o9} = **7** ft
Position of opening; P_{o9} = **124.5** ft
Width of opening; w_{o10} = **3** ft
Height of opening; h_{o10} = **7** ft
Height to underside of lintel over opening; l_{o10} = **7** ft
Position of opening; P_{o10} = **138** ft
Width of opening; w_{o11} = **3** ft
Height of opening; h_{o11} = **7** ft
Height to underside of lintel over opening; l_{o11} = **7** ft
Position of opening; P_{o11} = **151.5** ft
Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3} - w_{o4} \times h_{o4} - w_{o5} \times h_{o5} - w_{o6} \times h_{o6} - w_{o7} \times h_{o7} - w_{o8} \times h_{o8} - w_{o9} \times h_{o9} - w_{o10} \times h_{o10} - w_{o11} \times h_{o11}$ = **1307.369** 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
Vertical anchor stiffness; k_a = **1** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2
Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **395.31** lb/ft

Floor live load acting on top of panel; L_f = **805.622** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **11209** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **6376** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-1501;	-810;	656;	8118;	537;	211;	0;	0;
Ch2;	1501;	810;	656;	8118;	537;	211;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;
Ch7;	0;	0;	0;	0;	0;	0;	0;	0;
Ch8;	0;	0;	0;	0;	0;	0;	0;	0;
Ch9;	0;	0;	0;	0;	0;	0;	0;	0;
Ch10;	0;	0;	0;	0;	0;	0;	0;	0;
Ch11;	0;	0;	0;	0;	0;	0;	0;	0;
Ch12;	0;	0;	0;	0;	0;	0;	0;	0;
Ch13;	0;	0;	0;	0;	0;	0;	0;	0;
Ch14;	0;	0;	0;	0;	0;	0;	0;	0;
Ch15;	0;	0;	0;	0;	0;	0;	0;	0;
Ch16;	0;	0;	0;	0;	0;	0;	0;	0;
Ch17;	0;	0;	0;	0;	0;	0;	0;	0;
Ch18;	0;	0;	0;	0;	0;	0;	0;	0;
Ch19;	0;	0;	0;	0;	0;	0;	0;	0;
Ch20;	0;	0;	0;	0;	0;	0;	0;	0;
Ch21;	0;	0;	0;	0;	0;	0;	0;	0;
Ch22;	0;	0;	0;	0;	0;	0;	0;	0;
Ch23;	0;	0;	0;	0;	0;	0;	0;	0;
Ch24;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **16.5** ft

Shear wall aspect ratio; h / b_1 = **0.575**

Segment 2 wall length; b_2 = **10.5** ft

Shear wall aspect ratio; h / b_2 = **0.904**

Segment 3 wall length; b_3 = **10.5** ft

Shear wall aspect ratio; h / b_3 = **0.904**

Segment 4 wall length; b_4 = **10.5** ft

Shear wall aspect ratio; h / b_4 = **0.904**

Segment 5 wall length; b_5 = **10.5** ft

Shear wall aspect ratio; h / b_5 = **0.904**

Segment 6 wall length; b_6 = **10.5** ft

Shear wall aspect ratio; h / b_6 = **0.904**

Segment 7 wall length; b_7 = **10.5** ft

Shear wall aspect ratio; $h / b_7 =$ **0.904**

Segment 8 wall length; $b_8 =$ **10.5** ft

Shear wall aspect ratio; $h / b_8 =$ **0.904**

Segment 9 wall length; $b_9 =$ **10.5** ft

Shear wall aspect ratio; $h / b_9 =$ **0.904**

Segment 10 wall length; $b_{10} =$ **10.5** ft

Shear wall aspect ratio; $h / b_{10} =$ **0.904**

Segment 11 wall length; $b_{11} =$ **10.5** ft

Shear wall aspect ratio; $h / b_{11} =$ **0.904**

Segment 12 wall length; $b_{12} =$ **7.604** ft

Shear wall aspect ratio; $h / b_{12} =$ **1.248**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **575.152** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **0.003** kips/in

Unit shear capacity, widest segment; $v_{sww1} = v_w / 2 =$ **365** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww1} / k_a =$ **3463.850** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sww1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1 =$ **1992.479** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **903.810** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **0.001** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2 =$ **232.28** plf

Segment 2 shear capacity; $v_{sww2} = v_w / 2 =$ **365** plf

$v_{dsww2} / v_{sww2} =$ **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **903.810** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **0.001** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 =$ **232.28** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2 =$ **365** plf

$v_{dsww3} / v_{sww3} =$ **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) =$ **903.810** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) =$ **0.001** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsww4} = \delta_{Cap} \times k_4 / b_4 =$ **232.28** plf

Segment 4 shear capacity; $v_{sww4} = v_w / 2 =$ **365** plf

$v_{dsww4} / v_{sww4} =$ **0.636**

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

Segment 5 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_5) =$ **903.810** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5) =$ **0.001** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsww5} = \delta_{Cap} \times k_5 / b_5 =$ **232.28** plf

Segment 5 shear capacity; $v_{sww5} = v_w / 2 =$ **365** plf

$v_{dsww5} / v_{sww5} =$ **0.636**

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

Segment 6 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_6) =$ **903.810** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6) =$ **0.001** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsww6} = \delta_{Cap} \times k_6 / b_6 =$ **232.28** plf

Segment 6 shear capacity; $v_{sww6} = v_w / 2 =$ **365** plf

$v_{dsww6} / v_{sww6} =$ **0.636**

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

Segment 7 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_7) =$ **903.810** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7) =$ **0.001** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsww7} = \delta_{Cap} \times k_7 / b_7 =$ **232.28** plf

Segment 7 shear capacity; $v_{sww7} = v_w / 2 =$ **365** plf

$v_{dsww7} / v_{sww7} =$ **0.636**

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

Segment 8 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_8) =$ **903.810** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8) =$ **0.001** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsww8} = \delta_{Cap} \times k_8 / b_8 =$ **232.28** plf

Segment 8 shear capacity; $v_{sww8} = v_w / 2 =$ **365** plf

$v_{dsww8} / v_{sww8} =$ **0.636**

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

Segment 9 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_9) =$ **903.810** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9) =$ **0.001** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsww9} = \delta_{Cap} \times k_9 / b_9 =$ **232.28** plf

Segment 9 shear capacity; $v_{sww9} = v_w / 2 =$ **365** plf

$v_{dsww9} / v_{sww9} =$ **0.636**

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

Segment 10 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{10}) =$ **903.810** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10}) =$ **0.001** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsww10} = \delta_{Cap} \times k_{10} / b_{10} =$ **232.28** plf

Segment 10 shear capacity; $v_{sww10} = v_w / 2 =$ **365** plf

$v_{dsww10} / v_{sww10} =$ **0.636**

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

Segment 11 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{11}) =$ **903.810** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11})$ = **0.001** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsww11} = \delta_{Cap} \times k_{11} / b_{11}$ = **232.28** plf

Segment 11 shear capacity; $v_{sww11} = v_w / 2$ = **365** plf

v_{dsww11} / v_{sww11} = **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12})$ = **1247.995** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12})$ = **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsww12} = \delta_{Cap} \times k_{12} / b_{12}$ = **168.22** plf

Segment 12 shear capacity; $v_{sww12} = v_w / 2$ = **365** plf

v_{dsww12} / v_{sww12} = **0.461**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **6.725** kips

Shear capacity for wind loading; $V_w = v_{sww1} \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + \min(v_{sww3},v_{dsww3}) \times b_3 + \min(v_{sww4},v_{dsww4}) \times b_4 + \min(v_{sww5},v_{dsww5}) \times b_5 + \min(v_{sww6},v_{dsww6}) \times b_6 + \min(v_{sww7},v_{dsww7}) \times b_7 + \min(v_{sww8},v_{dsww8}) \times b_8 + \min(v_{sww9},v_{dsww9}) \times b_9 + \min(v_{sww10},v_{dsww10}) \times b_{10} + \min(v_{sww11},v_{dsww11}) \times b_{11} + \min(v_{sww12},v_{dsww12}) \times b_{12}$ = **31.691** kips

V_{w_max} / V_w = **0.212**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **575.152** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **0.003** kips/in

Unit shear capacity, widest segment; $v_{sws1} = v_s / 2$ = **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws1} / k_a$ = **2467.400** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sws1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1$ = **1419.3** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **903.810** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **0.001** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsws2} = \delta_{Cap} \times k_2 / b_2$ = **165.46** plf

Segment 2 shear capacity; $v_{sws2} = v_s / 2$ = **260** plf

v_{dsws2} / v_{sws2} = **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **903.810** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **0.001** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3$ = **165.46** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2$ = **260** plf

v_{dsws3} / v_{sws3} = **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) =$ **903.810** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) =$ **0.001** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsws4} = \delta_{Cap} \times k_4 / b_4 =$ **165.46** plf

Segment 4 shear capacity; $v_{sws4} = v_s / 2 =$ **260** plf

$v_{dsws4} / v_{sws4} =$ **0.636**

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

Segment 5 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_5) =$ **903.810** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5) =$ **0.001** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsws5} = \delta_{Cap} \times k_5 / b_5 =$ **165.46** plf

Segment 5 shear capacity; $v_{sws5} = v_s / 2 =$ **260** plf

$v_{dsws5} / v_{sws5} =$ **0.636**

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

Segment 6 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_6) =$ **903.810** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6) =$ **0.001** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsws6} = \delta_{Cap} \times k_6 / b_6 =$ **165.46** plf

Segment 6 shear capacity; $v_{sws6} = v_s / 2 =$ **260** plf

$v_{dsws6} / v_{sws6} =$ **0.636**

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

Segment 7 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_7) =$ **903.810** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7) =$ **0.001** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsws7} = \delta_{Cap} \times k_7 / b_7 =$ **165.46** plf

Segment 7 shear capacity; $v_{sws7} = v_s / 2 =$ **260** plf

$v_{dsws7} / v_{sws7} =$ **0.636**

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

Segment 8 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_8) =$ **903.810** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8) =$ **0.001** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsws8} = \delta_{Cap} \times k_8 / b_8 =$ **165.46** plf

Segment 8 shear capacity; $v_{sws8} = v_s / 2 =$ **260** plf

$v_{dsws8} / v_{sws8} =$ **0.636**

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

Segment 9 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_9) =$ **903.810** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9) =$ **0.001** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsws9} = \delta_{Cap} \times k_9 / b_9 =$ **165.46** plf

Segment 9 shear capacity; $v_{sws9} = v_s / 2 =$ **260** plf

$v_{dsws9} / v_{sws9} =$ **0.636**

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

Segment 10 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{10}) =$ **903.810** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10}) =$ **0.001** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsws10} = \delta_{Cap} \times k_{10} / b_{10} =$ **165.46** plf

Segment 10 shear capacity; $v_{sws10} = v_s / 2 =$ **260** plf

$v_{dsws10} / v_{sws10} =$ **0.636**

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

Segment 11 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{11}) =$ **903.810** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11}) =$ **0.001** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsws11} = \delta_{Cap} \times k_{11} / b_{11} =$ **165.46** plf

Segment 11 shear capacity; $v_{sws11} = v_s / 2 =$ **260** plf

$v_{dsws11} / v_{sws11} =$ **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12}) =$ **1247.995** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12}) =$ **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsws12} = \delta_{Cap} \times k_{12} / b_{12} =$ **119.83** plf

Segment 12 shear capacity; $v_{sws12} = v_s / 2 =$ **260** plf

$v_{dsws12} / v_{sws12} =$ **0.461**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **4.463** kips

Shear capacity for seismic loading; $V_s = v_{sws1} \times b_1 + \min(v_{sws2},v_{dsws2}) \times b_2 + \min(v_{sws3},v_{dsws3}) \times b_3 + \min(v_{sws4},v_{dsws4}) \times b_4 + \min(v_{sws5},v_{dsws5}) \times b_5 + \min(v_{sws6},v_{dsws6}) \times b_6 + \min(v_{sws7},v_{dsws7}) \times b_7 + \min(v_{sws8},v_{dsws8}) \times b_8 + \min(v_{sws9},v_{dsws9}) \times b_9 + \min(v_{sws10},v_{dsws10}) \times b_{10} + \min(v_{sws11},v_{dsws11}) \times b_{11} + \min(v_{sws12},v_{dsws12}) \times b_{12} =$ **22.575** kips

$V_{s\_max} / V_s =$ **0.198**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; $h / b_1 =$ **0.575**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1} =$ **6.444** kips

Maximum tensile force in chord; $T = V \times (k_1 / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 - P =$ **-5.709** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-761** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.552**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + -1 \times 0.45 \times W_{ch1} + D_{C\_ch1} + 0.75 \times L_{f\_ch1} + 0.75 \times L_{r\_ch1} =$ **2.628** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 + P =$ **3.180** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **303** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.701**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.485**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **6.444** kips

Maximum tensile force in chord; $T = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 - P =$ **-5.709** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-761** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.552**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + 0.45 \times W_{ch2} + D_{C\_ch2} + 0.75 \times L_{f\_ch2} + 0.75 \times L_{r\_ch2} =$ **2.628** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 + P =$ **3.180** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **303** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.701**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.485**

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

Chord capacity for chord 3

Shear wall aspect ratio; $h / b_2 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 5

Shear wall aspect ratio; $h / b_3 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 + P$ = **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.166**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 - P$ = **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 + P$ = **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.166**

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

Chord capacity for chord 7

Shear wall aspect ratio; h / b_4 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 8

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 9

Shear wall aspect ratio; $h / b_5 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_5 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 10

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_5 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 11

Shear wall aspect ratio; h / b_6 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_6 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 - P$ = **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 + P$ = **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.166**

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

Chord capacity for chord 12

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_6 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 - P$ = **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 + P$ = **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.166**

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

Chord capacity for chord 13

Shear wall aspect ratio; h / b_7 = **0.904**

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **6.725** kips

Axial force for maximum tension; P = (0.6 × (D + S_{wt} × h)) × b_7 / 2 = **1.574** kips

Maximum tensile force in chord; T = V × (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) × h / b_7 - P = **-1.106** kips

Maximum applied tensile stress; f_t = T / A_{en} = **-148** lb/in^2

Design tensile stress; F_t' = F_t × C_D × C_{Mt} × C_{tt} × C_{Ft} × C_i = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; V = 0.45 × W = **5.044** kips

Axial force for maximum compression; P = ((D + S_{wt} × h) + 0.75 × L_f) × s / 2 = **0.736** kips

Maximum compressive force in chord; C = V × (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) × h / b_7 + P = **1.087** kips

Maximum applied compressive stress; f_c = C / A_e = **104** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **432** lb/in^2

f_c / F_c' = **0.240**

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 = **625** lb/in^2

f_c / F_{c_perp}' = **0.166**

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

Chord capacity for chord 14

Load combination 5

Shear force for maximum tension; V = 0.6 × W = **6.725** kips

Axial force for maximum tension; P = (0.6 × (D + S_{wt} × h)) × b_7 / 2 = **1.574** kips

Maximum tensile force in chord; T = V × (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) × h / b_7 - P = **-1.106** kips

Maximum applied tensile stress; f_t = T / A_{en} = **-148** lb/in^2

Design tensile stress; F_t' = F_t × C_D × C_{Mt} × C_{tt} × C_{Ft} × C_i = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; V = 0.45 × W = **5.044** kips

Axial force for maximum compression; P = ((D + S_{wt} × h) + 0.75 × L_f) × s / 2 = **0.736** kips

Maximum compressive force in chord; C = V × (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) × h / b_7 + P = **1.087** kips

Maximum applied compressive stress; f_c = C / A_e = **104** lb/in^2

Design compressive stress; F_c' = F_c × C_D × C_{Mc} × C_{tc} × C_{Fc} × C_i × C_P = **432** lb/in^2

f_c / F_c' = **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 15

Shear wall aspect ratio; $h / b_8 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_8 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 16

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_8 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.166**

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

Chord capacity for chord 17

Shear wall aspect ratio; h / b_9 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_9 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 - P$ = **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 + P$ = **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.166**

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

Chord capacity for chord 18

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_9 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 - P$ = **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 19

Shear wall aspect ratio; $h / b_{10} =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{10} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 20

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{10} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{10} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 21

Shear wall aspect ratio; $h / b_{11} =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{11} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 22

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{11} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} - P =$ **-1.106** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-148** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.107**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{11} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} + P =$ **1.087** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **104** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.240**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.166**

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

Chord capacity for chord 23

Shear wall aspect ratio; $h / b_{12} =$ **1.248**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{12} / 2 =$ **1.14** kips

Maximum tensile force in chord; $T = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} - P =$ **-0.801** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-107** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.077**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} + P =$ **0.990** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **94** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.218**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.151**

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

Chord capacity for chord 24

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **6.725** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{12} / 2 =$ **1.14** kips

Maximum tensile force in chord; $T = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} - P =$ **-0.801** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-107** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.077**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **5.044** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{12} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} + P =$ **0.990** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **94** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.218**

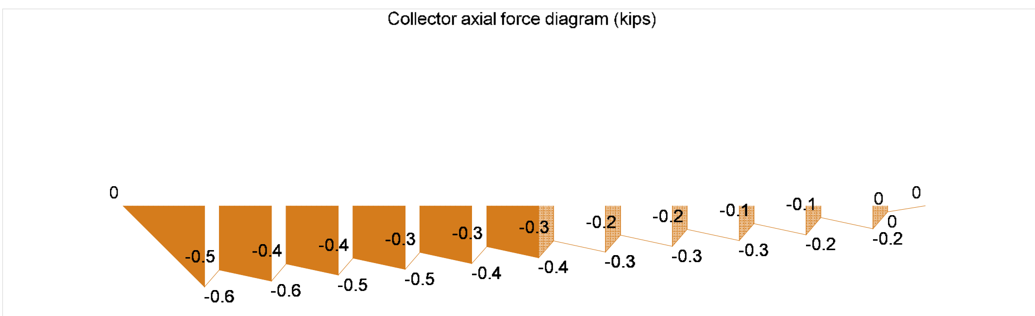
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.151**

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

Collector capacity



Collector seismic design force factor; $F_{Coll} =$ **1**

Maximum shear force on wall; $V_{max} = max(F_{Coll} \times V_{s\_max}, V_{w\_max}) =$ **6.725** kips

Maximum force in collector; $P_{coll} =$ **0.594** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s) =$ **57** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **0.041**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s) =$ **57** lb/in^2

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

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **2484** lb/in^2

$f_c / F_c' =$ **0.023**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W =$ **11.209** kips

Deflection limit; $\Delta_{w\_allow} = h / 360 =$ **0.316** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1$ = **129.1** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1},D_{T\_ch2}) + \max(\text{abs}(W_{ch1}),\text{abs}(W_{ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.085** in

$\delta_{sww1} / \Delta_{w\_allow}$ = **0.268**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.055** in

$\delta_{sww2} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.055** in

$\delta_{sww3} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{sww4} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_4$ = **0.055** in

$\delta_{sww4} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{sww5} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_5$ = **0.055** in

$\delta_{sww5} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{sww6} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_6$ = **0.055** in

$\delta_{sww6} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{sww7} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_7$ = **0.055** in

$\delta_{sww7} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{sww8} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_8$ = **0.055** in

$\delta_{sww8} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{sww9} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_9$ = **0.055** in

$\delta_{sww9} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10}$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{sww10} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{10}$ = **0.055** in

$\delta_{sww10} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{11} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11}$ = **82.16** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{sww11} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{11}$ = **0.055** in

$\delta_{sww11} / \Delta_{w\_allow}$ = **0.174**

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12}$ = **59.5** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{sww12} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{12}$ = **0.041** in

$\delta_{sww12} / \Delta_{w\_allow}$ = **0.129**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s} = E_q$ = **6.376** kips

Deflection limit; $\Delta_{s\_allow} = 0.020 \times h$ = **2.278** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Segment 1

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1$ = **73.43** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2 - (0.6 - 0.2 \times S_{DS}) \times \min(D_{T\_ch1},D_{T\_ch2}) + \max(\text{abs}(E_{q\_ch1}),\text{abs}(E_{q\_ch2})))$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{swse1} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_1$ = **0.048** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws1} = C_{d\delta} \times \delta_{swse1} / I_e$ = **0.193** in

$\delta_{sws1} / \Delta_{s\_allow}$ = **0.085**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_2 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2$ = **46.73** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{swse2} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_2$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws2} = C_{d\delta} \times \delta_{swse2} / I_e$ = **0.126** in

$\delta_{sws2} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3$ = **46.73** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{swse3} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_3$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws3} = C_{d\delta} \times \delta_{swse3} / I_e$ = **0.126** in

$\delta_{sws3} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4$ = **46.73** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{swse4} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_4$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws4} = C_{d\delta} \times \delta_{swse4} / I_e$ = **0.126** in

$\delta_{sws4} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_5 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5$ = **46.73** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{swse5} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_5$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws5} = C_{d\delta} \times \delta_{swse5} / I_e$ = **0.126** in

$\delta_{sws5} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6$ = **46.73** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{swse6} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_6$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws6} = C_{d\delta} \times \delta_{swse6} / I_e$ = **0.126** in

$\delta_{sws6} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_7 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7$ = **46.73** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{swse7} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_7$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws7} = C_{d\delta} \times \delta_{swse7} / I_e$ = **0.126** in

$\delta_{sws7} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_8 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8$ = **46.73** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{swse8} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_8$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws8} = C_{d\delta} \times \delta_{swse8} / I_e$ = **0.126** in

$\delta_{sws8} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9$ = **46.73** lb/ft

Anchor tension force; $T_\delta = max(0\ kips, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{swse9} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_9$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws9} = C_{d\delta} \times \delta_{swse9} / I_e$ = **0.126** in

$\delta_{sws9} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{10} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10}$ = **46.73** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{swse10} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{10}$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws10} = C_{d\delta} \times \delta_{swse10} / I_e$ = **0.126** in

$\delta_{sws10} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11}$ = **46.73** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{swse11} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{11}$ = **0.031** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws11} = C_{d\delta} \times \delta_{swse11} / I_e$ = **0.126** in

$\delta_{sws11} / \Delta_{s\_allow}$ = **0.055**

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{12} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12}$ = **33.85** lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{swse12} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{12}$ = **0.023** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws12} = C_{d\delta} \times \delta_{swse12} / I_e$ = **0.093** in

$\delta_{sws12} / \Delta_{s\_allow}$ = **0.041**

PASS - Shear wall deflection is less than deflection limit

WOOD SHEAR WALL DESIGN (NDS)

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

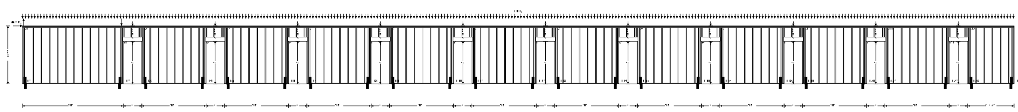
Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	31691	8323	0.263	PASS
Chord capacity	lb/in^2	432	292	0.676	PASS
Collector capacity	lb/in^2	1380	70	0.051	PASS
Deflection	in	0.316	0.104	0.328	PASS

Panel details

Structural wood panel sheathing on one side

Panel height; h = **9.49** ft

Panel length; b = **162.104** ft



Panel opening details

Width of opening; w_{o1} = **3** ft

Height of opening; h_{o1} = **7** ft

Height to underside of lintel over opening; l_{o1} = **7** ft

Position of opening; P_{o1} = **16.5** ft

Width of opening; w_{o2} = **3** ft

Height of opening; h_{o2} = **7** ft

Height to underside of lintel over opening; l_{o2} = **7** ft

Position of opening; P_{o2} = **30** ft

Width of opening; w_{o3} = **3** ft

Height of opening; h_{o3} = **7** ft

Height to underside of lintel over opening; l_{o3} = **7** ft

Position of opening; P_{o3} = **43.5** ft

Width of opening; w_{o4} = **3** ft

Height of opening; h_{o4} = **7** ft

Height to underside of lintel over opening; l_{o4} = **7** ft

Position of opening; P_{o4} = **57** ft

Width of opening; w_{o5} = **3** ft

Height of opening; h_{o5} = **7** ft

Height to underside of lintel over opening; l_{o5} = **7** ft

Position of opening; P_{o5} = **70.5** ft

Width of opening; w_{o6} = **3** ft

Height of opening; h_{o6} = **7** ft

Height to underside of lintel over opening; l_{o6} = **7** ft

Position of opening; P_{o6} = **84** ft
Width of opening; w_{o7} = **3** ft
Height of opening; h_{o7} = **7** ft
Height to underside of lintel over opening; l_{o7} = **7** ft
Position of opening; P_{o7} = **97.5** ft
Width of opening; w_{o8} = **3** ft
Height of opening; h_{o8} = **7** ft
Height to underside of lintel over opening; l_{o8} = **7** ft
Position of opening; P_{o8} = **111** ft
Width of opening; w_{o9} = **3** ft
Height of opening; h_{o9} = **7** ft
Height to underside of lintel over opening; l_{o9} = **7** ft
Position of opening; P_{o9} = **124.5** ft
Width of opening; w_{o10} = **3** ft
Height of opening; h_{o10} = **7** ft
Height to underside of lintel over opening; l_{o10} = **7** ft
Position of opening; P_{o10} = **138** ft
Width of opening; w_{o11} = **3** ft
Height of opening; h_{o11} = **7** ft
Height to underside of lintel over opening; l_{o11} = **7** ft
Position of opening; P_{o11} = **151.5** ft
Total area of wall; $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3} - w_{o4} \times h_{o4} - w_{o5} \times h_{o5} - w_{o6} \times h_{o6} - w_{o7} \times h_{o7} - w_{o8} \times h_{o8} - w_{o9} \times h_{o9} - w_{o10} \times h_{o10} - w_{o11} \times h_{o11}$ = **1307.369** 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; 3 x 2" x 4"
Dressed end post size; 3 x 1.5" x 3.5"
Cross-sectional area of end posts; A_e = **15.75** in^2
Hole diameter; Dia = **1** in
Net cross-sectional area of end posts; A_{en} = **11.25** 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
Vertical anchor stiffness; k_a = **1** lb/in

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

Species, grade and size classification; Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity; G = **0.50**
Tension parallel to grain; F_t = **575** lb/in^2
Compression parallel to grain; F_c = **1350** lb/in^2
Compression perpendicular to grain; F_{c_perp} = **625** lb/in^2

Modulus of elasticity; E = **1600000** lb/in^2

Minimum modulus of elasticity; E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material; 7/16" wood panel oriented strandboard sheathing

Fastener type; 8d common nails at 6"centers

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

Nominal unit shear capacity for seismic design; v_s = **520** lb/ft

Nominal unit shear capacity for wind design; v_w = **730** lb/ft

Apparent shear wall shear stiffness; G_a = **15** kips/in

Loading details

Dead load acting on top of panel; D = **395.31** lb/ft

Floor live load acting on top of panel; L_f = **805.622** lb/ft

Self weight of panel; S_{wt} = **11** lb/ft^2

In plane wind load acting at head of panel; W = **13871** lbs

Wind load serviceability factor; f_{Wserv} = **1.00**

In plane seismic load acting at head of panel; E_q = **6905** lbs

Design spectral response accel. par., short periods; S_{DS} = **0.106**

Chord forces from shear walls above

Chord	$W_{ch[i]}$ (lbs)	$E_{q\_ch[i]}$ (lbs)	$D_{C\_ch[i]}$ (lbs)	$D_{T\_ch[i]}$ (lbs)	$L_{f\_ch[i]}$ (lbs)	$L_{r\_ch[i]}$ (lbs)	$S_{ch[i]}$ (lbs)	$R_{ch[i]}$ (lbs)
Ch1;	-2726;	-1507;	989;	12241;	1074;	211;	0;	0;
Ch2;	2726;	1507;	989;	12241;	1074;	211;	0;	0;
Ch3;	0;	0;	0;	0;	0;	0;	0;	0;
Ch4;	0;	0;	0;	0;	0;	0;	0;	0;
Ch5;	0;	0;	0;	0;	0;	0;	0;	0;
Ch6;	0;	0;	0;	0;	0;	0;	0;	0;
Ch7;	0;	0;	0;	0;	0;	0;	0;	0;
Ch8;	0;	0;	0;	0;	0;	0;	0;	0;
Ch9;	0;	0;	0;	0;	0;	0;	0;	0;
Ch10;	0;	0;	0;	0;	0;	0;	0;	0;
Ch11;	0;	0;	0;	0;	0;	0;	0;	0;
Ch12;	0;	0;	0;	0;	0;	0;	0;	0;
Ch13;	0;	0;	0;	0;	0;	0;	0;	0;
Ch14;	0;	0;	0;	0;	0;	0;	0;	0;
Ch15;	0;	0;	0;	0;	0;	0;	0;	0;
Ch16;	0;	0;	0;	0;	0;	0;	0;	0;
Ch17;	0;	0;	0;	0;	0;	0;	0;	0;
Ch18;	0;	0;	0;	0;	0;	0;	0;	0;
Ch19;	0;	0;	0;	0;	0;	0;	0;	0;
Ch20;	0;	0;	0;	0;	0;	0;	0;	0;
Ch21;	0;	0;	0;	0;	0;	0;	0;	0;
Ch22;	0;	0;	0;	0;	0;	0;	0;	0;
Ch23;	0;	0;	0;	0;	0;	0;	0;	0;

Ch24;	0;	0;	0;	0;	0;	0;	0;	0;

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; $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value; $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **450** psi

Reference compression design value; $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi

For sawn lumber; c = **0.8**

Column stability factor – eqn.3.7-1; $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.17**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio; 3.5

Segment 1 wall length; b_1 = **16.5** ft

Shear wall aspect ratio; h / b_1 = **0.575**

Segment 2 wall length; b_2 = **10.5** ft

Shear wall aspect ratio; h / b_2 = **0.904**

Segment 3 wall length; b_3 = **10.5** ft

Shear wall aspect ratio; h / b_3 = **0.904**

Segment 4 wall length; b_4 = **10.5** ft

Shear wall aspect ratio; h / b_4 = **0.904**

Segment 5 wall length; b_5 = **10.5** ft

Shear wall aspect ratio; h / b_5 = **0.904**

Segment 6 wall length; b_6 = **10.5** ft

Shear wall aspect ratio; $h / b_6 =$ **0.904**

Segment 7 wall length; $b_7 =$ **10.5** ft

Shear wall aspect ratio; $h / b_7 =$ **0.904**

Segment 8 wall length; $b_8 =$ **10.5** ft

Shear wall aspect ratio; $h / b_8 =$ **0.904**

Segment 9 wall length; $b_9 =$ **10.5** ft

Shear wall aspect ratio; $h / b_9 =$ **0.904**

Segment 10 wall length; $b_{10} =$ **10.5** ft

Shear wall aspect ratio; $h / b_{10} =$ **0.904**

Segment 11 wall length; $b_{11} =$ **10.5** ft

Shear wall aspect ratio; $h / b_{11} =$ **0.904**

Segment 12 wall length; $b_{12} =$ **7.604** ft

Shear wall aspect ratio; $h / b_{12} =$ **1.248**

Segmented shear wall capacity - Equal deflection method

Wind loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1) =$ **575.152** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1) =$ **0.003** kips/in

Unit shear capacity, widest segment; $v_{sww1} = v_w / 2 =$ **365** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sww1} / k_a =$ **3463.850** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sww1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sww1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1 =$ **1992.476** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2) =$ **903.810** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2) =$ **0.001** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsww2} = \delta_{Cap} \times k_2 / b_2 =$ **232.28** plf

Segment 2 shear capacity; $v_{sww2} = v_w / 2 =$ **365** plf

$v_{dsww2} / v_{sww2} =$ **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3) =$ **903.810** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3) =$ **0.001** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsww3} = \delta_{Cap} \times k_3 / b_3 =$ **232.28** plf

Segment 3 shear capacity; $v_{sww3} = v_w / 2 =$ **365** plf

$v_{dsww3} / v_{sww3} =$ **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) =$ **903.810** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) =$ **0.001** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsww4} = \delta_{Cap} \times k_4 / b_4 =$ **232.28** plf

Segment 4 shear capacity; $v_{sww4} = v_w / 2 =$ **365** plf

$v_{dsww4} / v_{sww4} =$ **0.636**

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

Segment 5 vertical unit deflection; Δ_{a1} = h / ($k_a \times b_5$) = **903.810** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5)$ = **0.001** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsww5} = \delta_{Cap} \times k_5 / b_5$ = **232.28** plf

Segment 5 shear capacity; $v_{sww5} = v_w / 2$ = **365** plf

v_{dsww5} / v_{sww5} = **0.636**

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

Segment 6 vertical unit deflection; Δ_{a1} = h / ($k_a \times b_6$) = **903.810** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6)$ = **0.001** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsww6} = \delta_{Cap} \times k_6 / b_6$ = **232.28** plf

Segment 6 shear capacity; $v_{sww6} = v_w / 2$ = **365** plf

v_{dsww6} / v_{sww6} = **0.636**

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

Segment 7 vertical unit deflection; Δ_{a1} = h / ($k_a \times b_7$) = **903.810** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7)$ = **0.001** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsww7} = \delta_{Cap} \times k_7 / b_7$ = **232.28** plf

Segment 7 shear capacity; $v_{sww7} = v_w / 2$ = **365** plf

v_{dsww7} / v_{sww7} = **0.636**

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

Segment 8 vertical unit deflection; Δ_{a1} = h / ($k_a \times b_8$) = **903.810** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8)$ = **0.001** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsww8} = \delta_{Cap} \times k_8 / b_8$ = **232.28** plf

Segment 8 shear capacity; $v_{sww8} = v_w / 2$ = **365** plf

v_{dsww8} / v_{sww8} = **0.636**

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

Segment 9 vertical unit deflection; Δ_{a1} = h / ($k_a \times b_9$) = **903.810** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9)$ = **0.001** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsww9} = \delta_{Cap} \times k_9 / b_9$ = **232.28** plf

Segment 9 shear capacity; $v_{sww9} = v_w / 2$ = **365** plf

v_{dsww9} / v_{sww9} = **0.636**

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

Segment 10 vertical unit deflection; Δ_{a1} = h / ($k_a \times b_{10}$) = **903.810** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10})$ = **0.001** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsww10} = \delta_{Cap} \times k_{10} / b_{10}$ = **232.28** plf

Segment 10 shear capacity; $v_{sww10} = v_w / 2$ = **365** plf

v_{dsww10} / v_{sww10} = **0.636**

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

Segment 11 vertical unit deflection; Δ_{a1} = h / ($k_a \times b_{11}$) = **903.810** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11})$ = **0.001** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsww11} = \delta_{Cap} \times k_{11} / b_{11}$ = **232.28** plf

Segment 11 shear capacity; $v_{sww11} = v_w / 2$ = **365** plf

v_{dsww11} / v_{sww11} = **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12})$ = **1247.995** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12})$ = **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsww12} = \delta_{Cap} \times k_{12} / b_{12}$ = **168.22** plf

Segment 12 shear capacity; $v_{sww12} = v_w / 2$ = **365** plf

v_{dsww12} / v_{sww12} = **0.461**

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

Maximum shear force under wind loading; $V_{w\_max} = 0.6 \times W$ = **8.323** kips

Shear capacity for wind loading; $V_w = v_{sww1} \times b_1 + \min(v_{sww2},v_{dsww2}) \times b_2 + \min(v_{sww3},v_{dsww3}) \times b_3 + \min(v_{sww4},v_{dsww4}) \times b_4 + \min(v_{sww5},v_{dsww5}) \times b_5 + \min(v_{sww6},v_{dsww6}) \times b_6 + \min(v_{sww7},v_{dsww7}) \times b_7 + \min(v_{sww8},v_{dsww8}) \times b_8 + \min(v_{sww9},v_{dsww9}) \times b_9 + \min(v_{sww10},v_{dsww10}) \times b_{10} + \min(v_{sww11},v_{dsww11}) \times b_{11} + \min(v_{sww12},v_{dsww12}) \times b_{12}$ = **31.691** kips

V_{w_max} / V_w = **0.263**

PASS - Shear capacity for wind load exceeds maximum shear force

Seismic loading:

Segment 1 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_1)$ = **575.152** in/kip

Segment 1 stiffness; $k_1 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_1^2) + h / (G_a \times b_1) + h \times \Delta_{a1} / b_1)$ = **0.003** kips/in

Unit shear capacity, widest segment; $v_{sws1} = v_s / 2$ = **260** plf

Vertical deflction under capacity load; $\Delta_{a\_Cap} = h \times v_{sws1} / k_a$ = **2467.400** in

Deflection under capacity load; $\delta_{Cap} = 2 \times v_{sws1} \times h^3 / (3 \times E \times A_e \times b_1) + v_{sws1} \times h / (G_a) + h \times \Delta_{a\_Cap} / b_1$ = **1419.298** in

Segment 2 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_2)$ = **903.810** in/kip

Segment 2 stiffness; $k_2 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_2^2) + h / (G_a \times b_2) + h \times \Delta_{a1} / b_2)$ = **0.001** kips/in

Segment 2 unit shear at δ_{Cap}; $v_{dsws2} = \delta_{Cap} \times k_2 / b_2$ = **165.46** plf

Segment 2 shear capacity; $v_{sws2} = v_s / 2$ = **260** plf

v_{dsws2} / v_{sws2} = **0.636**

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

Segment 3 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_3)$ = **903.810** in/kip

Segment 3 stiffness; $k_3 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_3^2) + h / (G_a \times b_3) + h \times \Delta_{a1} / b_3)$ = **0.001** kips/in

Segment 3 unit shear at δ_{Cap}; $v_{dsws3} = \delta_{Cap} \times k_3 / b_3$ = **165.46** plf

Segment 3 shear capacity; $v_{sws3} = v_s / 2$ = **260** plf

v_{dsws3} / v_{sws3} = **0.636**

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

Segment 4 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_4) =$ **903.810** in/kip

Segment 4 stiffness; $k_4 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_4^2) + h / (G_a \times b_4) + h \times \Delta_{a1} / b_4) =$ **0.001** kips/in

Segment 4 unit shear at δ_{Cap}; $v_{dsws4} = \delta_{Cap} \times k_4 / b_4 =$ **165.46** plf

Segment 4 shear capacity; $v_{sws4} = v_s / 2 =$ **260** plf

$v_{dsws4} / v_{sws4} =$ **0.636**

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

Segment 5 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_5) =$ **903.810** in/kip

Segment 5 stiffness; $k_5 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_5^2) + h / (G_a \times b_5) + h \times \Delta_{a1} / b_5) =$ **0.001** kips/in

Segment 5 unit shear at δ_{Cap}; $v_{dsws5} = \delta_{Cap} \times k_5 / b_5 =$ **165.46** plf

Segment 5 shear capacity; $v_{sws5} = v_s / 2 =$ **260** plf

$v_{dsws5} / v_{sws5} =$ **0.636**

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

Segment 6 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_6) =$ **903.810** in/kip

Segment 6 stiffness; $k_6 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_6^2) + h / (G_a \times b_6) + h \times \Delta_{a1} / b_6) =$ **0.001** kips/in

Segment 6 unit shear at δ_{Cap}; $v_{dsws6} = \delta_{Cap} \times k_6 / b_6 =$ **165.46** plf

Segment 6 shear capacity; $v_{sws6} = v_s / 2 =$ **260** plf

$v_{dsws6} / v_{sws6} =$ **0.636**

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

Segment 7 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_7) =$ **903.810** in/kip

Segment 7 stiffness; $k_7 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_7^2) + h / (G_a \times b_7) + h \times \Delta_{a1} / b_7) =$ **0.001** kips/in

Segment 7 unit shear at δ_{Cap}; $v_{dsws7} = \delta_{Cap} \times k_7 / b_7 =$ **165.46** plf

Segment 7 shear capacity; $v_{sws7} = v_s / 2 =$ **260** plf

$v_{dsws7} / v_{sws7} =$ **0.636**

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

Segment 8 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_8) =$ **903.810** in/kip

Segment 8 stiffness; $k_8 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_8^2) + h / (G_a \times b_8) + h \times \Delta_{a1} / b_8) =$ **0.001** kips/in

Segment 8 unit shear at δ_{Cap}; $v_{dsws8} = \delta_{Cap} \times k_8 / b_8 =$ **165.46** plf

Segment 8 shear capacity; $v_{sws8} = v_s / 2 =$ **260** plf

$v_{dsws8} / v_{sws8} =$ **0.636**

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

Segment 9 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_9) =$ **903.810** in/kip

Segment 9 stiffness; $k_9 = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_9^2) + h / (G_a \times b_9) + h \times \Delta_{a1} / b_9) =$ **0.001** kips/in

Segment 9 unit shear at δ_{Cap}; $v_{dsws9} = \delta_{Cap} \times k_9 / b_9 =$ **165.46** plf

Segment 9 shear capacity; $v_{sws9} = v_s / 2 =$ **260** plf

$v_{dsws9} / v_{sws9} =$ **0.636**

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

Segment 10 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{10}) =$ **903.810** in/kip

Segment 10 stiffness; $k_{10} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{10}^2) + h / (G_a \times b_{10}) + h \times \Delta_{a1} / b_{10}) =$ **0.001** kips/in

Segment 10 unit shear at δ_{Cap}; $v_{dsws10} = \delta_{Cap} \times k_{10} / b_{10} =$ **165.46** plf

Segment 10 shear capacity; $v_{sws10} = v_s / 2 =$ **260** plf

$v_{dsws10} / v_{sws10} =$ **0.636**

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

Segment 11 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{11}) =$ **903.810** in/kip

Segment 11 stiffness; $k_{11} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{11}^2) + h / (G_a \times b_{11}) + h \times \Delta_{a1} / b_{11}) =$ **0.001** kips/in

Segment 11 unit shear at δ_{Cap}; $v_{dsws11} = \delta_{Cap} \times k_{11} / b_{11} =$ **165.46** plf

Segment 11 shear capacity; $v_{sws11} = v_s / 2 =$ **260** plf

$v_{dsws11} / v_{sws11} =$ **0.636**

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

Segment 12 vertical unit deflection; $\Delta_{a1} = h / (k_a \times b_{12}) =$ **1247.995** in/kip

Segment 12 stiffness; $k_{12} = 1 / (2 \times h^3 / (3 \times E \times A_e \times b_{12}^2) + h / (G_a \times b_{12}) + h \times \Delta_{a1} / b_{12}) =$ **0.001** kips/in

Segment 12 unit shear at δ_{Cap}; $v_{dsws12} = \delta_{Cap} \times k_{12} / b_{12} =$ **119.83** plf

Segment 12 shear capacity; $v_{sws12} = v_s / 2 =$ **260** plf

$v_{dsws12} / v_{sws12} =$ **0.461**

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

Maximum shear force under seismic loading; $V_{s\_max} = 0.7 \times E_q =$ **4.834** kips

Shear capacity for seismic loading; $V_s = v_{sws1} \times b_1 + \min(v_{sws2},v_{dsws2}) \times b_2 + \min(v_{sws3},v_{dsws3}) \times b_3 + \min(v_{sws4},v_{dsws4}) \times b_4 + \min(v_{sws5},v_{dsws5}) \times b_5 + \min(v_{sws6},v_{dsws6}) \times b_6 + \min(v_{sws7},v_{dsws7}) \times b_7 + \min(v_{sws8},v_{dsws8}) \times b_8 + \min(v_{sws9},v_{dsws9}) \times b_9 + \min(v_{sws10},v_{dsws10}) \times b_{10} + \min(v_{sws11},v_{dsws11}) \times b_{11} + \min(v_{sws12},v_{dsws12}) \times b_{12}$ = **22.575** kips

$V_{s\_max} / V_s =$ **0.214**

PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chord 1

Shear wall aspect ratio; $h / b_1 =$ **0.575**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + 0.6 \times W_{ch1} + 0.6 \times D_{T\_ch1} =$ **8.182** kips

Maximum tensile force in chord; $T = V \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 - P =$ **-7.272** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-646** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.468**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + -1 \times 0.45 \times W_{ch1} + D_{C\_ch1} + 0.75 \times L_{f\_ch1} + 0.75 \times L_{r\_ch1} =$ **3.916** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 + P =$ **4.598** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **292** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.676**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.467**

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

Chord capacity for chord 2

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 + -1 \times 0.6 \times W_{ch2} + 0.6 \times D_{T\_ch2} =$ **8.182** kips

Maximum tensile force in chord; $T = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 - P =$ **-7.272** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-646** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.468**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 + 0.45 \times W_{ch2} + D_{C\_ch2} + 0.75 \times L_{f\_ch2} + 0.75 \times L_{r\_ch2} =$ **3.916** kips

Maximum compressive force in chord; $C = V \times (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_1 + P =$ **4.598** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **292** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.676**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.467**

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

Chord capacity for chord 3

Shear wall aspect ratio; $h / b_2 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 4

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_2 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_2 + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 5

Shear wall aspect ratio; $h / b_3 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_3 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 + P$ = **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 6

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_3 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 - P$ = **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_3 + P$ = **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 7

Shear wall aspect ratio; h / b_4 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in²

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in²

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in²

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in²

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in²

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 8

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_4 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in²

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in²

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_4 + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in²

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in²

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in²

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 9

Shear wall aspect ratio; $h / b_5 =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_5 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 10

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_5 / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_5 + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 11

Shear wall aspect ratio; $h / b_6 = \mathbf{0.904}$

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{8.323}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_6 / 2 = \mathbf{1.574}$ kips

Maximum tensile force in chord; $T = V \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 - P = \mathbf{-0.995}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-88}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.064}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{6.242}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 = \mathbf{0.736}$ kips

Maximum compressive force in chord; $C = V \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 + P = \mathbf{1.170}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{74}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{432}$ lb/in^2

$f_c / F_c' = \mathbf{0.172}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

$f_c / F_{c\_perp}' = \mathbf{0.119}$

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

Chord capacity for chord 12

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W = \mathbf{8.323}$ kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_6 / 2 = \mathbf{1.574}$ kips

Maximum tensile force in chord; $T = V \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 - P = \mathbf{-0.995}$ kips

Maximum applied tensile stress; $f_t = T / A_{en} = \mathbf{-88}$ lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = \mathbf{1380}$ lb/in^2

$f_t / F_t' = \mathbf{-0.064}$

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W = \mathbf{6.242}$ kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 = \mathbf{0.736}$ kips

Maximum compressive force in chord; $C = V \times (k_6 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_6 + P = \mathbf{1.170}$ kips

Maximum applied compressive stress; $f_c = C / A_e = \mathbf{74}$ lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = \mathbf{432}$ lb/in^2

$f_c / F_c' = \mathbf{0.172}$

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b = \mathbf{625}$ lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 13

Shear wall aspect ratio; h / b_7 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_7 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 - P$ = **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 + P$ = **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 14

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_7 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 - P$ = **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_7 + P$ = **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 15

Shear wall aspect ratio; h / b_8 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_8 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 - P$ = **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 + P$ = **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 16

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_8 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 - P$ = **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_8 + P$ = **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 17

Shear wall aspect ratio; h / b_9 = **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_9 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 - P$ = **-0.995**
kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 + P$ = **1.170**
kips

Maximum applied compressive stress; $f_c = C / A_e$ = **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.119**

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

Chord capacity for chord 18

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W$ = **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_9 / 2$ = **1.574** kips

Maximum tensile force in chord; $T = V \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 - P$ = **-0.995**
kips

Maximum applied tensile stress; $f_t = T / A_{en}$ = **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_9 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_9 + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 19

Shear wall aspect ratio; $h / b_{10} =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{10} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{10} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{10} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 20

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{10} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{10} / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{10} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{10} + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 21

Shear wall aspect ratio; $h / b_{11} =$ **0.904**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{11} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 22

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{11} / 2 =$ **1.574** kips

Maximum tensile force in chord; $T = V \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} - P =$ **-0.995** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-88** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.064**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{11} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{11} + P =$ **1.170** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **74** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.172**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.119**

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

Chord capacity for chord 23

Shear wall aspect ratio; $h / b_{12} =$ **1.248**

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{12} / 2 =$ **1.14** kips

Maximum tensile force in chord; $T = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} - P =$ **-0.721** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-64** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1380** lb/in^2

$f_t / F_t' =$ **-0.046**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W =$ **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} + P =$ **1.050** kips

Maximum applied compressive stress; $f_c = C / A_e =$ **67** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **432** lb/in^2

$f_c / F_c' =$ **0.154**

PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **625** lb/in^2

$f_c / F_{c\_perp}' =$ **0.107**

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

Chord capacity for chord 24

Load combination 5

Shear force for maximum tension; $V = 0.6 \times W =$ **8.323** kips

Axial force for maximum tension; $P = (0.6 \times (D + S_{wt} \times h)) \times b_{12} / 2 =$ **1.14** kips

Maximum tensile force in chord; $T = V \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} - P =$ **-0.721** kips

Maximum applied tensile stress; $f_t = T / A_{en} =$ **-64** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **-0.046**

PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 3

Shear force for maximum compression; $V = 0.45 \times W$ = **6.242** kips

Axial force for maximum compression; $P = ((D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2$ = **0.736** kips

Maximum compressive force in chord; $C = V \times (k_{12} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) \times h / b_{12} + P$ = **1.050** kips

Maximum applied compressive stress; $f_c = C / A_e$ = **67** lb/in^2

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **432** lb/in^2

f_c / F_c' = **0.154**

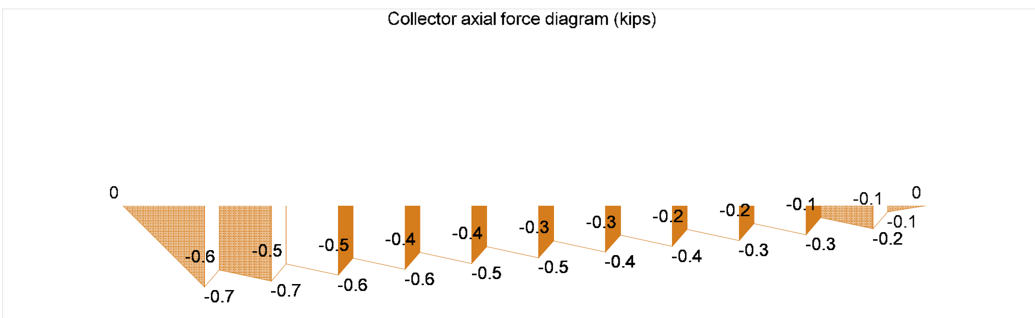
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate; $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.107**

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

Collector capacity



Collector seismic design force factor; F_{Coll} = **1**

Maximum shear force on wall; $V_{max} = \max(F_{Coll} \times V_{s\_max}, V_{w\_max})$ = **8.323** kips

Maximum force in collector; P_{coll} = **0.734** kips

Maximum applied tensile stress; $f_t = P_{coll} / (2 \times A_s)$ = **70** lb/in^2

Design tensile stress; $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.051**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress; $f_c = P_{coll} / (2 \times A_s)$ = **70** lb/in^2

Column stability factor; C_P = **1.00**

Design compressive stress; $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2484** lb/in^2

f_c / F_c' = **0.028**

PASS - Design compressive stress exceeds maximum applied compressive stress

Wind load deflection

Design shear force; $V_{\delta w} = f_{Wserv} \times W$ = **13.871** kips

Deflection limit; $\Delta_{w\_allow} = h / 360$ = **0.316** in

Segment 1

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_1 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1 = \mathbf{159.76}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2 - 0.6 \times \min(D_{T\_ch1},D_{T\_ch2}) + \max(\text{abs}(W_{ch1}),\text{abs}(W_{ch2}))) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 1 deflection – Eqn. 4.3-1; $\delta_{sww1} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_1) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_1 = \mathbf{0.104}$ in

$\delta_{sww1} / \Delta_{w\_allow} = \mathbf{0.328}$

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_2 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 2 deflection – Eqn. 4.3-1; $\delta_{sww2} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_2) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_2 = \mathbf{0.067}$ in

$\delta_{sww2} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_3 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 3 deflection – Eqn. 4.3-1; $\delta_{sww3} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_3) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_3 = \mathbf{0.067}$ in

$\delta_{sww3} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_4 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 4 deflection – Eqn. 4.3-1; $\delta_{sww4} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_4) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_4 = \mathbf{0.067}$ in

$\delta_{sww4} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{sww5} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_5 = \mathbf{0.067}$ in

$\delta_{sww5} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{sww6} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_6 = \mathbf{0.067}$ in

$\delta_{sww6} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{sww7} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_7 = \mathbf{0.067}$ in

$\delta_{sww7} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{sww8} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_8 = \mathbf{0.067}$ in

$\delta_{sww8} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9 = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{sww9} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_9 = \mathbf{0.067}$ in

$\delta_{sww9} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{10} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10} = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{sww10} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{10} = \mathbf{0.067}$ in

$\delta_{sww10} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{11} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11} = \mathbf{101.67}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{sww11} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{11} = \mathbf{0.067}$ in

$\delta_{sww11} / \Delta_{w\_allow} = \mathbf{0.212}$

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta w} = V_{\delta w} \times (k_{12} / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12} = \mathbf{73.63}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{sww12} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b_{12} = \mathbf{0.049}$ in

δ_{sww12} / Δ_{w_allow} = **0.156**

PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force; $V_{\delta s}$ = E_q = **6.905** kips

Deflection limit; Δ_{s_allow}= 0.020 × h = **2.278** in

Deflection ampification factor; $C_{d\delta}$ = **4**

Seismic importance factor; I_e = **1**

Segment 1

Induced unit shear; $v_{\delta s}$ = $V_{\delta s}$ × (k_1 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_1 = **79.53** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s}$ × h - (0.6 - 0.2 × S_{DS}) × (D + S_{wt} × h) × b / 2 - (0.6 - 0.2 × S_{DS}) × min(D_{T_ch1},D_{T_ch2}) + max(abs(E_{q_ch1}),abs(E_{q_ch2}))) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** in

Segment 1 deflection – Eqn. 4.3-1; δ_{swse1} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b_1) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b_1 = **0.052** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; δ_{sws1} = $C_{d\delta}$ × δ_{swse1} / I_e = **0.206** in

δ_{sws1} / Δ_{s_allow} = **0.091**

PASS - Shear wall deflection is less than deflection limit

Segment 2

Induced unit shear; $v_{\delta s}$ = $V_{\delta s}$ × (k_2 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_2 = **50.61** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s}$ × h - (0.6 - 0.2 × S_{DS}) × (D + S_{wt} × h) × b / 2) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** in

Segment 2 deflection – Eqn. 4.3-1; δ_{swse2} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b_2) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b_2 = **0.033** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; δ_{sws2} = $C_{d\delta}$ × δ_{swse2} / I_e = **0.133** in

δ_{sws2} / Δ_{s_allow} = **0.059**

PASS - Shear wall deflection is less than deflection limit

Segment 3

Induced unit shear; $v_{\delta s}$ = $V_{\delta s}$ × (k_3 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_3 = **50.61** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s}$ × h - (0.6 - 0.2 × S_{DS}) × (D + S_{wt} × h) × b / 2) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** in

Segment 3 deflection – Eqn. 4.3-1; δ_{swse3} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b_3) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b_3 = **0.033** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; δ_{sws3} = $C_{d\delta}$ × δ_{swse3} / I_e = **0.133** in

δ_{sws3} / Δ_{s_allow} = **0.059**

PASS - Shear wall deflection is less than deflection limit

Segment 4

Induced unit shear; $v_{\delta s}$ = $V_{\delta s}$ × (k_4 / sum(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_4 = **50.61** lb/ft

Anchor tension force; T_δ = max(0 kips,$v_{\delta s}$ × h - (0.6 - 0.2 × S_{DS}) × (D + S_{wt} × h) × b / 2) = **0.000** kips

Vertical elongation at anchor; Δ_a = T_δ / k_a = **0.000** in

Segment 4 deflection – Eqn. 4.3-1; δ_{swse4} = 2 × $v_{\delta s}$ × h^3 / (3 × E × A_e × b_4) + $v_{\delta s}$ × h / (G_a) + h × Δ_a / b_4 = **0.033** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; δ_{sws4} = $C_{d\delta}$ × δ_{swse4} / I_e = **0.133** in

δ_{sws4} / Δ_{s_allow} = **0.059**

PASS - Shear wall deflection is less than deflection limit

Segment 5

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_5 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_5 = \mathbf{50.61}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 5 deflection – Eqn. 4.3-1; $\delta_{swse5} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_5) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_5 = \mathbf{0.033}$ in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws5} = C_{d\delta} \times \delta_{swse5} / I_e = \mathbf{0.133}$ in

$\delta_{sws5} / \Delta_{s\_allow} = \mathbf{0.059}$

PASS - Shear wall deflection is less than deflection limit

Segment 6

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_6 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_6 = \mathbf{50.61}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 6 deflection – Eqn. 4.3-1; $\delta_{swse6} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_6) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_6 = \mathbf{0.033}$ in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws6} = C_{d\delta} \times \delta_{swse6} / I_e = \mathbf{0.133}$ in

$\delta_{sws6} / \Delta_{s\_allow} = \mathbf{0.059}$

PASS - Shear wall deflection is less than deflection limit

Segment 7

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_7 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_7 = \mathbf{50.61}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 7 deflection – Eqn. 4.3-1; $\delta_{swse7} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_7) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_7 = \mathbf{0.033}$ in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws7} = C_{d\delta} \times \delta_{swse7} / I_e = \mathbf{0.133}$ in

$\delta_{sws7} / \Delta_{s\_allow} = \mathbf{0.059}$

PASS - Shear wall deflection is less than deflection limit

Segment 8

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_8 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_8 = \mathbf{50.61}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 8 deflection – Eqn. 4.3-1; $\delta_{swse8} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_8) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_8 = \mathbf{0.033}$ in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws8} = C_{d\delta} \times \delta_{swse8} / I_e = \mathbf{0.133}$ in

$\delta_{sws8} / \Delta_{s\_allow} = \mathbf{0.059}$

PASS - Shear wall deflection is less than deflection limit

Segment 9

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_9 / \text{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_9 = \mathbf{50.61}$ lb/ft

Anchor tension force; $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) = \mathbf{0.000}$ kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a = \mathbf{0.000}$ in

Segment 9 deflection – Eqn. 4.3-1; $\delta_{swse9} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_9) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_9 = \mathbf{0.033}$ in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws9} = C_{d\delta} \times \delta_{swse9} / I_e$ = **0.133** in

$\delta_{sws9} / \Delta_{s\_allow}$ = **0.059**

PASS - Shear wall deflection is less than deflection limit

Segment 10

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{10} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{10}$ = **50.61** lb/ft

Anchor tension force; $T_\delta = \max(0\ \mathrm{kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 10 deflection – Eqn. 4.3-1; $\delta_{swse10} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{10}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{10}$ = **0.033** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws10} = C_{d\delta} \times \delta_{swse10} / I_e$ = **0.133** in

$\delta_{sws10} / \Delta_{s\_allow}$ = **0.059**

PASS - Shear wall deflection is less than deflection limit

Segment 11

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{11} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{11}$ = **50.61** lb/ft

Anchor tension force; $T_\delta = \max(0\ \mathrm{kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 11 deflection – Eqn. 4.3-1; $\delta_{swse11} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{11}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{11}$ = **0.033** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws11} = C_{d\delta} \times \delta_{swse11} / I_e$ = **0.133** in

$\delta_{sws11} / \Delta_{s\_allow}$ = **0.059**

PASS - Shear wall deflection is less than deflection limit

Segment 12

Induced unit shear; $v_{\delta s} = V_{\delta s} \times (k_{12} / \mathrm{sum}(k_{10},k_{11},k_{12},k_1,k_2,k_3,k_4,k_5,k_6,k_7,k_8,k_9)) / b_{12}$ = **36.65** lb/ft

Anchor tension force; $T_\delta = \max(0\ \mathrm{kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **0.000** kips

Vertical elongation at anchor; $\Delta_a = T_\delta / k_a$ = **0.000** in

Segment 12 deflection – Eqn. 4.3-1; $\delta_{swse12} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b_{12}) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b_{12}$ = **0.024** in

Amp. seis. deflection – ASCE7 Eqn. 12.8-15; $\delta_{sws12} = C_{d\delta} \times \delta_{swse12} / I_e$ = **0.098** in

$\delta_{sws12} / \Delta_{s\_allow}$ = **0.043**

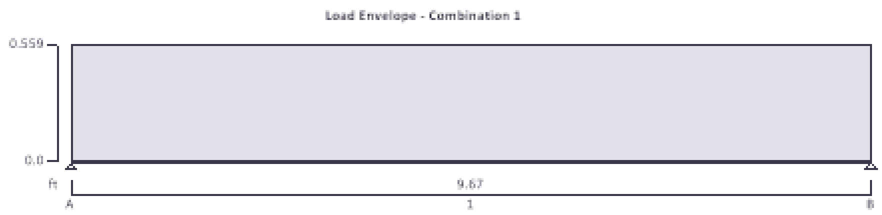
PASS - Shear wall deflection is less than deflection limit

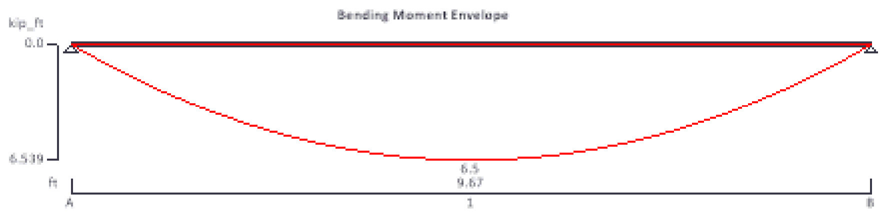
STAIRS

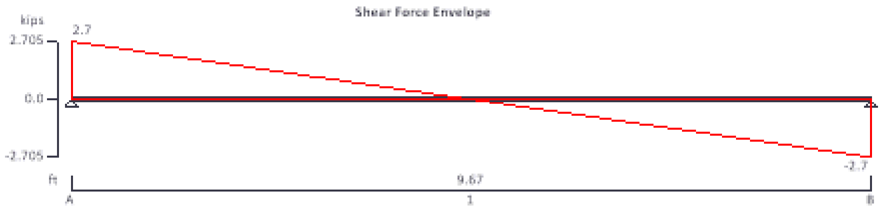
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

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

Tedds calculation version 1.7.09







Applied loading

Beam loads

	Dead self weight of beam × 1	
	Dead full UDL 67 lb/ft	
	Live full UDL 476 lb/ft	

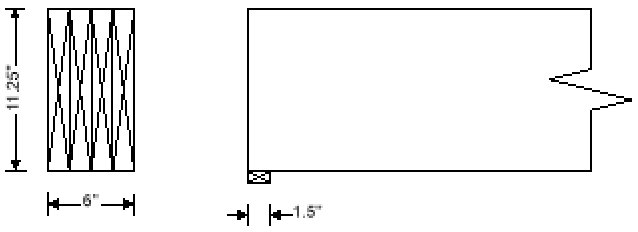
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
	Support B	Dead × 1.00
		Live × 1.00

Analysis results

Maximum moment	M_{max} = **6539** lb_ft	M_{min} = **0** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **6539** lb_ft	
Maximum shear	F_{max} = **2705** lb	F_{min} = **-2705** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **2705** lb	
Total load on member	W_{tot} = **5409** lb	
Reaction at support A	R_{A_max} = **2705** lb	R_{A_min} = **2705** lb

Unfactored dead load reaction at support A	R_{A_Dead} = **403** lb
Unfactored live load reaction at support A	R_{A_Live} = **2301** lb
Reaction at support B	R_{B_max} = **2705** lb R_{B_min} = **2705** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **403** lb
Unfactored live load reaction at support B	R_{B_Live} = **2301** lb



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **12** in
Dressed depth of sections	d = **11.25** in
Number of sections in member	N = **4**
Overall breadth of member	$b_b = N \times b$ = **6** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Length of span	L_{s1} = **9.67** ft
Length of bearing	L_b = **1.5** in
Load duration	**Ten years**

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **67.50** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **126.56** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **67.50** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **711.91** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **202.50** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.00**
Size factor for tension - Table 4A	C_{Ft} = **1.00**
Size factor for compression - Table 4A	C_{Fc} = **1.00**
Flat use factor - Table 4A	C_{fu} = **1.20**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.15**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **1.50**

- Beam is fully restrained

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

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_{ic\_perp} \times C_b$ = **625** lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **301** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.481**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1035** lb/in^2

Actual bending stress $f_b = M / S_x$ = **620** lb/in^2

f_b / F_b' = **0.599**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i$ = **180** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A)$ = **60** lb/in^2

f_v / F_v' = **0.334**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1600000** lb/in^2

Design deflection $\delta_{adm} = 0.003 \times L_{s1}$ = **0.348** in

Total deflection δ_{b_s1} = **0.097** in

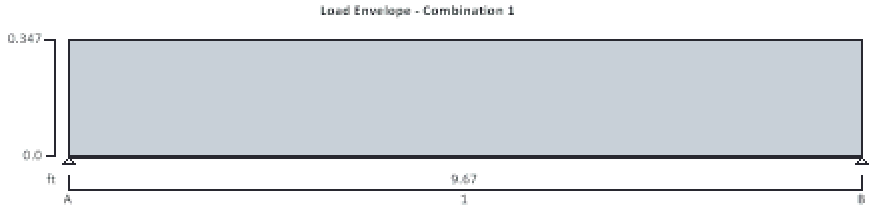
$\delta_{b\_s1} / \delta_{adm}$ = **0.278**

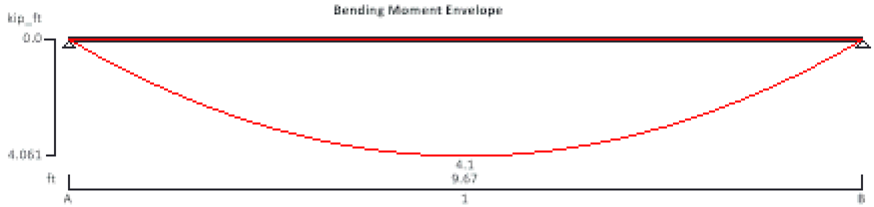
PASS - Total deflection is less than design deflection

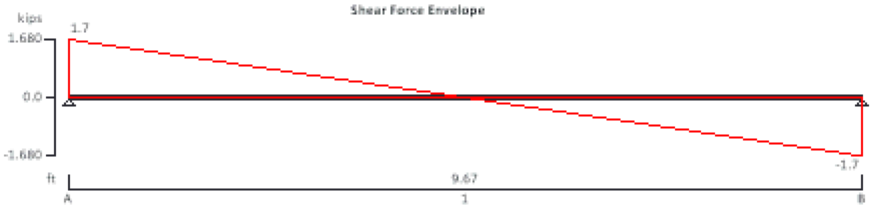
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

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

Tedds calculation version 1.7.10







Applied loading

Beam loads

Dead self weight of beam × 1
Dead full UDL 44 lb/ft
Live full UDL 287 lb/ft

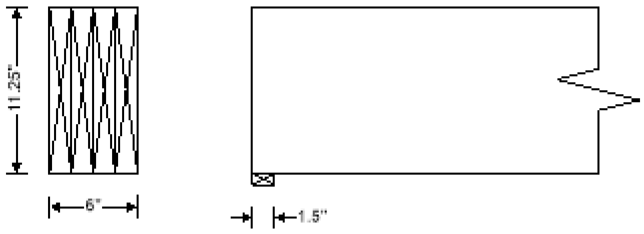
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
	Support B	Dead × 1.00
		Live × 1.00

Analysis results

Maximum moment	M_{max} = **4061** lb_ft	M_{min} = **0** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **4061** lb_ft	
Maximum shear	F_{max} = **1680** lb	F_{min} = **-1680** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1680** lb	
Total load on member	W_{tot} = **3359** lb	
Reaction at support A	R_{A_max} = **1680** lb	R_{A_min} = **1680** lb

Unfactored dead load reaction at support A R_{A_Dead} = **292** lb
Unfactored live load reaction at support A R_{A_Live} = **1388** lb
Reaction at support B R_{B_max} = **1680** lb R_{B_min} = **1680** lb
Unfactored dead load reaction at support B R_{B_Dead} = **292** lb
Unfactored live load reaction at support B R_{B_Live} = **1388** lb



Sawn lumber section details

Nominal breadth of sections b_{nom} = **2** in
Dressed breadth of sections b = **1.5** in
Nominal depth of sections d_{nom} = **12** in
Dressed depth of sections d = **11.25** in
Number of sections in member N = **4**
Overall breadth of member $b_b = N \times b$ = **6** in
Species, grade and size classification Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain F_b = **900** lb/in^2
Tension parallel to grain F_t = **575** lb/in^2
Compression parallel to grain F_c = **1350** lb/in^2
Compression perpendicular to grain F_{c_perp} = **625** lb/in^2
Shear parallel to grain F_v = **180** lb/in^2
Modulus of elasticity E = **1600000** lb/in^2
Modulus of elasticity, stability calculations E_{min} = **580000** lb/in^2
Mean shear modulus $G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition **Dry**
Length of span L_{s1} = **9.67** ft
Length of bearing L_b = **1.5** in
Load duration **Ten years**

Section properties

Cross sectional area of member $A = N \times b \times d$ = **67.50** in^2
Section modulus $S_x = N \times b \times d^2 / 6$ = **126.56** in^3
$S_y = d \times (N \times b)^2 / 6$ = **67.50** in^3
Second moment of area $I_x = N \times b \times d^3 / 12$ = **711.91** in^4
$I_y = d \times (N \times b)^3 / 12$ = **202.50** in^4

Adjustment factors

Load duration factor - Table 2.3.2 C_D = **1.00**
Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for bending - Table 4A C_{Fb} = **1.00**
Size factor for tension - Table 4A C_{Ft} = **1.00**
Size factor for compression - Table 4A C_{Fc} = **1.00**
Flat use factor - Table 4A C_{fu} = **1.20**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9	C_r = **1.15**
Bearing area factor - cl.3.10.4	C_b = **1.00**
Depth-to-breadth ratio	$d_{nom} / (N \times b_{nom})$ = **1.50**
- Beam is fully restrained	
Beam stability factor - cl.3.3.3	C_L = **1.00**

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain	$F_{c\_perp}' = F_{c\_perp} \times C_t \times C_{ic\_perp} \times C_b$ = **625** lb/in^2
Applied compression stress perpendicular to grain	$f_{c\_perp} = R_{A\_max} / (N \times b \times L_b)$ = **187** lb/in^2
	f_{c_perp} / F_{c_perp}' = **0.299**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress	$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **1035** lb/in^2
Actual bending stress	$f_b = M / S_x$ = **385** lb/in^2
	f_b / F_b' = **0.372**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress	$F_v' = F_v \times C_D \times C_t \times C_i$ = **180** lb/in^2
Actual shear stress - eq.3.4-2	$f_v = 3 \times F / (2 \times A)$ = **37** lb/in^2
	f_v / F_v' = **0.207**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

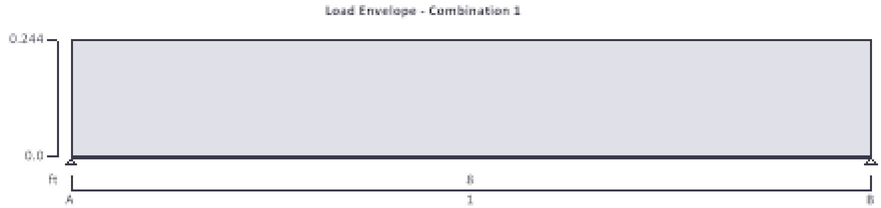
Modulus of elasticity for deflection	$E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1600000** lb/in^2
Design deflection	$\delta_{adm} = 0.003 \times L_{s1}$ = **0.348** in
Total deflection	δ_{b_s1} = **0.060** in
	$\delta_{b\_s1} / \delta_{adm}$ = **0.172**

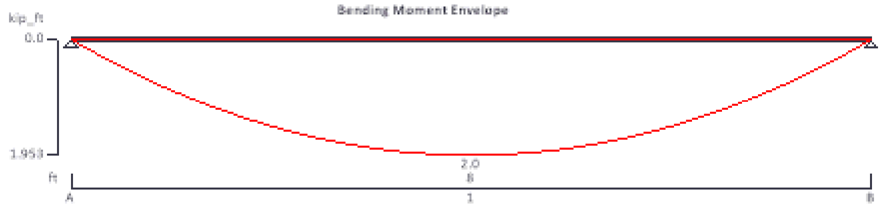
PASS - Total deflection is less than design deflection

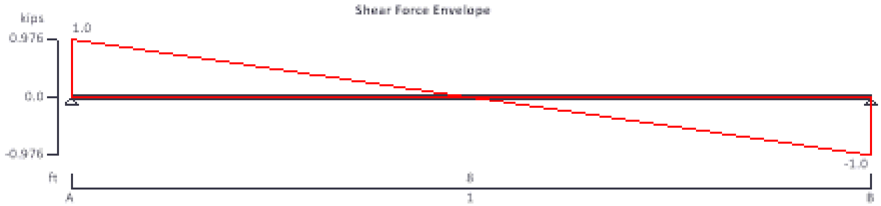
STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

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

Tedds calculation version 1.7.09







Applied loading

Beam loads

	Dead self weight of beam × 1
	Dead full UDL 40 lb/ft
	Live full UDL 200 lb/ft

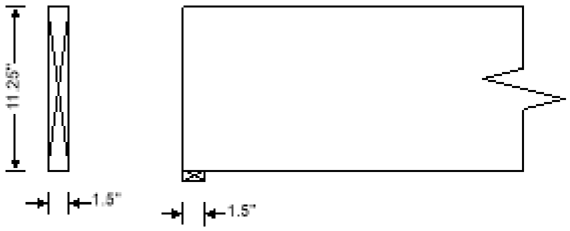
Load combinations

Load combination 1	Support A	Dead × 1.00
		Live × 1.00
	Span 1	Dead × 1.00
		Live × 1.00
	Support B	Dead × 1.00
		Live × 1.00

Analysis results

Maximum moment	M_{max} = **1953** lb_ft	M_{min} = **0** lb_ft
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **1953** lb_ft	
Maximum shear	F_{max} = **976** lb	F_{min} = **-976** lb
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **976** lb	
Total load on member	W_{tot} = **1953** lb	
Reaction at support A	R_{A_max} = **976** lb	R_{A_min} = **976** lb

Unfactored dead load reaction at support A	R_{A_Dead} = **176** lb	
Unfactored live load reaction at support A	R_{A_Live} = **800** lb	
Reaction at support B	R_{B_max} = **976** lb	R_{B_min} = **976** lb
Unfactored dead load reaction at support B	R_{B_Dead} = **176** lb	
Unfactored live load reaction at support B	R_{B_Live} = **800** lb	



Sawn lumber section details

Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **12** in
Dressed depth of sections	d = **11.25** in
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **1.5** in
Species, grade and size classification	Douglas Fir-Larch, No.2 grade, 2" & wider
Bending parallel to grain	F_b = **900** lb/in^2
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Shear parallel to grain	F_v = **180** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Modulus of elasticity, stability calculations	E_{min} = **580000** lb/in^2
Mean shear modulus	$G_{def} = E / 16$ = **100000** lb/in^2

Member details

Service condition	**Dry**
Length of span	L_{s1} = **8** ft
Length of bearing	L_b = **1.5** in
Load duration	**Ten years**

Section properties

Cross sectional area of member	$A = N \times b \times d$ = **16.87** in^2
Section modulus	$S_x = N \times b \times d^2 / 6$ = **31.64** in^3
	$S_y = d \times (N \times b)^2 / 6$ = **4.22** in^3
Second moment of area	$I_x = N \times b \times d^3 / 12$ = **177.98** in^4
	$I_y = d \times (N \times b)^3 / 12$ = **3.16** in^4

Adjustment factors

Load duration factor - Table 2.3.2	C_D = **1.00**
Temperature factor - Table 2.3.3	C_t = **1.00**
Size factor for bending - Table 4A	C_{Fb} = **1.00**
Size factor for tension - Table 4A	C_{Ft} = **1.00**
Size factor for compression - Table 4A	C_{Fc} = **1.00**
Flat use factor - Table 4A	C_{fu} = **1.20**

Incising factor for modulus of elasticity - Table 4.3.8

C_{iE} = **1.00**

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

C_i = **1.00**

Incising factor for perpendicular compression - Table 4.3.8

C_{ic_perp} = **1.00**

Repetitive member factor - cl.4.3.9 C_r = **1.00**

Bearing area factor - cl.3.10.4 C_b = **1.00**

Depth-to-breadth ratio $d_{nom} / (N \times b_{nom})$ = **6.00**

- Beam is fully restrained

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

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c\_perp}' = F_{c\_perp} \times C_t \times C_{ic\_perp} \times C_b$ = **625** lb/in^2

Applied compression stress perpendicular to grain $f_{c\_perp} = R_{B\_max} / (N \times b \times L_b)$ = **434** lb/in^2

f_{c_perp} / F_{c_perp}' = **0.694**

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress $F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r$ = **900** lb/in^2

Actual bending stress $f_b = M / S_x$ = **741** lb/in^2

f_b / F_b' = **0.823**

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress $F_v' = F_v \times C_D \times C_t \times C_i$ = **180** lb/in^2

Actual shear stress - eq.3.4-2 $f_v = 3 \times F / (2 \times A)$ = **87** lb/in^2

f_v / F_v' = **0.482**

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection $E' = E \times C_{ME} \times C_t \times C_{iE}$ = **1600000** lb/in^2

Design deflection $\delta_{adm} = 0.003 \times L_{s1}$ = **0.288** in

Total deflection δ_{b_s1} = **0.079** in

$\delta_{b\_s1} / \delta_{adm}$ = **0.274**

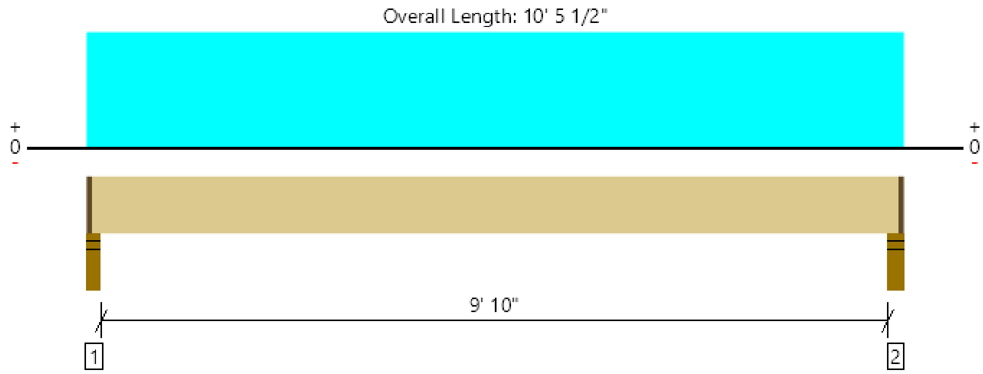
PASS - Total deflection is less than design deflection

Level			
Member Name	**Results**	**Current Solution**	**Comments**
Floor: Joist	Passed	1 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL @ 16" OC	

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

Level, Floor: Joist

1 piece(s) 1 3/4" x 11 1/4" 2.0E Microllam® LVL @ 16" OC



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	834 @ 2 1/2"	2461 (2.25")	Passed (34%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	650 @ 1' 2 3/4"	3741	Passed (17%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2043 @ 5' 2 1/2"	8391	Passed (24%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.075 @ 5' 2 1/2"	0.250	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.092 @ 5' 2 1/2"	0.500	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	66	40	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 4% increase in the moment capacity has been added to account for repetitive member usage.
- A structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Weyerhaeuser Edge™ Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: None.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.50"	157	694	851	1 1/4" Rim Board
2 - Stud wall - DF	4.00"	2.75"	1.50"	158	700	858	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 3" o/c	
Bottom Edge (Lu)	10' 3" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 10' 5 1/2"	16"	22.6	100.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
George Luetje BSE Structural Engineers (913) 492-7400 gluetje@bsestructural.com	

CONNECTIONS

Holdowns (cont.)

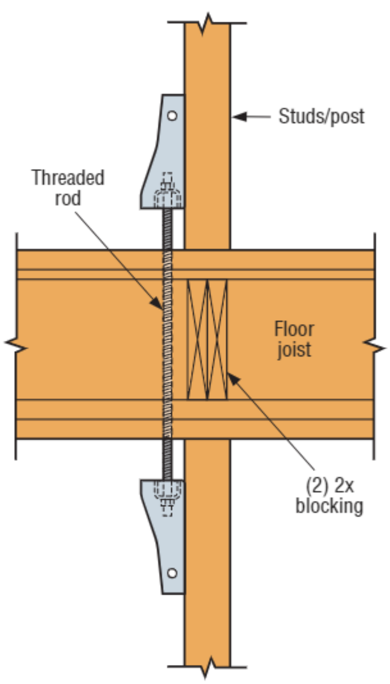
These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

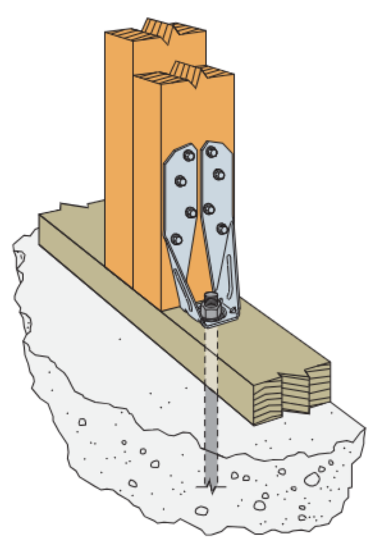
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Ga.	Dimensions (in.) W	H	B	CL	SO	Fasteners (in.) Anchor Bolt Dia. (in.)	Wood Fasteners	Minimum Wood Member Size (in.)	Allowable Tension Loads (160) DF/SP	SPF/HF	Deflection at Allowable Load (in.)	Code Ref.
DTT1Z	14	1½	7⅛	1 7/16	¾	3/16	⅜	(6) #9 x 1½" SD	1½ x 3½	840	840	0.17	IBC, FL, LA
								(6) 0.148 x 1½		910	640	0.167	
								(8) 0.148 x 1½		910	850	0.167	
DTT2Z	14	3¼	6 15/16	1⅝	13/16	3/16	½	(8) ¼ x 1½ SDS	1½ x 3½	1,825	1,800	0.105	
								(8) ¼ x 1½ SDS	3 x 3½	2,145	1,835	0.128	
DTT2Z-SDS2.5								(8) ¼ x 2½ SDS	3 x 3½	2,145	2,105	0.128	
HDU2-SDS2.5	14	3	8 11/16	3¼	1 5/16	1⅜	⅝	(6) ¼ x 2½ SDS	3 x 3½	3,075	2,215	0.088	
HDU4-SDS2.5	14	3	10 15/16	3¼	1 5/16	1⅜	⅝	(10) ¼ x 2½ SDS	3 x 3½	4,565	3,285	0.114	
HDU5-SDS2.5	14	3	13 3/16	3¼	1 5/16	1⅜	⅝	(14) ¼ x 2½ SDS	3 x 3½	5,645	4,340	0.115	
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	(20) ¼ x 2½ SDS	3 x 3½	6,765	5,820	0.11	
									3½ x 3½	6,970	5,995	0.116	
									3½ x 4½	7,870	6,580	0.113	
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	(30) ¼ x 2½ SDS	3½ x 5½	9,535	8,030	0.137	
									3½ x 7¼	11,175	9,610	0.137	
HDU14-SDS2.5	7	3	25 11/16	3½	1 9/16	1 9/16	1	(36) ¼ x 2½ SDS	3½ x 5½	10,770	9,260	0.122	—
									3½ x 7¼	14,390	12,375	0.177	IBC, FL, LA
									5½ x 5½	14,445	12,425	0.172	

1. HDU14 requires heavy-hex anchor nut to achieve tabulated loads (supplied with holdown).
2. HDU14 loads on 4x6 post are applicable to installation on either the narrow or the wide face of the post.
3. **Fasteners:** Nail dimensions are listed diameter by length. SD and SDS screws are Simpson Strong-Tie® Strong-Drive SD Connector and SDS Heavy-Duty Connector screws. See pp. 21–22 for fastener information.



Typical HDU Tie Between Floors



Typical DTT2Z Installation

UPDATED 07/01/22

Holdowns (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

Model No.	Material		Dimensions (in.)							Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load	Code Ref.
	Base (in.)	Body (ga.)	HB	SB	W	H	B	CL	SO	Anchor Dia. Bolt	Stud Bolts		DF/SP	SPF/HF		
HD3B	—	12	4¾	2½	2½	8⅝	2¼	1 5/16	⅜	⅝	(2) ⅝	1½ x 3½	1,895	1,610	0.156	IBC, FL, LA
												2½ x 3½	2,525	2,145	0.169	
												3 x 3½	3,130	3,050	0.12	
												3½ x 3½	3,130	3,050	0.12	
HD5B	3/16	10	5¼	3	2½	9⅜	2½	1¼	2	⅝	(2) ¾	1½ x 3½	2,405	2,070	0.153	
												2½ x 3½	3,750	3,190	0.129	
												3 x 3½	4,505	3,785	0.156	
												3½ x 3½	4,935	4,195	0.15	
HD7B	3/16	10	5¼	3	2½	12⅜	2½	1¼	2	⅞	(3) ¾	3 x 3½	6,645	5,650	0.142	
												3½ x 3½	7,310	6,215	0.154	
												3½ x 4½	7,345	6,245	0.155	
HD9B	⅜	7	6⅛	3½	2⅞	14	2½	1¼	2⅜	⅞	(3) ⅞	3½ x 3½	7,740	6,580	0.159	
												3½ x 4½	9,920	8,430	0.178	
												3½ x 5½	9,920	8,430	0.178	
												3½ x 7¼	10,035	8,530	0.179	
HD12	⅜	3	7	4	3½	20 5/16	4¼	2⅛	3⅝	1	(4) 1	3½ x 3½	11,350	9,215	0.171	
												3½ x 4½	12,665	10,765	0.171	
												5½ x 5½	14,220	12,085	0.162	
										1⅛	(4) 1	3½ x 3½	11,775	9,215	0.171	
												3½ x 4½	13,335	11,055	0.177	
												3½ x 7¼	15,435	13,120	0.194	
												5½ x 5½	15,510	12,690	0.162	
HD19	⅜	3	7	4	3½	24½	4¼	2⅛	3⅝	1⅛	(5) 1	3½ x 7¼	16,735	14,225	0.191	
												5½ x 5½	16,775	12,690	0.2	
										1¼	(5) 1	3½ x 7¼	19,360	15,270	0.18	
												5½ x 5½	19,070	16,210	0.137	

1. To achieve published loads, machine bolts shall be installed with the nut on the opposite side of the holdown. If this orientation is reversed, the designer shall reduce the allowable loads shown per NDS requirements when bolt threads are in the shear plane.
2. All references to bolts are for structural quality through bolts (not lag screw or carriage bolts) equal to or better than ASTM A307, Grade A.
3. HD19 with 1¼" anchor rod requires No.1 post (or better) to achieve published loads.

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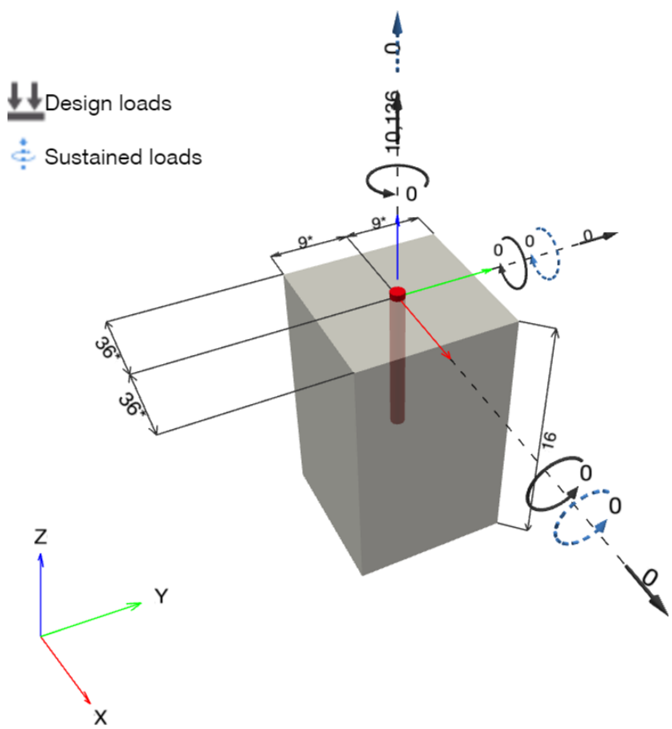
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Jul 19, 2023 (1)	Date:	7/28/2023
Fastening point:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 7/8**
Item number:	not available (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 9.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 36
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 16.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 10{,}136; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;$	no	96

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	10,136	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi\ N_n$	Status
Steel Strength*	10,136	20,085	51	OK
Bond Strength**	10,136	14,033	73	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	10,136	10,590	96	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4868

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	58,000

Calculations

N_{sa} [lb]
26,780

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
26,780	0.750	20,085	10,136

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3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na} \ \psi_{cp,Na} \ N_{ba}$ — ACI 318-19 Eq. (17.6.5.1a)

$\phi \ N_a \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2 \ c_{Na})^2$ — ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10 \ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ — ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ — ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	0.875	9.000	9.000	1.000	1,334

c_{ac} [in.]	λ_a
18.751	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
12.584	453.04	633.46	0.915

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	33,007

Results

N_a [lb]	ϕ_{bond}	$\phi \ N_a$ [lb]	N_{ua} [lb]
21,589	0.650	14,033	10,136

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
9.000	9.000	1.000	18.751	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
486.00	729.00	0.900	1.000	27,155

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
16,293	0.650	10,590	10,136

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Design:	Concrete - Jul 19, 2023 (1)	Date:	7/28/2023
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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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Design:	Concrete - Jul 19, 2023 (1)	Date:	7/28/2023
Fastening point:			

6 Installation data

Anchor type and diameter: HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 7/8

Profile: -

Item number: not available (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Hole diameter in the fixture: -

Maximum installation torque: 1,500 in.lb

Plate thickness (input): -

Hole diameter in the base material: 1.000 in.

Hole depth in the base material: 9.000 in.

Drilling method: Hammer drilled

Minimum thickness of the base material: 11.000 in.

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

7/8 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	36.000	36.000	9.000	9.000

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Fastening point:			

7 Remarks; Your Cooperation Duties

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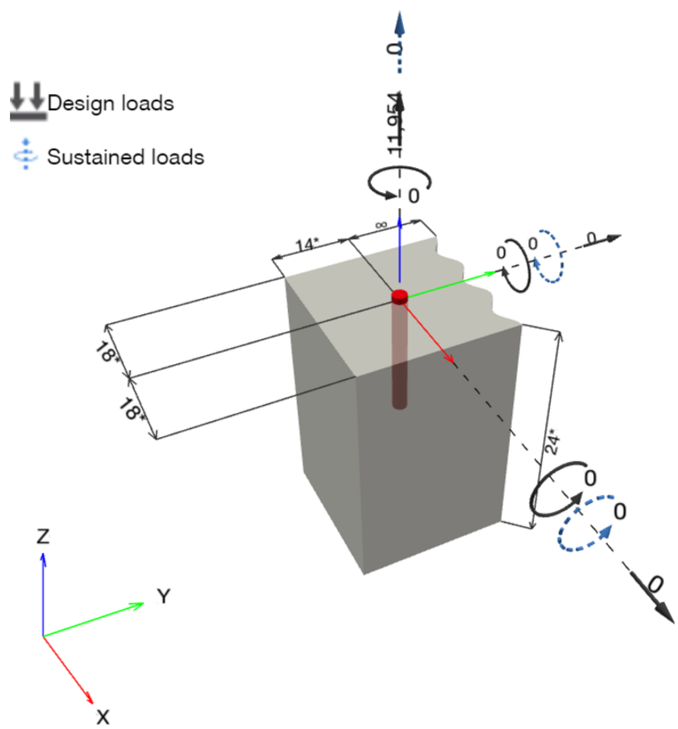
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 7/8**
Item number:	2198007 HAS-E-55 7/8"x10" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 7.500 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 55
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 24.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: > No. 4 bar with stirrups

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 11,954; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	90

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11,954	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	**Capacity ϕ N_n [lb]**	**Utilization $\beta_N = N_{ua}/\phi$ N_n**	**Status**
Steel Strength*	11,954	25,973	47	OK
Bond Strength**	11,954	17,879	67	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	11,954	13,427	90	OK

* highest loaded anchor **anchor group (anchors in tension)

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3.1 Steel Strength

N_{sa} = ESR value   refer to ICC-ES ESR-4868

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.46	75,000

Calculations

N_{sa} [lb]
34,630

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
34,630	0.750	25,973	11,954

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3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$ ACI 318-19 Eq. (17.6.5.1a)

$\phi\ N_a \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	0.875	7.500	14.000	1.000	1,334

c_{ac} [in.]	λ_a
11.367	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
12.584	633.46	633.46	1.000

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	27,506

Results

N_a [lb]	ϕ_{bond}	$\phi\ N_a$ [lb]	N_{ua} [lb]
27,506	0.650	17,879	11,954

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3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N} \ \psi_{c,N} \ \psi_{cp,N} \ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi \ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 \ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \ \lambda_a \ \sqrt{f'_c} \ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
7.500	14.000	1.000	11.367	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
506.25	506.25	1.000	1.000	20,657

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi \ N_{cb}$ [lb]	N_{ua} [lb]
20,657	0.650	13,427	11,954

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 7/8

Item number: 2198007 HAS-E-55 7/8"x10" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Maximum installation torque: 1,500 in.lb

Hole diameter in the base material: 1.000 in.

Hole depth in the base material: 7.500 in.

Minimum thickness of the base material: 9.500 in.

7/8 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	18.000	18.000	14.000	-

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7 Remarks; Your Cooperation Duties

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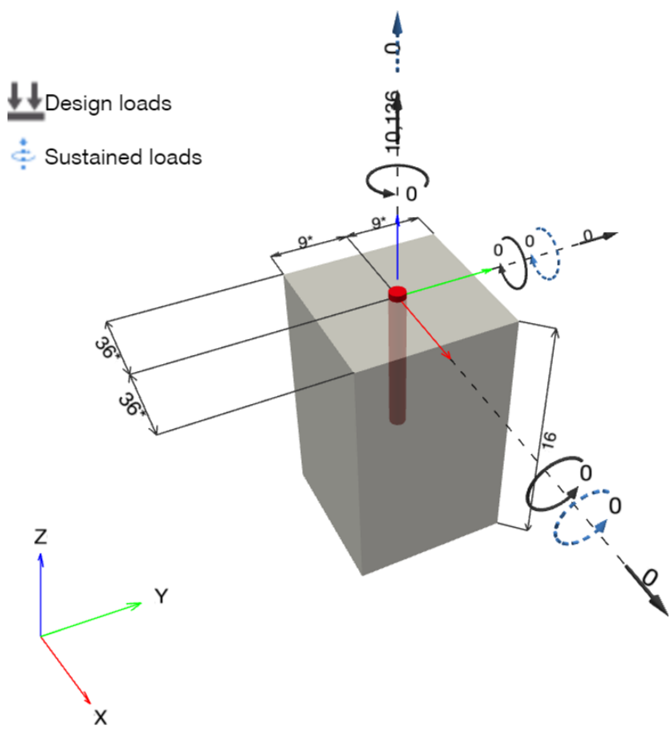
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 1**
Item number:	2198033 HAS-V-36 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 9.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 36
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 16.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Geometry [in.] & Loading [lb, in.lb]



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1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 10,136; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	96

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	10,136	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	10,136	26,348	39	OK
Bond Strength**	10,136	13,991	73	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	10,136	10,590	96	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4868

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	58,000

Calculations

N_{sa} [lb]
35,130

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
35,130	0.750	26,348	10,136

3.2 Bond Strength

N_a	$= \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$	ACI 318-19 Eq. (17.6.5.1a)
$\phi\ N_a$	$\geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Na}	see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)	
A_{Na0}	$= (2\ c_{Na})^2$	ACI 318-19 Eq. (17.6.5.1.2a)
c_{Na}	$= 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-19 Eq. (17.6.5.1.2b)
$\psi_{ed,Na}$	$= 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.4.1b)
$\psi_{cp,Na}$	$= \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.5.1b)
N_{ba}	$= \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	1.000	9.000	9.000	1.000	1,370

c_{ac} [in.]	λ_a
17.775	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
14.382	517.75	827.38	0.888

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	38,745

Results

N_a [lb]	ϕ_{bond}	$\phi\ N_a$ [lb]	N_{ua} [lb]
21,524	0.650	13,991	10,136

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-19 Eq. (17.6.2.1a)
$\phi\ N_{cb} \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-19 Eq. (17.6.2.1.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.2.6.1b)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
9.000	9.000	1.000	17.775	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
486.00	729.00	0.900	1.000	27,155

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
16,293	0.650	10,590	10,136

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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6 Installation data

Anchor type and diameter: HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 1

Profile: -

Item number: 2198033 HAS-V-36 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Hole diameter in the fixture: -

Maximum installation torque: 1,800 in.lb

Plate thickness (input): -

Hole diameter in the base material: 1.125 in.

Hole depth in the base material: 9.000 in.

Drilling method: Hammer drilled

Minimum thickness of the base material: 11.250 in.

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

1 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	36.000	36.000	9.000	9.000

7 Remarks; Your Cooperation Duties

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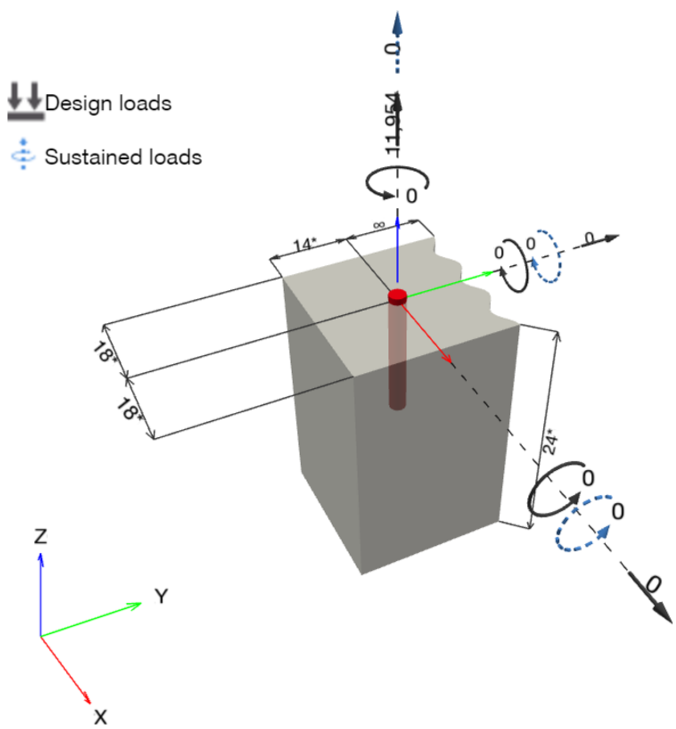
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 1**
Item number:	2198009 HAS-E-55 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 7.500 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 55
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2022 | 11/1/2024
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	
Profile:	
Base material:	cracked concrete, Custom, f_c' = 3,500 psi; h = 24.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: > No. 4 bar with stirrups

Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 11{,}954$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	90

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	11,954	0	0	0

max. concrete compressive strain:	- [‰]
max. concrete compressive stress:	- [psi]
resulting tension force in (x/y)=(0.000/0.000):	0 [lb]
resulting compression force in (x/y)=(0.000/0.000):	0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	11,954	34,072	36	OK
Bond Strength**	11,954	20,543	59	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	11,954	13,427	90	OK

* highest loaded anchor **anchor group (anchors in tension)

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Jul 19, 2023	Date:	7/28/2023
Fastening point:			

3.1 Steel Strength

N_{sa} = ESR value   refer to ICC-ES ESR-4868
$\phi \; N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.61	75,000

Calculations

N_{sa} [lb]
45,430

Results

N_{sa} [lb]	ϕ_{steel}	$\phi \; N_{sa}$ [lb]	N_{ua} [lb]
45,430	0.750	34,072	11,954

3.2 Bond Strength

$N_a = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$ ACI 318-19 Eq. (17.6.5.1a)

$\phi\ N_a \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,296	1.000	7.500	14.000	1.000	1,370

c_{ac} [in.]	λ_a
10.776	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
14.382	816.39	827.38	0.992

$\psi_{cp,Na}$	N_{ba} [lb]
1.000	32,288

Results

N_a [lb]	ϕ_{bond}	$\phi\ N_a$ [lb]	N_{ua} [lb]
31,605	0.650	20,543	11,954

3.3 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{cb} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
7.500	14.000	1.000	10.776	17	1.000	3,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
506.25	506.25	1.000	1.000	20,657

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
20,657	0.650	13,427	11,954

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 V3 + HAS-E-55 (ASTM F1554 Gr.55) 1

Item number: 2198009 HAS-E-55 1"x12" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Maximum installation torque: 1,800 in.lb

Hole diameter in the base material: 1.125 in.

Hole depth in the base material: 7.500 in.

Minimum thickness of the base material: 9.750 in.

1 Hilti HAS Carbon steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	18.000	18.000	14.000	-

Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Jul 19, 2023	Date:	7/28/2023
Fastening point:			

7 Remarks; Your Cooperation Duties

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Current Date: 8/1/2023 11:54 AM
Units system: English
File name: G:\2023\23-283 Wood Spring Suites - Lee's Summit, MO\Calculations\G - CMU\cmu elevator wall-door.msw

Design Results

Masonry wall

GENERAL INFORMATION:

**Global status : ** OK

Design code	:	TMS 402-11 ASD

Geometry:

Total height	:	516.38 [in]
Total length	:	119.25 [in]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Pinned
Rigidity elements	:	Columns

Materials:

Material	:	CMU 1.5-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Partial grouting
Mortar bed type	:	Full bed
Masonry compression strength (F`m)	:	216000 [Lb/ft2]
Steel tension strength (fy)	:	8.64E06 [Lb/ft2]
Steel allowable tension strength (Fs)	:	4.608E06 [Lb/ft2]
Joint reinforcement allowable tension strength (Fs)	:	4.32E06 [Lb/ft2]
Steel elasticity modulus (Es)	:	4.176E09 [Lb/ft2]
Masonry elasticity modulus (Em)	:	1.944E08 [Lb/ft2]
Masonry unit weight	:	135 [Lb/ft3]

Number of stories: **5**

Story	Story height [in]	Wall thickness [in]	Effective unit weight [Lb/ft3]
1	48.00	7.63	98.80
2	114.62	7.63	98.80
3	113.88	7.63	98.80
4	113.88	7.63	98.80
5	120.00	5.63	100.18

Openings:

Reference	X Coordinate [in]	Y Coordinate [in]	Width [in]	Height [in]
Lower left	18.12	48.00	47.04	92.04
Lower left	18.12	162.60	47.04	92.04
Lower left	18.12	276.48	47.04	92.04
Lower left	18.12	390.36	47.04	92.04

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
W	No	WIND	Wind Load
S	No	SNOW	Snow Load
E	No	EARTH	Earth Pressure Load
EQ	No	EQ	Seismic
D1	Yes		DL
D2	Yes		DL+LL
D3	Yes		DL+LL+E
D4	Yes		DL+LL+0.6E
D5	Yes		DL+S
D6	Yes		DL+0.75LL
D7	Yes		DL+0.75S
D8	Yes		DL+0.75LL+0.75S
D9	Yes		DL+0.6W
D10	Yes		DL+0.7EQ
D11	Yes		DL+0.75LL+0.45W+0.75S
D12	Yes		DL+0.525EQ
D13	Yes		DL+0.75S
D14	Yes		DL+0.525EQ+0.75S
D15	Yes		0.6DL+0.6W
D16	Yes		0.6DL+0.6W+E
D17	Yes		0.6DL+0.6W+0.6E
D18	Yes		0.6DL+0.7EQ
D19	Yes		0.6DL+0.7EQ+E
D20	Yes		0.6DL+0.7EQ+0.6E
S1	Yes		DL
S2	Yes		DL+LL
S3	Yes		DL+LL+E
S4	Yes		DL+LL+0.6E
S5	Yes		DL+S
S6	Yes		DL+0.75LL
S7	Yes		DL+0.75S
S8	Yes		DL+0.75LL+0.75S
S9	Yes		DL+0.6W
S10	Yes		DL+0.7EQ
S11	Yes		DL+0.75LL+0.45W+0.75S
S12	Yes		DL+0.525EQ
S13	Yes		DL+0.75S
S14	Yes		DL+0.525EQ+0.75S
S15	Yes		0.6DL+0.6W
S16	Yes		0.6DL+0.6W+E
S17	Yes		0.6DL+0.6W+0.6E
S18	Yes		0.6DL+0.7EQ
S19	Yes		0.6DL+0.7EQ+E
S20	Yes		0.6DL+0.7EQ+0.6E

Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [in]
1	DL	Vertical	135.00	3.81
1	LL	Vertical	375.00	3.81
2	DL	Vertical	135.00	3.81
2	LL	Vertical	375.00	3.81
3	DL	Vertical	135.00	3.81
3	LL	Vertical	375.00	3.81
4	DL	Vertical	135.00	3.81

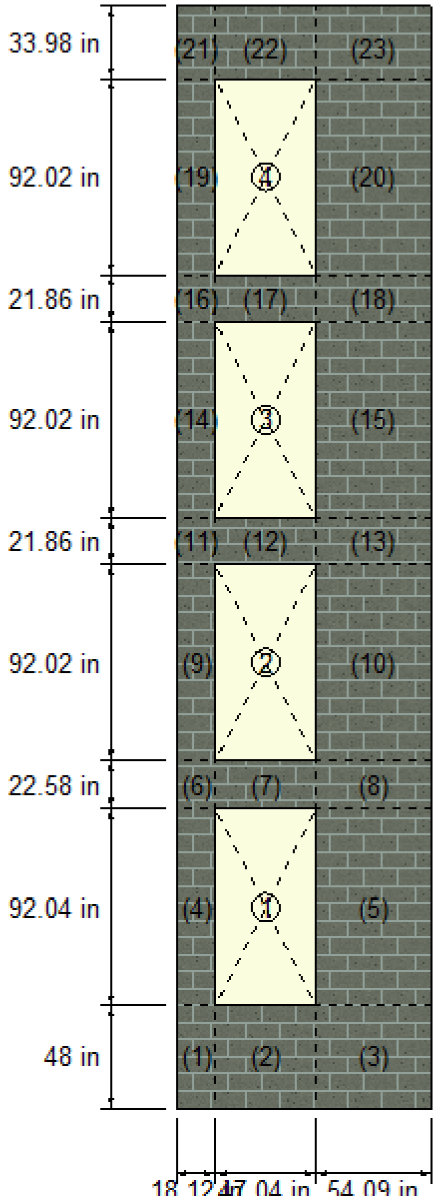
4	LL	Vertical	375.00	3.81
5	DL	Vertical	135.00	3.81
5	LL	Vertical	375.00	3.81
2	W	Horizontal	6.00	0.00
3	W	Horizontal	13.00	0.00
4	W	Horizontal	20.00	0.00
5	W	Horizontal	27.00	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	E	0.41
2	W	-0.01
3	W	-0.01
4	W	-0.01
5	W	-0.01
Parapet	W	-0.01

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [in]	Y Coordinate [in]	Width [in]	Height [in]
1	0.00	0.00	18.12	48.00
2	18.12	0.00	47.04	48.00
3	65.16	0.00	54.09	48.00
4	0.00	48.00	18.12	92.04
5	65.16	48.00	54.09	92.04
6	0.00	140.04	18.12	22.58
7	18.12	140.04	47.04	22.58
8	65.16	140.04	54.09	22.58
9	0.00	162.62	18.12	92.02
10	65.16	162.62	54.09	92.02
11	0.00	254.64	18.12	21.86
12	18.12	254.64	47.04	21.86
13	65.16	254.64	54.09	21.86
14	0.00	276.50	18.12	92.02
15	65.16	276.50	54.09	92.02
16	0.00	368.52	18.12	21.86

17	18.12	368.52	47.04	21.86
18	65.16	368.52	54.09	21.86
19	0.00	390.38	18.12	92.02
20	65.16	390.38	54.09	92.02
21	0.00	482.40	18.12	27.98
22	18.12	482.40	47.04	27.98
23	65.16	482.40	54.09	27.98

Vertical reinforcement:

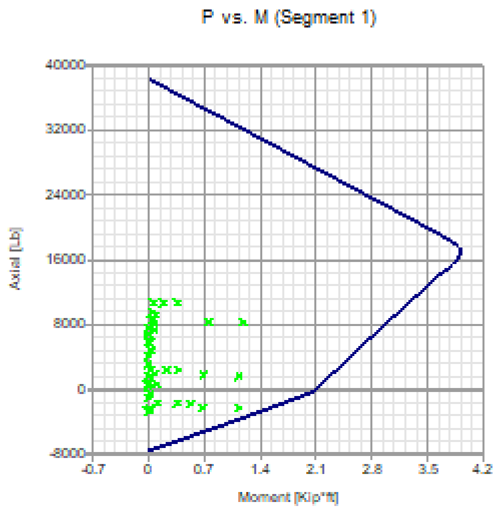
Segment	Bars	Spacing [in]	Ld [in]
1	1-#5	24.00	39.33
2	2-#5	32.00	39.33
3	2-#5	32.00	39.33
4	2-#5	8.00	39.33
5	2-#5	32.00	39.33
6	2-#5	8.00	39.33
7	2-#5	32.00	39.33
8	2-#5	32.00	39.33
9	2-#5	8.00	39.33
10	2-#5	32.00	39.33
11	2-#5	8.00	39.33
12	2-#5	32.00	39.33
13	2-#5	32.00	39.33
14	1-#5	32.00	39.33
15	2-#5	32.00	39.33
16	1-#5	32.00	39.33
17	2-#5	32.00	39.33
18	2-#5	32.00	39.33
19	1-#5	32.00	39.33
20	2-#5	32.00	39.33
21	1-#5	32.00	39.33
22	2-#5	32.00	39.33
23	2-#5	32.00	39.33

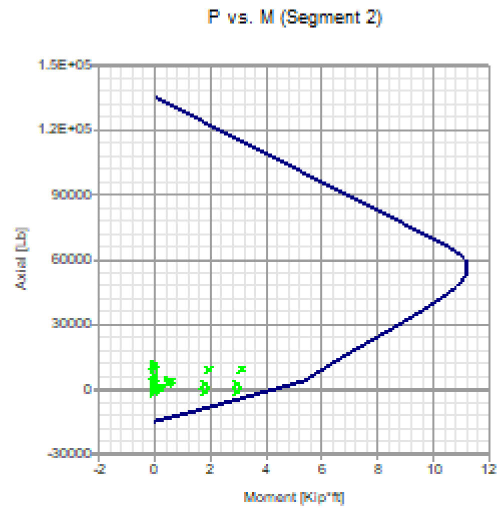
Results: Combined axial flexure

Segment	Condition	P [Lb]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D19(Max)	-2213.46	-1.13	1.52	0.74
2	D19(Max)	-723.57	-2.96	4.00	0.74
3	D16(Max)	9737.16	-3.22	6.78	0.48
4	D19(Max)	-1731.04	0.60	2.86	0.21
5	D19(Max)	14199.45	1.63	7.34	0.22
6	D3(Bottom)	9788.61	-0.23	3.79	0.06
7	D3(Max)	2655.15	-0.62	4.93	0.13
8	D16(Max)	7506.20	-0.49	6.50	0.08
9	D10(Max)	1088.43	0.23	3.06	0.07
10	D10(Bottom)	14001.41	0.60	7.32	0.08
11	D19(Max)	699.47	0.16	3.03	0.05
12	D2(Max)	1344.02	-0.06	4.57	0.01
13	D19(Max)	6039.19	0.43	6.32	0.07
14	D19(Bottom)	406.43	0.20	1.73	0.11
15	D19(Bottom)	5626.42	0.55	6.27	0.09
16	D18(Top)	337.77	0.18	1.71	0.10
17	D3(Top)	2064.56	-0.60	4.77	0.13
18	D18(Max)	3137.91	0.48	5.69	0.08
19	D3(Top)	2056.13	-0.33	1.31	0.25

20	D3(Top)	3945.38	-0.70	3.74	0.19
21	D3(Max)	2056.08	-0.33	1.31	0.25
22	D3(Max)	1961.17	-0.62	3.14	0.20
23	D2(Top)	2587.82	-0.75	3.63	0.21

Interaction diagrams, P vs. M:





Results: Axial compression

Segment	Condition	P [Lb]	Pa [Lb]	Ratio
1	D4(Top)	10739.44	20876.51	0.51
2	D4(Bottom)	11581.08	73767.10	0.16
3	D2(Top)	24655.30	84822.75	0.29
4	D2(Max)	10399.76	44455.16	0.23
5	D4(Bottom)	22829.06	77960.58	0.29
6	D4(Bottom)	9788.61	44455.16	0.22
7	D4(Max)	2655.15	67799.32	0.04
8	D2(Max)	20840.76	77960.58	0.27
9	D2(Max)	7806.20	44547.39	0.18
10	D2(Max)	16694.29	78068.25	0.21
11	D2(Bottom)	7065.95	44547.39	0.16
12	D2(Max)	1344.02	67892.97	0.02
13	D4(Bottom)	14536.14	78068.25	0.19
14	D4(Bottom)	5812.73	19253.28	0.30
15	D4(Bottom)	11732.28	78068.25	0.15
16	D4(Bottom)	4571.34	19253.28	0.24
17	D4(Max)	2064.56	67892.97	0.03
18	D4(Bottom)	9096.92	78068.25	0.12
19	D2(Max)	2904.67	13373.89	0.22
20	D2(Max)	5585.13	52892.95	0.11
21	D4(Bottom)	2056.08	13373.89	0.15
22	D4(Max)	1961.17	45998.98	0.04
23	D2(Bottom)	3945.33	52892.95	0.07

Results: Axial tension

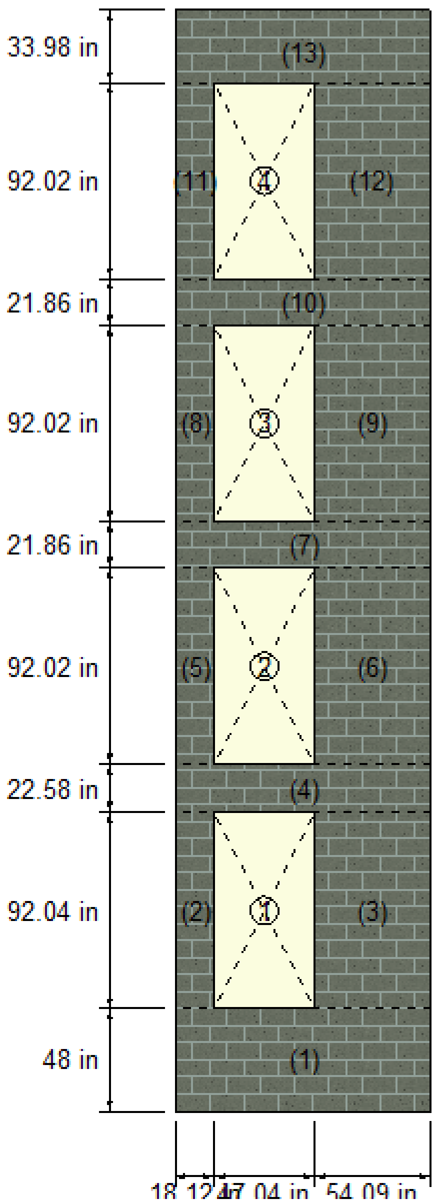
Segment	Condition	ft [Lb/ft2]	Fs [Lb/ft2]	Ratio
1	D20(Bottom)	1614104.00	4608000.00	0.35
2	D20(Max)	228645.90	4608000.00	0.05
3	D1(Top)	0.00	4608000.00	0.00
4	D18(Max)	369046.70	4608000.00	0.08
5	D1(Top)	0.00	4608000.00	0.00
6	D20(Bottom)	350045.50	4608000.00	0.08
7	D1(Top)	0.00	4608000.00	0.00
8	D1(Top)	0.00	4608000.00	0.00
9	D19(Max)	149039.60	4608000.00	0.03
10	D1(Top)	0.00	4608000.00	0.00
11	D20(Bottom)	127398.40	4608000.00	0.03
12	D1(Top)	0.00	4608000.00	0.00
13	D1(Top)	0.00	4608000.00	0.00
14	D1(Top)	0.00	4608000.00	0.00
15	D1(Top)	0.00	4608000.00	0.00
16	D1(Top)	0.00	4608000.00	0.00
17	D1(Top)	0.00	4608000.00	0.00
18	D1(Top)	0.00	4608000.00	0.00
19	D1(Top)	0.00	4608000.00	0.00
20	D1(Top)	0.00	4608000.00	0.00
21	D1(Top)	0.00	4608000.00	0.00
22	D1(Top)	0.00	4608000.00	0.00
23	D1(Top)	0.00	4608000.00	0.00

Results: Shear

Segment	Condition	fv [Lb/ft2]	Fv [Lb/ft2]	Ratio
1	D19(Top)	3180.505	6274.233	0.51
2	D19(Max)	1521.761	6274.233	0.24
3	D19(Top)	2695.143	8904.033	0.30
4	D19(Bottom)	415.885	6274.233	0.07
5	D19(Bottom)	338.345	8751.441	0.04
6	D18(Max)	325.645	6274.233	0.05
7	D2(Max)	264.814	6806.869	0.04
8	D18(Top)	230.633	8126.631	0.03
9	D18(Bottom)	226.472	6274.233	0.04
10	D18(Max)	201.328	7990.612	0.03
11	D19(Top)	239.913	6638.497	0.04
12	D2(Max)	46.808	6543.850	0.01
13	D19(Top)	204.085	7327.818	0.03
14	D19(Bottom)	211.889	6485.892	0.03
15	D19(Max)	190.565	7255.807	0.03
16	D10(Max)	385.099	6811.068	0.06
17	D2(Max)	312.122	6688.392	0.05
18	D18(Max)	234.406	6821.666	0.03
19	D10(Bottom)	355.055	6936.273	0.05
20	D10(Bottom)	294.312	7194.871	0.04
21	D2(Bottom)	263.489	7725.361	0.03
22	D3(Bottom)	344.145	6872.098	0.05
23	D2(Bottom)	174.562	7207.038	0.02

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [in]	Y Coordinate [in]	Width [in]	Height [in]
1	0.00	0.00	119.25	48.00
2	0.00	48.00	18.12	92.04
3	65.16	48.00	54.09	92.04
4	0.00	140.04	119.25	22.58
5	0.00	162.62	18.12	92.02
6	65.16	162.62	54.09	92.02
7	0.00	254.64	119.25	21.86
8	0.00	276.50	18.12	92.02
9	65.16	276.50	54.09	92.02
10	0.00	368.52	119.25	21.86
11	0.00	390.38	18.12	92.02
12	65.16	390.38	54.09	92.02
13	0.00	482.40	119.25	27.98

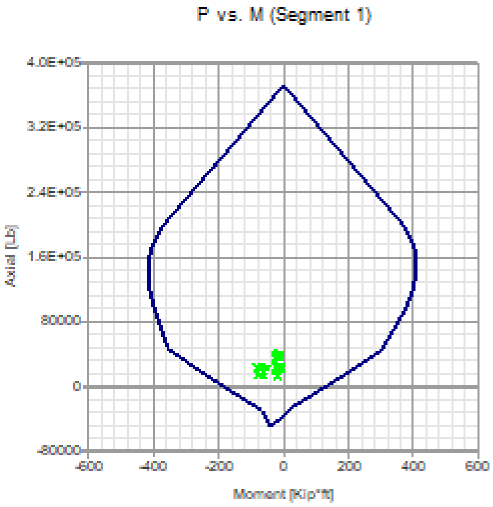
Reinforcement:

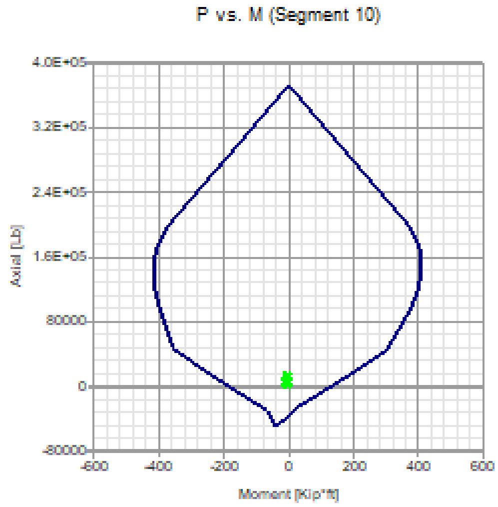
	Vertical reinforcement			Horizontal reinforcement		
Segment	**Bars**	**Spacing** [in]	**Ld** [in]	**Bars**	**Spacing** [in]	**Ld** [in]
1	1-#5	24.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
2	2-#5	8.00	0.00	--	0.00	0.00
3	2-#5	32.00	0.00	--	0.00	0.00
4	2-#5	8.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
5	2-#5	8.00	0.00	--	0.00	0.00
6	2-#5	32.00	0.00	--	0.00	0.00
7	2-#5	8.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
8	1-#5	32.00	0.00	--	0.00	0.00
9	2-#5	32.00	0.00	--	0.00	0.00
10	1-#5	32.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
11	1-#5	32.00	0.00	--	0.00	0.00
12	2-#5	32.00	0.00	--	0.00	0.00
13	1-#5	32.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00
	2-#5	32.00	0.00	--	0.00	0.00

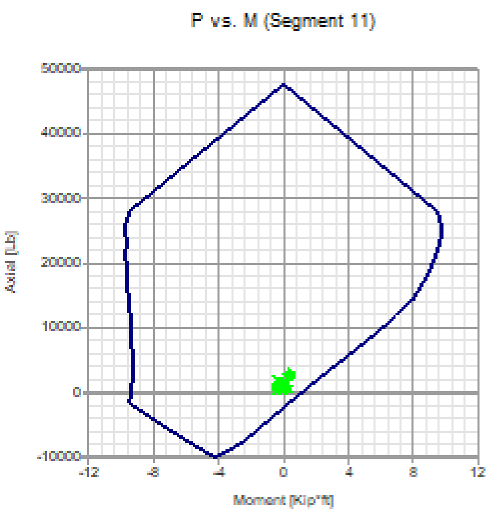
Results: Combined axial flexure

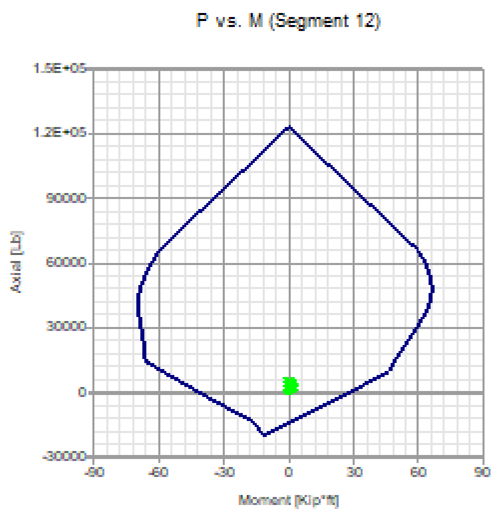
Segment	**Condition**	**P** [Lb]	**M** [Kip*ft]	**Ma** [Kip*ft]	**Ratio**
1	D18(Bottom)	14578.38	-76.14	237.12	0.32
2	D18(Bottom)	-1727.85	-1.55	10.17	0.15
3	D20(Bottom)	14194.00	-17.28	66.78	0.26
4	D20(Bottom)	10700.76	-42.52	294.20	0.14
5	D20(Bottom)	-692.77	-1.74	10.73	0.16
6	D18(Bottom)	9832.50	-6.48	59.11	0.11
7	D10(Bottom)	12236.94	-23.97	299.95	0.08
8	D20(Top)	318.65	0.71	1.54	0.46
9	D18(Top)	3706.93	3.04	35.22	0.09
10	D10(Max)	6653.99	-7.88	207.54	0.04
11	D18(Top)	247.70	0.33	1.23	0.27
12	D10(Top)	2009.95	2.69	31.48	0.09
13	D20(Bottom)	1630.67	-0.56	183.75	0.00

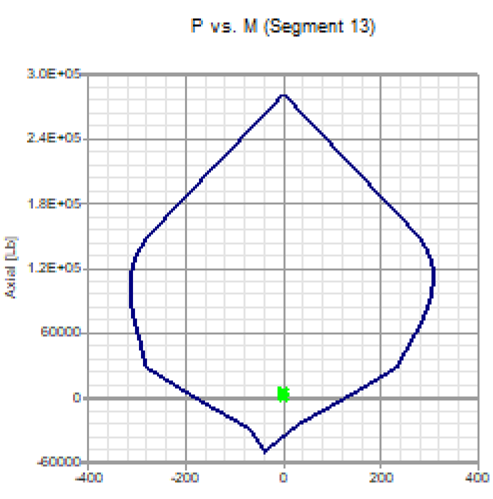
Interaction diagrams, P vs. M:

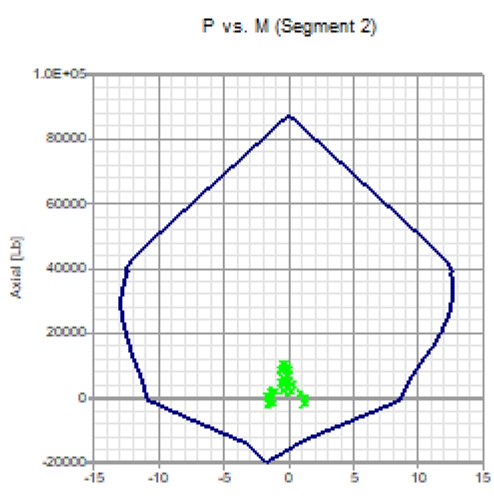

P vs. M (Segment 1)
4.0E+05
3.2E+05
2.4E+05
1.6E+05
80000
0
-80000
-600
-400
-200
0
200
400
600
Axial [Lb]
Moment [Kip*ft]

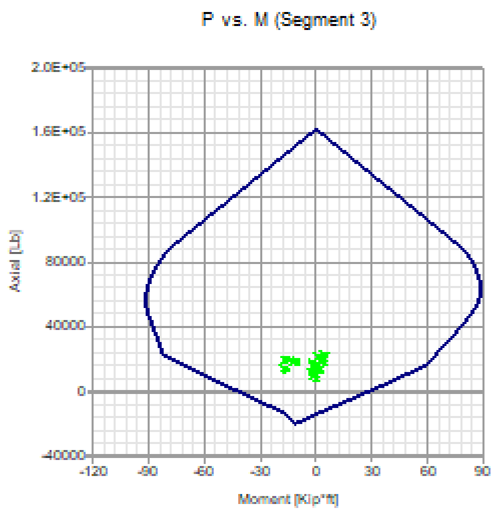

P vs. M (Segment 10)
4.0E+05
3.2E+05
2.4E+05
1.6E+05
80000
0
-80000
-600
-400
-200
0
200
400
600
Axial [Lb]
Moment [Kip*ft]

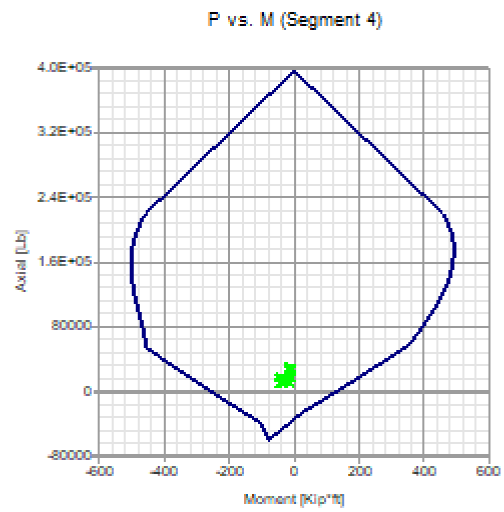

P vs. M (Segment 11)
50000
40000
30000
20000
10000
0
-10000
-12
-8
-4
0
4
8
12
Axial [Lb]
Moment [Kip*ft]

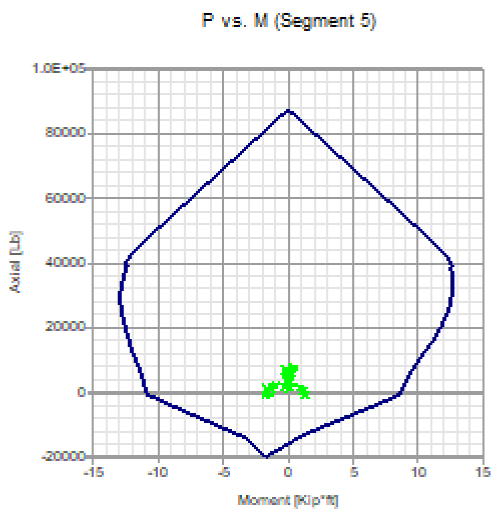

P vs. M (Segment 12)
1.5E+05
1.2E+05
90000
60000
30000
0
-30000
-90
-60
-30
0
30
60
90
Axial [Lb]
Moment [Kip*ft]

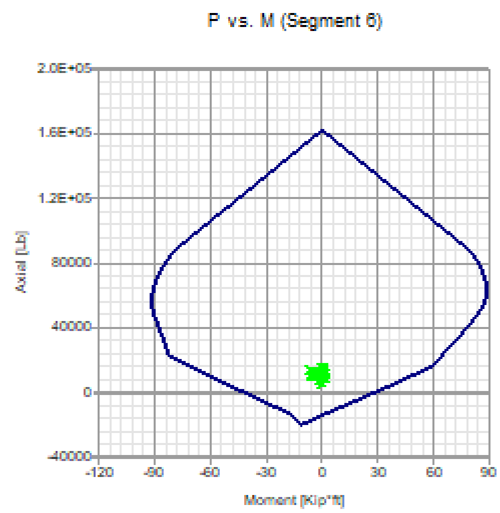

P vs. M (Segment 13)
3.0E+05
2.4E+05
1.8E+05
1.2E+05
60000
0
-60000
-400
-200
0
200
400
Axial [Lb]
Moment [Kip*ft]

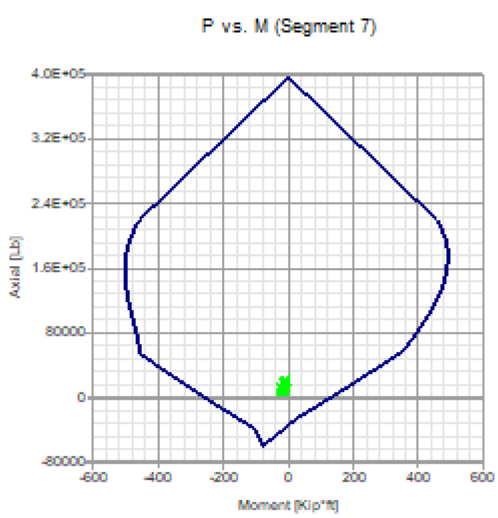

P vs. M (Segment 2)
1.0E+05
80000
60000
40000
20000
0
-20000
-15
-10
-5
0
5
10
15
Axial [Lb]
Moment [Kip*ft]

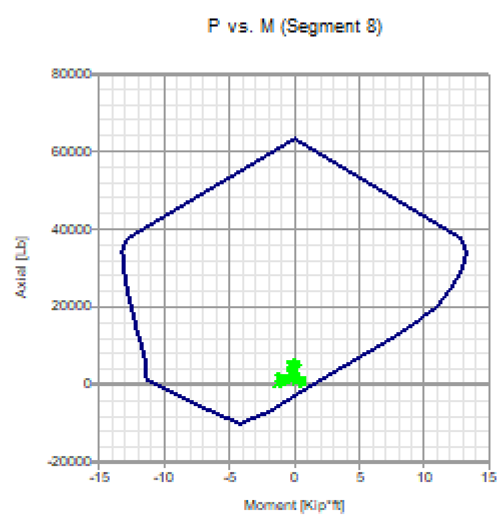

P vs. M (Segment 3)
Axial [Lb]
Moment [Kip*ft]

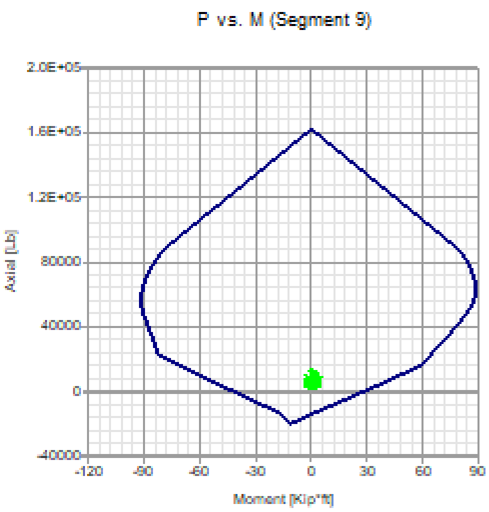

P vs. M (Segment 4)
Axial [Lb]
Moment [Kip*ft]


P vs. M (Segment 5)
Axial [Lb]
Moment [Kip*ft]


P vs. M (Segment 6)
Axial [Lb]
Moment [Kip*ft]


P vs. M (Segment 7)
Axial [Lb]
Moment [Kip*ft]


P vs. M (Segment 8)
Axial [Lb]
Moment [Kip*ft]



Results: Axial compression

Segment	Condition	P [Lb]	Pa [Lb]	Ratio
1	D2(Bottom)	41343.59	202750.10	0.20
2	D2(Max)	10547.25	42022.73	0.25
3	D2(Bottom)	22834.65	80993.65	0.28
4	D4(Bottom)	30687.53	196430.10	0.16
5	D2(Bottom)	7807.49	42106.51	0.19
6	D4(Bottom)	16695.71	81109.07	0.21
7	D2(Bottom)	22146.95	196732.30	0.11
8	D2(Bottom)	5813.25	31335.44	0.19
9	D2(Bottom)	11732.79	81109.07	0.14
10	D4(Bottom)	14534.01	185664.40	0.08
11	D2(Bottom)	2904.30	20569.65	0.14
12	D2(Bottom)	5585.25	54587.05	0.10
13	D2(Bottom)	6504.10	124137.30	0.05

Results: Axial tension

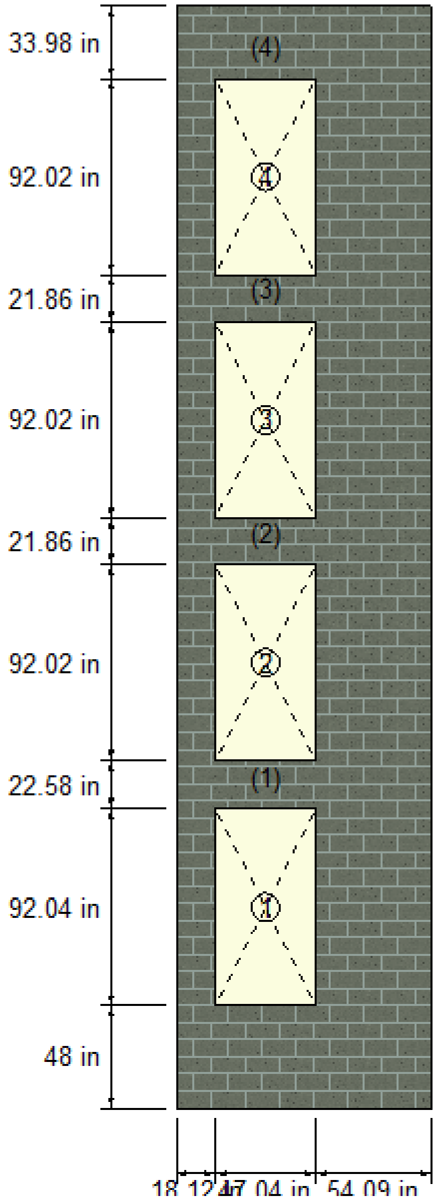
Segment	Condition	ft [Lb/ft2]	Fs [Lb/ft2]	Ratio
1	D1(Top)	0.00	4608000.00	0.00
2	D18(Bottom)	401306.70	4608000.00	0.09
3	D1(Top)	0.00	4608000.00	0.00
4	D1(Top)	0.00	4608000.00	0.00
5	D18(Bottom)	160900.30	4608000.00	0.03
6	D1(Top)	0.00	4608000.00	0.00
7	D1(Top)	0.00	4608000.00	0.00
8	D1(Top)	0.00	4608000.00	0.00
9	D1(Top)	0.00	4608000.00	0.00
10	D1(Top)	0.00	4608000.00	0.00
11	D1(Top)	0.00	4608000.00	0.00
12	D1(Top)	0.00	4608000.00	0.00
13	D1(Top)	0.00	4608000.00	0.00

Results: Shear

Segment	Condition	fv [Lb/ft2]	Fv [Lb/ft2]	Ratio
1	D20(Bottom)	940.504	7258.377	0.13
2	D10(Max)	654.046	6529.748	0.10
3	D18(Max)	1644.997	8540.369	0.19
4	D20(Max)	627.887	6931.954	0.09
5	D20(Max)	694.861	6274.233	0.11
6	D18(Bottom)	1061.570	7899.738	0.13
7	D20(Max)	451.637	6702.997	0.07
8	D20(Bottom)	685.021	6457.204	0.11
9	D10(Max)	666.502	8266.079	0.08
10	D10(Max)	249.167	7147.605	0.03
11	D20(Max)	422.201	6497.396	0.06
12	D10(Max)	399.179	6860.318	0.06
13	D18(Bottom)	77.632	9962.625	0.01

LINTEL DESIGN:

Status : OK



Geometry:

Lintel	X Coordinate [in]	Y Coordinate [in]	Length [in]	Depth [in]
1	18.12	48.00	47.04	16.00
2	18.12	162.60	47.04	16.00
3	18.12	276.48	47.04	16.00
4	18.12	390.36	47.04	16.00

Reinforcement:

	Top long. reinforcement		Bottom long. reinforcement		Transverse reinforcement		
Lintel	Bars	Extent [in]	Bars	Extent [in]	Bars	Spacing [in]	Ld [in]
1	1-#4	15.00	1-#4	6.50	#4	16.00	8.31
2	1-#4	9.00	1-#4	4.50	--	0.00	0.00
3	1-#4	3.50	1-#4	0.50	--	0.00	0.00
4	1-#4	0.00	1-#4	1.00	--	0.00	0.00

Results: Bending

Lintel	Condition	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D10(Bottom)	-4.15	5.97	0.70
2	D18(Bottom)	-3.15	5.79	0.54
3	D18(Bottom)	-1.81	5.79	0.31
4	D2(Top)	0.63	5.79	0.11

Results: Shear

Lintel	Condition	fv [Lb/ft2]	Fv [Lb/ft2]	Ratio
1	D10(Bottom)	8117.635	11154.192	0.73
2	D10(Bottom)	5536.018	6274.233	0.88
3	D10(Bottom)	4248.860	6274.233	0.68
4	D4(Bottom)	3047.858	6274.233	0.49

Results: Deflection

Lintel	Condition	δs [in]	δmax [in]	Ratio
1		0.00	0.00	0.00
2		0.00	0.00	0.00
3		0.00	0.00	0.00
4		0.00	0.00	0.00

Notes:

* P = Axial load
* Pa = Allowable compressive force due to axial load.
* M = Moment at the section under consideration.
* Ma = Wall allowable moment due to axial force or lintel pure flexure allowable moment
* fa = Calculated compressive stress due to axial load only
* fb = Calculated compressive stress due to axial flexure only
* ft = Calculated axial tension
* Fa = Allowable compressive stress due to axial load only
* Fb = Allowable compressive stress due to axial flexure only
* fv = Calculated shear stress
* Fs = Allowable tensile or compressive stress
* Fv = Allowable shear stress
* ld = Embedment length
* As = Effective cross sectional area of reinforcement
* δs = Calculated deflection
* δmax = Maximum allowable deflection

Current Date: 8/1/2023 12:00 PM
Units system: English
File name: G:\2023\23-283 Wood Spring Suites - Lee's Summit, MO\Calculations\G - CMU\cmu elevator wall-solid.msw

Design Results

Masonry wall

GENERAL INFORMATION:

**Global status : ** OK

Design code	:	TMS 402-16 SD
Geometry:		
Total height	:	490.34 [in]
Total length	:	119.25 [in]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Pinned
Rigidity elements	:	Columns
Materials:		
Material	:	CMU 1.5-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Partial grouting
Mortar bed type	:	Full bed
Masonry compression strength (F`m)	:	216000 [Lb/ft2]
Steel tension strength (fy)	:	8.64E06 [Lb/ft2]
Steel allowable tension strength (Fs)	:	3.456E06 [Lb/ft2]
Joint reinforcement allowable tension strength (Fs)	:	4.32E06 [Lb/ft2]
Steel elasticity modulus (Es)	:	4.176E09 [Lb/ft2]
Masonry elasticity modulus (Em)	:	1.944E08 [Lb/ft2]
Masonry unit weight	:	135 [Lb/ft3]

Number of stories: 5

Story	Story height [in]	Wall thickness [in]	Effective unit weight [Lb/ft3]
1	48.00	7.63	79.93
2	114.62	7.63	79.93
3	113.88	7.63	79.93
4	113.88	7.63	79.93
5	93.96	5.63	82.97

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LL	Live Load
W	No	WIND	Wind Load
S	No	SNOW	Snow Load
E	No	EARTH	Earth Pressure Load
EQ	No	EQ	Seismic
D1	Yes		DL
D2	Yes		DL+LL
D3	Yes		DL+LL+E
D4	Yes		DL+LL+0.6E

D5	Yes	DL+S
D6	Yes	DL+0.75LL
D7	Yes	DL+0.75S
D8	Yes	DL+0.75LL+0.75S
D9	Yes	DL+0.6W
D10	Yes	DL+0.7EQ
D11	Yes	DL+0.75LL+0.45W+0.75S
D12	Yes	DL+0.525EQ
D13	Yes	DL+0.75S
D14	Yes	DL+0.525EQ+0.75S
D15	Yes	0.6DL+0.6W
D16	Yes	0.6DL+0.6W+E
D17	Yes	0.6DL+0.6W+0.6E
D18	Yes	0.6DL+0.7EQ
D19	Yes	0.6DL+0.7EQ+E
D20	Yes	0.6DL+0.7EQ+0.6E
S1	Yes	DL
S2	Yes	DL+LL
S3	Yes	DL+LL+E
S4	Yes	DL+LL+0.6E
S5	Yes	DL+S
S6	Yes	DL+0.75LL
S7	Yes	DL+0.75S
S8	Yes	DL+0.75LL+0.75S
S9	Yes	DL+0.6W
S10	Yes	DL+0.7EQ
S11	Yes	DL+0.75LL+0.45W+0.75S
S12	Yes	DL+0.525EQ
S13	Yes	DL+0.75S
S14	Yes	DL+0.525EQ+0.75S
S15	Yes	0.6DL+0.6W
S16	Yes	0.6DL+0.6W+E
S17	Yes	0.6DL+0.6W+0.6E
S18	Yes	0.6DL+0.7EQ
S19	Yes	0.6DL+0.7EQ+E
S20	Yes	0.6DL+0.7EQ+0.6E

Distributed loads:

Consider self weight : DL

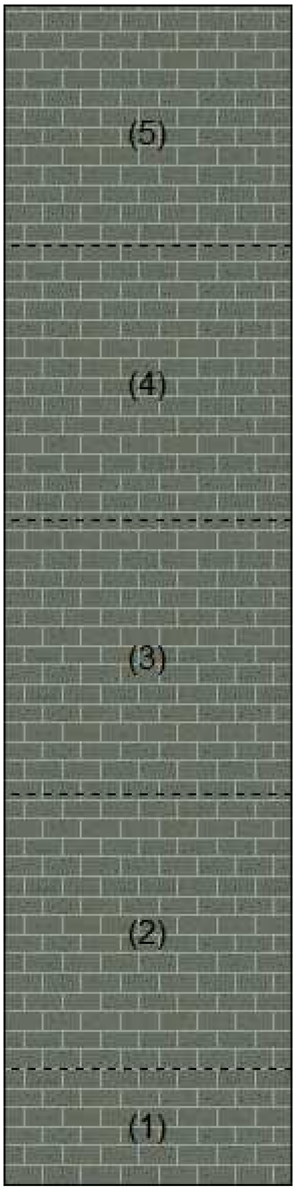
Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [in]
1	DL	Vertical	135.00	3.81
1	LL	Vertical	375.00	3.81
2	DL	Vertical	135.00	3.81
2	LL	Vertical	375.00	3.81
3	DL	Vertical	135.00	3.81
3	LL	Vertical	375.00	3.81
4	DL	Vertical	135.00	3.81
4	LL	Vertical	375.00	3.81
5	DL	Vertical	135.00	3.81
5	LL	Vertical	375.00	3.81
2	W	Horizontal	6.00	0.00
3	W	Horizontal	13.00	0.00
4	W	Horizontal	20.00	0.00
5	W	Horizontal	27.00	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	E	0.41
2	W	-0.01
3	W	-0.01
4	W	-0.01
5	W	-0.01
Parapet	W	-0.01

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [in]	Y Coordinate [in]	Width [in]	Height [in]
1	0.00	0.00	119.25	48.00
2	0.00	48.00	119.25	114.62
3	0.00	162.62	119.25	113.88
4	0.00	276.50	119.25	113.88
5	0.00	390.38	119.25	93.96

Vertical reinforcement:

Segment	Bars	Spacing [in]	Ld [in]
1	4-#5	32.00	39.33
2	4-#5	32.00	39.33
3	4-#5	32.00	39.33
4	4-#5	32.00	39.33
5	4-#5	32.00	39.33

Results: Combined axial flexure

Segment	Condition	Pu [Lb]	Mua [Kip*ft]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D16(Max)	15449.45	6.89	6.90	22.78	0.30
2	D19(Max)	14015.36	3.27	3.29	22.40	0.15
3	D10(Bottom)	17001.27	1.02	1.03	23.20	0.04
4	D19(Bottom)	6416.40	1.07	1.07	20.34	0.05
5	D3(Top)	5355.21	1.61	1.61	14.40	0.11

Results: Flexural reinforcement area

Segment	Condition	Pu [Lb]	As [in2]	Asmax [in2]	Ratio
1	D1(Top)	0.00	1.16	2.65	0.44
2	D1(Top)	0.00	1.16	2.65	0.44
3	D1(Top)	0.00	1.16	2.65	0.44
4	D1(Top)	0.00	1.16	2.65	0.44
5	D1(Top)	0.00	1.16	2.05	0.56

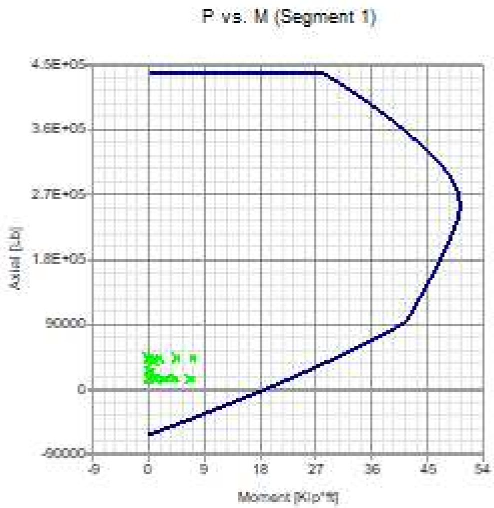
Intermediate results for axial-bending

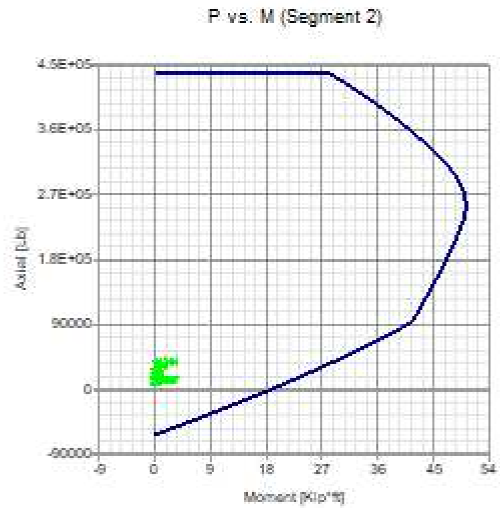
Segment	Condition	c [in]	d [in]	Mcr [Kip*ft]
1	D16(Max)	0.76	3.81	1.05
2	D19(Max)	0.74	3.81	1.02
3	D10(Bottom)	0.77	3.81	0.96
4	D19(Bottom)	0.67	3.81	0.90
5	D3(Top)	0.66	2.81	0.48

Inertias

Segment	Condition	Ig [in4]	Icr [in4]
1	D16(Max)	355.60	33.19
2	D19(Max)	355.60	32.71
3	D10(Bottom)	355.60	33.70
4	D19(Bottom)	355.60	30.20
5	D3(Top)	147.06	15.55

Interaction diagrams, P vs. M:





Results: Axial compression

Segment	Condition	Pu [Lb]	ϕPn [Lb]	Ratio
1	D4(Bottom)	45278.44	430860.10	0.11
2	D2(Max)	38233.45	396003.40	0.10
3	D4(Bottom)	28152.35	396550.30	0.07
4	D4(Bottom)	18126.42	396550.30	0.05
5	D2(Bottom)	8176.91	293804.90	0.03

Axial stress

Segment	Condition	Pu [Lb]	Pu/Ag [Lb/ft2]	Fn [Lb/ft2]	Ratio
1	D4(Bottom)	45278.44	12824.36	43200.00	0.30
2	D2(Max)	38233.45	10828.98	43200.00	0.25
3	D4(Bottom)	28152.35	7973.68	43200.00	0.18
4	D4(Bottom)	18126.42	5134.01	43200.00	0.12
5	D4(Bottom)	8176.91	3040.20	43200.00	0.07

Results: Shear

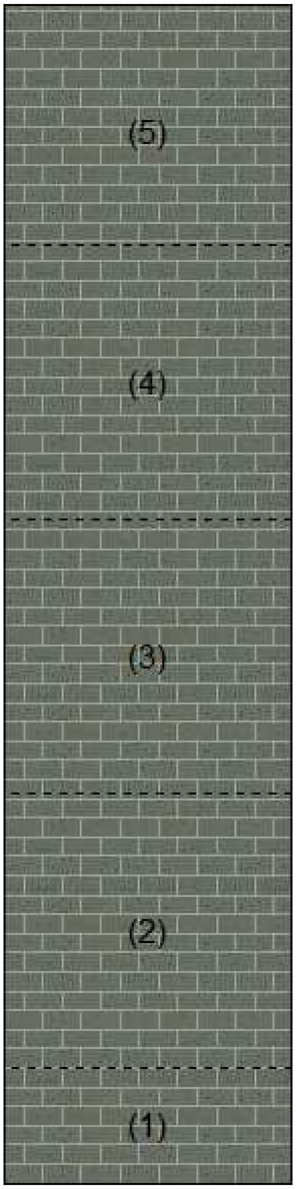
Segment	Condition	Vu [Lb]	ϕVn [Lb]	Ratio
1	D19(Top)	793.51	3492.80	0.23
2	D19(Max)	85.17	3473.56	0.02
3	D19(Top)	51.51	3339.86	0.02
4	D19(Max)	52.09	3320.63	0.02
5	D10(Bottom)	44.44	2444.69	0.02

Deflection

Segment	Condition	δs [in]	δmax [in]	δs/δmax
1	S3(Max)	0.00	0.34	0.01
2	S3(Bottom)	0.01	0.80	0.01
3	S10(Bottom)	0.00	0.80	0.00
4	S19(Bottom)	0.00	0.80	0.00
5	S2(Top)	0.01	0.66	0.01

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [in]	Y Coordinate [in]	Width [in]	Height [in]
1	0.00	0.00	119.25	48.00
2	0.00	48.00	119.25	114.62
3	0.00	162.62	119.25	113.88
4	0.00	276.50	119.25	113.88
5	0.00	390.38	119.25	93.96

Reinforcement:

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	4-#5	32.00	0.00	--	0.00	0.00
2	4-#5	32.00	0.00	--	0.00	0.00
3	4-#5	32.00	0.00	--	0.00	0.00
4	4-#5	32.00	0.00	--	0.00	0.00

5	4-#5	32.00	0.00	--	0.00	0.00

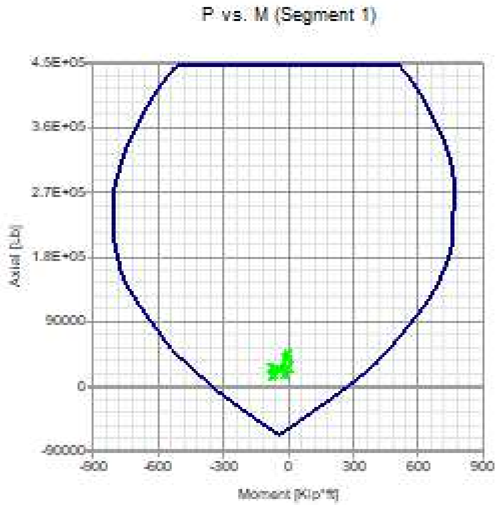
Results: Combined axial flexure

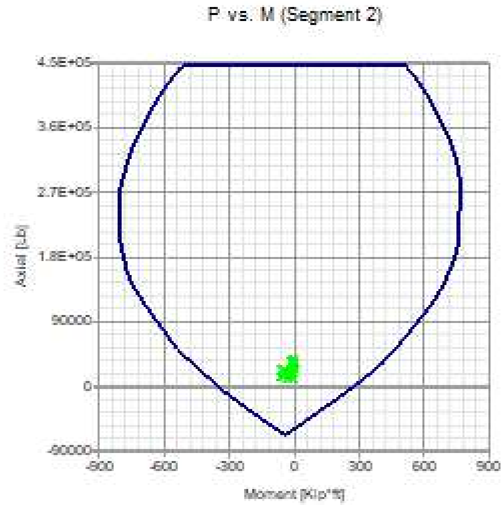
Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D18(Bottom)	16035.54	-74.21	407.20	0.18
2	D20(Bottom)	14015.32	-60.16	399.60	0.15
3	D18(Bottom)	10200.80	-31.69	385.37	0.08
4	D18(Bottom)	6416.44	-12.54	371.31	0.03
5	D20(Bottom)	2676.08	-2.46	342.28	0.01

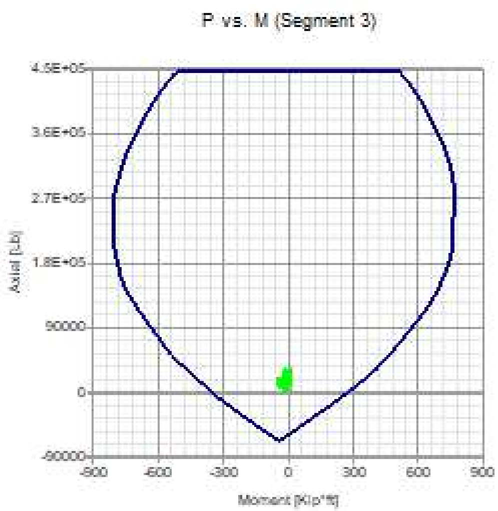
Results: Flexural reinforcement area

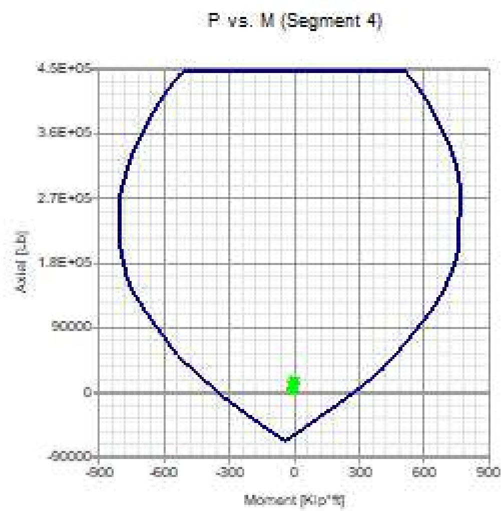
Segment	Condition	Pu [Lb]	As [in2]	Asmax [in2]	Ratio
1	D20(Bottom)	0.00	1.24	15.15	0.08
2	D20(Bottom)	0.00	1.24	15.15	0.08
3	D20(Bottom)	0.00	1.24	15.15	0.08
4	D17(Bottom)	0.00	1.24	15.15	0.08
5	D1(Top)	0.00	1.24	25.64	0.00

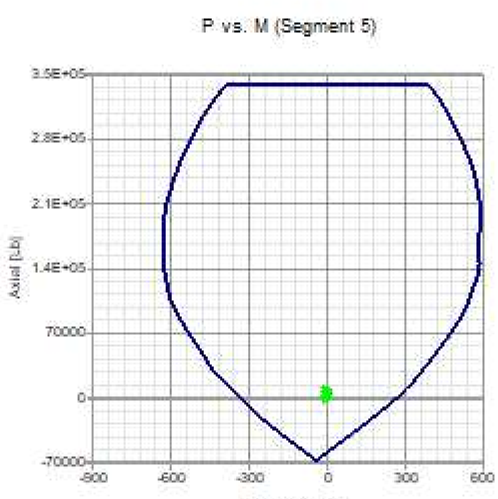
Interaction diagrams, P vs. M:


P vs. M (Segment 1)
Axial [Lb]
Moment [Kip*ft]
4.5E+05
3.6E+05
2.7E+05
1.8E+05
90000
0
-90000
-900
-600
-300
0
300
600
900


P vs. M (Segment 2)
Axial [Lb]
Moment [Kip*ft]
4.5E+05
3.6E+05
2.7E+05
1.8E+05
90000
0
-90000
-900
-600
-300
0
300
600
900


P vs. M (Segment 3)
Axial [Lb]
Moment [Kip*ft]
4.5E+05
3.6E+05
2.7E+05
1.8E+05
90000
0
-90000
-900
-600
-300
0
300
600
900


P vs. M (Segment 4)
Axial [Lb]
Moment [Kip*ft]
4.5E+05
3.6E+05
2.7E+05
1.8E+05
90000
0
-90000
-900
-600
-300
0
300
600
900


P vs. M (Segment 5)
Axial [Lb]
Moment [Kip*ft]
3.5E+05
2.8E+05
2.1E+05
1.4E+05
70000
0
-70000
-900
-600
-300
0
300
600

Results: Axial compression

Segment	Condition	Pu [Lb]	ϕPn [Lb]	Ratio
1	D2(Bottom)	45280.21	437280.20	0.10
2	D4(Bottom)	38233.33	401531.10	0.10
3	D2(Bottom)	28152.47	402092.00	0.07
4	D2(Bottom)	18126.55	402092.00	0.05
5	D2(Bottom)	8176.97	297501.90	0.03

Results: Shear

Segment	Condition	Vu [Lb]	ϕVn [Lb]	Ratio
1	D18(Bottom)	3856.82	28410.33	0.14
2	D20(Bottom)	3445.76	28107.29	0.12
3	D18(Bottom)	2485.93	27535.12	0.09
4	D18(Bottom)	1534.35	29515.65	0.05
5	D18(Bottom)	590.99	28673.27	0.02

Notes:

* Pu = Factored axial load
* Pn = Nominal compression strength
* δ = Moment magnification factor
* Mu = Factored total flexural moment
* Mua = Factored flexural moment from analysis
* Mn = Nominal moment strength
* Mcr = Nominal cracking moment
* ft = Stress due to flexural tension
* fc = Stress due to flexural compression
* Fn = Nominal stress
* Vu = Factored shear force
* Vn = Nominal shear strength
* Vf = Nominal shear friction strength
* δs = Calculated deflection
* δmax = Maximum allowable deflection
* ld = Embedment length
* Ag = Gross cross sectional area of a member
* As = Effective cross sectional area of reinforcement
* c = Distance from the fiber of maximum compressive strain to the neutral axis
* d = Distance from the extreme compression fiber to centroid of tension reinforcement

Current Date: 8/1/2023 12:02 PM
Units system: English
File name: G:\2023\23-283 Wood Spring Suites - Lee's Summit, MO\Calculations\G - CMU\Trash Enclosure Wall.msw

Design Results

Masonry wall

GENERAL INFORMATION:

**Global status : ** OK

Design code	:	TMS 402-16 SD
Geometry:		
Total height	:	96.00 [in]
Total length	:	168.00 [in]
Base support type	:	Continuous
Wall bottom restraint	:	Fixed
Column bottom restraint	:	Fixed
Rigidity elements	:	Flanges
Materials:		
Material	:	CMU 1.5-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Full grouting
Masonry compression strength (F`m)	:	216000 [Lb/ft2]
Steel tension strength (fy)	:	8.64E06 [Lb/ft2]
Steel allowable tension strength (Fs)	:	3.456E06 [Lb/ft2]
Steel elasticity modulus (Es)	:	4.176E09 [Lb/ft2]
Masonry elasticity modulus (Em)	:	1.944E08 [Lb/ft2]
Masonry unit weight	:	135 [Lb/ft3]

Number of stories: **1**

Story	Story height [in]	Wall thickness [in]	Effective unit weight [Lb/ft3]
1	96.00	7.63	135.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
WL	No	WIND	Wind Load
SM1	Yes		DL
DM1	Yes		DL
D1	Yes		DL
D2	Yes		DL+0.6WL
D3	Yes		0.6DL+0.6WL

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [in]
1	WL	Horizontal	3000.00	0.00	0.00

Distributed loads:

Consider self weight : DL

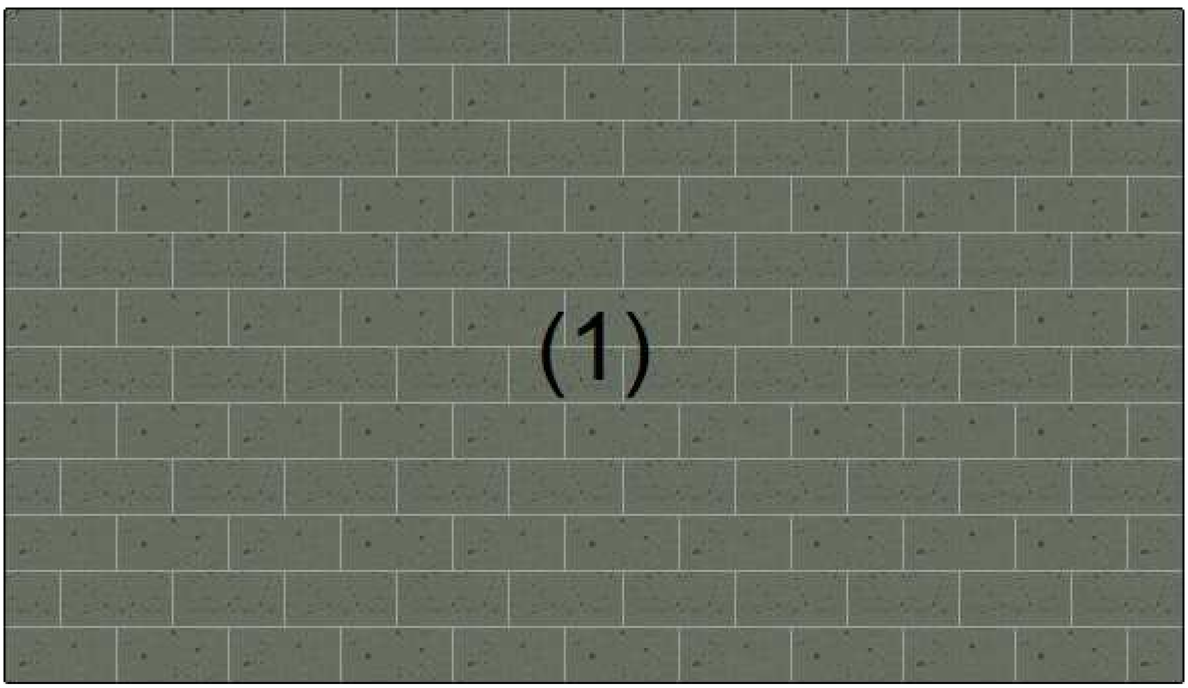
Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [in]
1	DL	Vertical	13.40	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	WL	0.02
Parapet	WL	0.02

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [in]	Y Coordinate [in]	Width [in]	Height [in]
1	0.00	0.00	168.00	96.00

Vertical reinforcement:

Segment	Bars	Spacing [in]	Ld [in]
1	4-#5	48.00	39.33

Results: Combined axial flexure

Segment	Condition	Pu [Lb]	Mua [Kip*ft]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D3(Bottom)	7575.26	2.12	2.12	19.98	0.11

Results: Flexural reinforcement area

Segment	Condition	Pu [Lb]	As [in2]	Asmax [in2]	Ratio
1	DM1(Max)	0.00	1.09	4.31	0.25

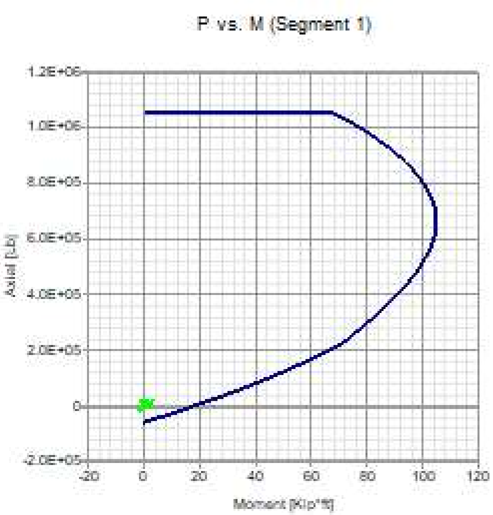
Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]	Mcr [Kip*ft]
1	D3(Bottom)	0.48	3.81	1.64

Inertias

Segment	Condition	Ig [in4]	Icr [in4]
1	D3(Bottom)	444.19	22.36

Interaction diagrams, P vs. M:



Results: Axial compression

Segment	Condition	Pu [Lb]	ϕPn [Lb]	Ratio
1	D2(Bottom)	11309.01	999281.00	0.01

Axial stress

Segment	Condition	Pu [Lb]	Pu/Ag [Lb/ft2]	Fn [Lb/ft2]	Ratio
1	D2(Max)	11309.01	1270.44	43200.00	0.03

Results: Shear

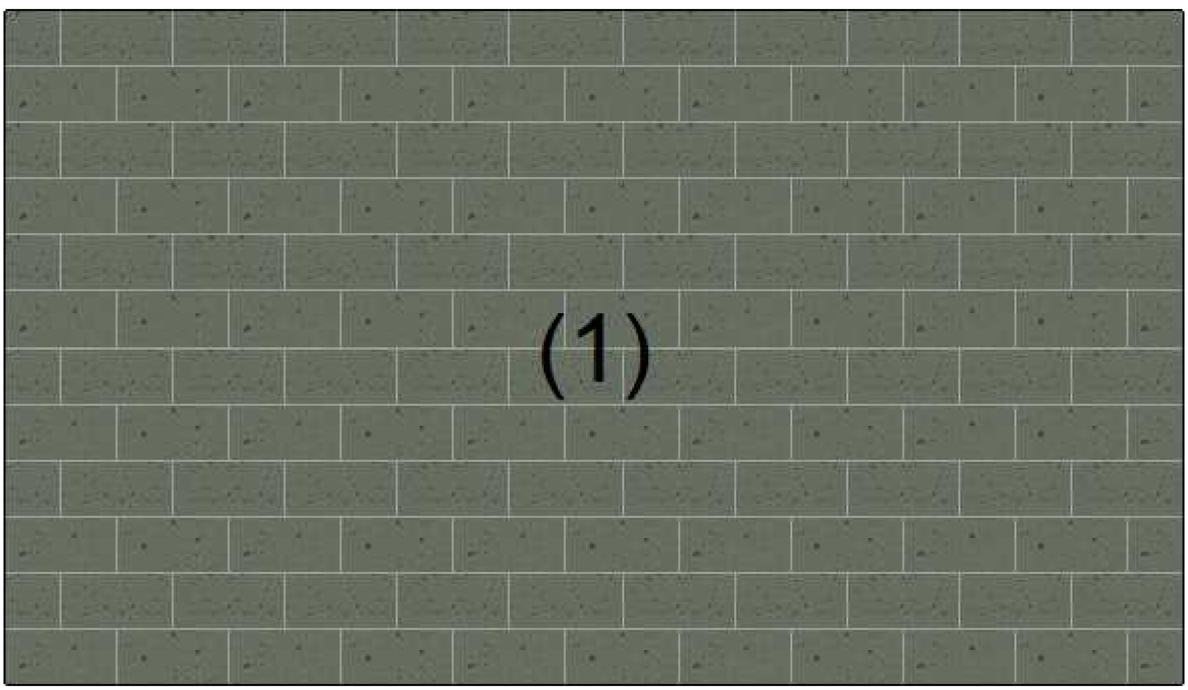
Segment	Condition	Vu [Lb]	ϕVn [Lb]	Ratio
1	D3(Bottom)	70.95	3299.71	0.02

Deflection

Segment	Condition	δs [in]	δmax [in]	δs/δmax
1	SM1(Max)	0.00	0.67	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [in]	Y Coordinate [in]	Width [in]	Height [in]
1	0.00	0.00	168.00	96.00

Reinforcement:

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	4-#5	48.00	0.00	--	0.00	0.00

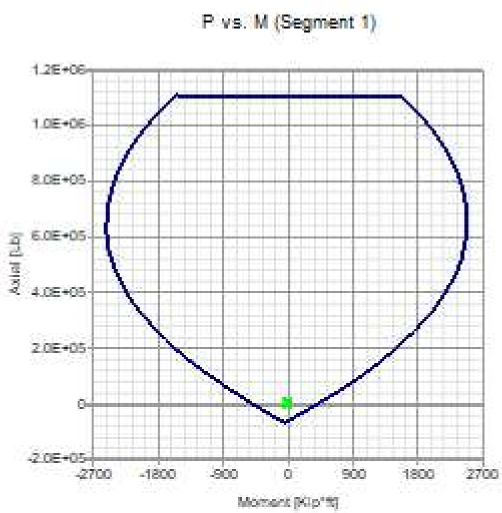
Results: Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D3(Bottom)	7578.54	-11.17	535.92	0.02

Results: Flexural reinforcement area

Segment	Condition	Pu [Lb]	As [in2]	Asmax [in2]	Ratio
1	DM1(Top)	0.00	1.24	88.30	0.00

Interaction diagrams, P vs. M:



Results: Axial compression

Segment	Condition	Pu [Lb]	ϕPn [Lb]	Ratio
1	D2(Bottom)	11312.57	999201.10	0.01

Results: Shear

Segment	Condition	Vu [Lb]	ϕVn [Lb]	Ratio
1	D3(Bottom)	1805.97	123042.90	0.01

Notes:

* Pu = Factored axial load
* Pn = Nominal compression strength
* δ = Moment magnification factor
* Mu = Factored total flexural moment
* Mua = Factored flexural moment from analysis
* Mn = Nominal moment strength
* Mcr = Nominal cracking moment
* ft = Stress due to flexural tension
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* δs = Calculated deflection
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* d = Distance from the extreme compression fiber to centroid of tension reinforcement