


ASTRA Engineering 7887 E. Belleview Ave Englewood, CO 80111	JOB NO. 20-115	DATE 07-5-21	DESIGNED BY: CHECKED BY:	PAGE 1.0
CLIENT: ROGUE ARCHITECTURE _____ 1660 Lincoln St. #100 _____ Denver, CO 80264 _____				
PROJECT: PARKER MIXED-USE RETAIL _____ 12225 Pardee Street _____ Parker, CO _____				

DESIGN CRITERIA AND MATERIALS						
DESIGN DATA				CODE DATA		
CONCRETE:	<u>Type of Concrete</u>		<u>F'c (psi)</u>	BUILDING CODE: 2018 IBC w/City Code Amendments SEISMIC CRITERIA: Design Category: "B" Site Classification: "D" Ss: 0.203g S1: 0.059g Use Group: 1 WIND CRITERIA: Wind Speed = 142 mph (ult.) I = 1.0 Exposure = "C" DESIGN DEAD LOADS: Wood Deck – 2.0 psf Wood Joist/Trusses – 3.5 psf MEP – 6.5 psf Insulation – 4.5 psf Finish Materials – 2.0 Misc. – 1.5 TOTAL ROOF (DL): 20 PSF ----- Subflooring – 2.5 psf MEP – 8.0 psf Insulation – 2.0 psf I-joist – 3.5 psf Finish Materials – 2.5 psf Misc. – 1.5 TOTAL FLOOR (DL): 20 PSF DESIGN LIVE LOADS: Ground. Snow – 35 psf Corridor Min. Live Load – 100 psf Floor Live Load – 40 psf		
	Slabs on Grade		3000			
	Footings		3000			
	All Others		4000			
STEEL REINFORCING:	ASTM A 615, Grade 60 for all Reinforcing ASTM A 706, Grade 60 for all Welded Bars ASTM A185, Wire per ASTM A82 all Welded Wire Fabric					
SOIL:	Soil Report By: Kumar & Associates Soil Report No: 18-8-180 Bearing Pressure: 2,500 psf					
STRUCTURAL STEEL:	Wide Flange Members - ASTM A992 (Fy = 50 ksi) Channels, Plates & Angles – ASTM A36 (Fy = 36 ksi) Pipe Steel – ASTM A53, Gr.-B (Fy = 35 ksi) Tube Steel – ASTM A500, Gr-B (Fy = 46 ksi)					
MASONRY:	No structural masonry units					
MORTAR:	No structural mortar					
MASONRY GROUT:	No structural masonry grout					
WOOD:	<u>Wood Type</u>	<u>Fb (psi)</u>	<u>Fv (psi)</u>	<u>E (ksi)</u>	<u>Fc (psi)</u>	
	Joist, Ledgers, Plates 4x or smaller	1000	150	1600	1300	



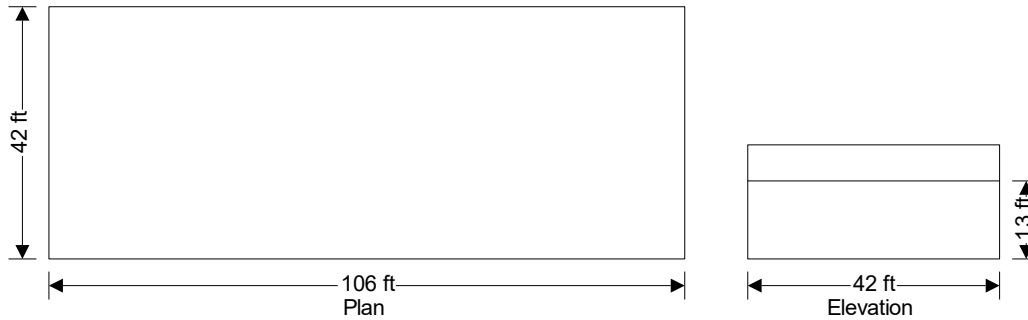
Project				Job Ref.	
Section Wind Force				Sheet no./rev. 1	
Calc. by N	Date 7/27/2020	Chk'd by	Date	App'd by	Date

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.05



Building data

Type of roof	Flat
Length of building	b = 106.00 ft
Width of building	d = 42.00 ft
Height to eaves	H = 13.00 ft
Height of parapet	h _p = 6.00 ft
Mean height	h = 13.00 ft

General wind load requirements

Basic wind speed	V = 142.0 mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K _d = 0.85
Ground elevation above sea level	Z _{gl} = 0 ft
Ground elevation factor	K _e = exp(-0.0000362 × Z _{gl} /1ft) = 1.00
Exposure category (cl 26.7.3)	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
Gust effect factor	G _f = 0.85
Minimum design wind loading (cl.27.4.7)	p _{min_r} = 8 lb/ft ²

Topography

Topography factor not significant	K _{zt} = 1.0
Velocity pressure equation	q = 0.00256 × K _z × K _{zt} × K _d × V ² × 1psf/mph ²

Velocity pressures table

z (ft)	K _z (Table 26.10-1)	q _z (psf)
13.00	0.85	37.30
19.00	0.89	39.05

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = 37.30 psf



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Parapet pressures and forces

Velocity pressure at top of parapet	$q_p = 39.05$ psf
Combined net pressure coefficient, leeward	$GC_{pnl} = -1.0$
Combined net parapet pressure, leeward	$p_{pl} = q_p \times GC_{pnl} = -39.05$ psf
Combined net pressure coefficient, windward	$GC_{pnw} = 1.5$
Combined net parapet pressure, windward	$p_{pw} = q_p \times GC_{pnw} = 58.58$ psf
Wind direction 0 deg:	
Leeward parapet force	$F_{w,wpL_0} = p_{pl} \times h_p \times b = -24.8$ kips
Windward parapet force	$F_{w,wpW_0} = p_{pw} \times h_p \times b = 37.3$ kips
Wind direction 90 deg:	
Leeward parapet force	$F_{w,wpL_90} = p_{pl} \times h_p \times d = -9.8$ kips
Windward parapet force	$F_{w,wpW_90} = p_{pw} \times h_p \times d = 14.8$ kips

Pressures and forces

Net pressure	$p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$
Net force	$F_w = p \times A_{ref}$

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A (-ve)	13.00	-0.90	37.30	-35.24	689.00	-24.28
B (-ve)	13.00	-0.90	37.30	-35.24	689.00	-24.28
C (-ve)	13.00	-0.50	37.30	-22.56	1378.00	-31.09
D (-ve)	13.00	-0.30	37.30	-16.22	1696.00	-27.52

Total vertical net force	$F_{w,v} = -107.17$ kips
Total horizontal net force	$F_{w,h} = 0.00$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A	13.00	0.80	37.30	18.65	1378.00	25.70
B	13.00	-0.50	37.30	-22.56	1378.00	-31.09
C	13.00	-0.70	37.30	-28.90	546.00	-15.78
D	13.00	-0.70	37.30	-28.90	546.00	-15.78

Overall loading

Projected vertical plan area of wall	$A_{vert,w_0} = b \times (H + h_p) = 2014.00$ ft ²
Projected vertical area of roof	$A_{vert,r_0} = 0.00$ ft ²
Minimum overall horizontal loading	$F_{w,total_min} = p_{min,w} \times A_{vert,w_0} + p_{min,r} \times A_{vert,r_0} = 32.22$ kips
Leeward net force	$F_l = F_{w,wB} + F_{w,wpL_0} = -55.9$ kips
Windward net force	$F_w = F_{w,wA} + F_{w,wpW_0} = 63.0$ kips
Overall horizontal loading	$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 118.9$ kips



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Roof load case 2 - Wind 0, GC_{pi} -0.18, -0C_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c _{pe}	Peak velocity pressure q _p (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A (+ve)	13.00	-0.18	37.30	1.01	689.00	0.69
B (+ve)	13.00	-0.18	37.30	1.01	689.00	0.69
C (+ve)	13.00	-0.18	37.30	1.01	1378.00	1.39
D (+ve)	13.00	-0.18	37.30	1.01	1696.00	1.71

Total vertical net force $F_{w,v} = 4.48$ kips

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

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0C_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c _{pe}	Peak velocity pressure q _p (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A	13.00	0.80	37.30	32.07	1378.00	44.20
B	13.00	-0.50	37.30	-9.14	1378.00	-12.59
C	13.00	-0.70	37.30	-15.48	546.00	-8.45
D	13.00	-0.70	37.30	-15.48	546.00	-8.45

Overall loading

Projected vertical plan area of wall $A_{vert_w_0} = b \times (H + h_p) = 2014.00$ ft²

Projected vertical area of roof $A_{vert_r_0} = 0.00$ ft²

Minimum overall horizontal loading $F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = 32.22$ kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl_0} = -37.4$ kips

Windward net force $F_w = F_{w,wA} + F_{w,wpw_0} = 81.5$ kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 118.9$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c _{pe}	Peak velocity pressure q _p (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A (-ve)	13.00	-0.90	37.30	-35.24	273.00	-9.62
B (-ve)	13.00	-0.90	37.30	-35.24	273.00	-9.62
C (-ve)	13.00	-0.50	37.30	-22.56	546.00	-12.32
D (-ve)	13.00	-0.30	37.30	-16.22	3360.00	-54.51

Total vertical net force $F_{w,v} = -86.07$ kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c _{pe}	Peak velocity pressure q _p (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A	13.00	0.80	37.30	18.65	546.00	10.18
B	13.00	-0.27	37.30	-15.39	546.00	-8.40



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Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
C	13.00	-0.70	37.30	-28.90	1378.00	-39.83
D	13.00	-0.70	37.30	-28.90	1378.00	-39.83

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_90} = d \times (H + h_p) = \mathbf{798.00 \text{ ft}^2}$$

Projected vertical area of roof

$$A_{vert_r_90} = \mathbf{0.00 \text{ ft}^2}$$

Minimum overall horizontal loading

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = \mathbf{12.77 \text{ kips}}$$

Leeward net force

$$F_l = F_{w,wB} + F_{w,wpl_90} = \mathbf{-18.2 \text{ kips}}$$

Windward net force

$$F_w = F_{w,wA} + F_{w,wpw_90} = \mathbf{24.9 \text{ kips}}$$

Overall horizontal loading

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = \mathbf{43.2 \text{ kips}}$$

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A (+ve)	13.00	-0.18	37.30	1.01	273.00	0.27
B (+ve)	13.00	-0.18	37.30	1.01	273.00	0.27
C (+ve)	13.00	-0.18	37.30	1.01	546.00	0.55
D (+ve)	13.00	-0.18	37.30	1.01	3360.00	3.38

Total vertical net force

$$F_{w,v} = \mathbf{4.48 \text{ kips}}$$

Total horizontal net force

$$F_{w,h} = \mathbf{0.00 \text{ kips}}$$

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A	13.00	0.80	37.30	32.07	546.00	17.51
B	13.00	-0.27	37.30	-1.97	546.00	-1.07
C	13.00	-0.70	37.30	-15.48	1378.00	-21.33
D	13.00	-0.70	37.30	-15.48	1378.00	-21.33

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_90} = d \times (H + h_p) = \mathbf{798.00 \text{ ft}^2}$$

Projected vertical area of roof

$$A_{vert_r_90} = \mathbf{0.00 \text{ ft}^2}$$

Minimum overall horizontal loading

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = \mathbf{12.77 \text{ kips}}$$

Leeward net force

$$F_l = F_{w,wB} + F_{w,wpl_90} = \mathbf{-10.9 \text{ kips}}$$

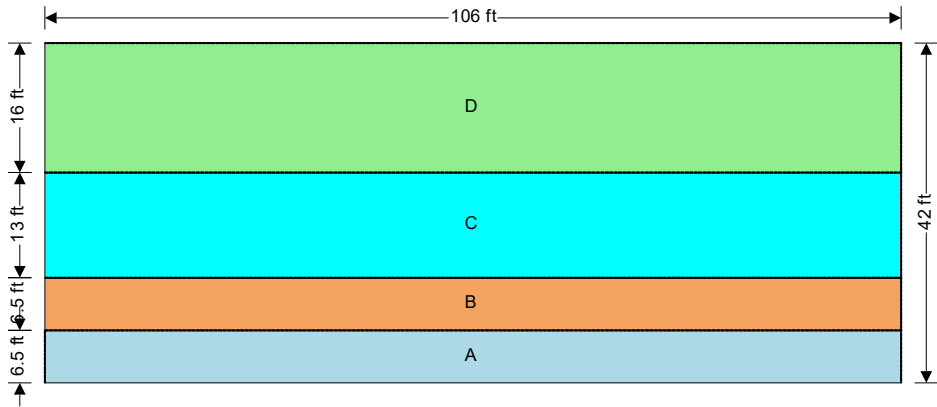
Windward net force

$$F_w = F_{w,wA} + F_{w,wpw_90} = \mathbf{32.3 \text{ kips}}$$

Overall horizontal loading

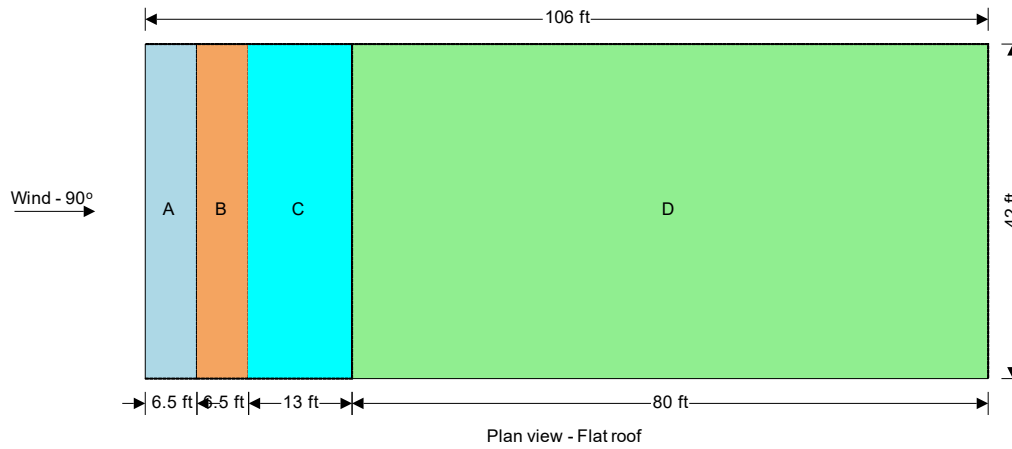
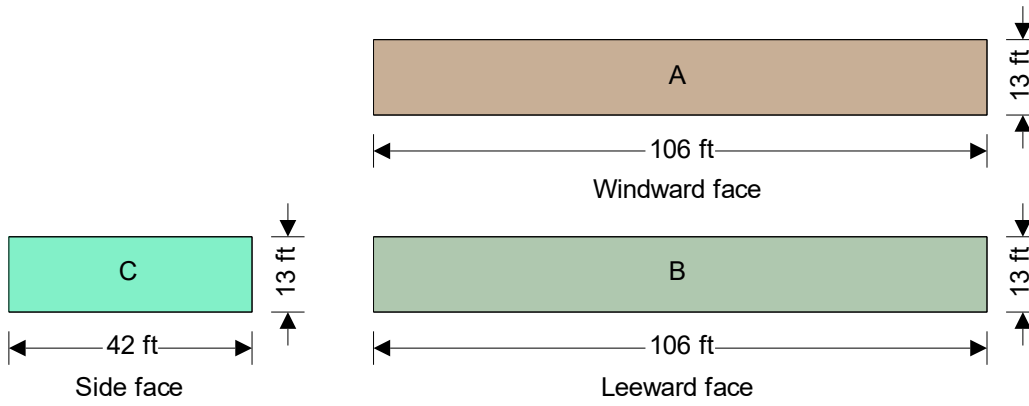
$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = \mathbf{43.2 \text{ kips}}$$

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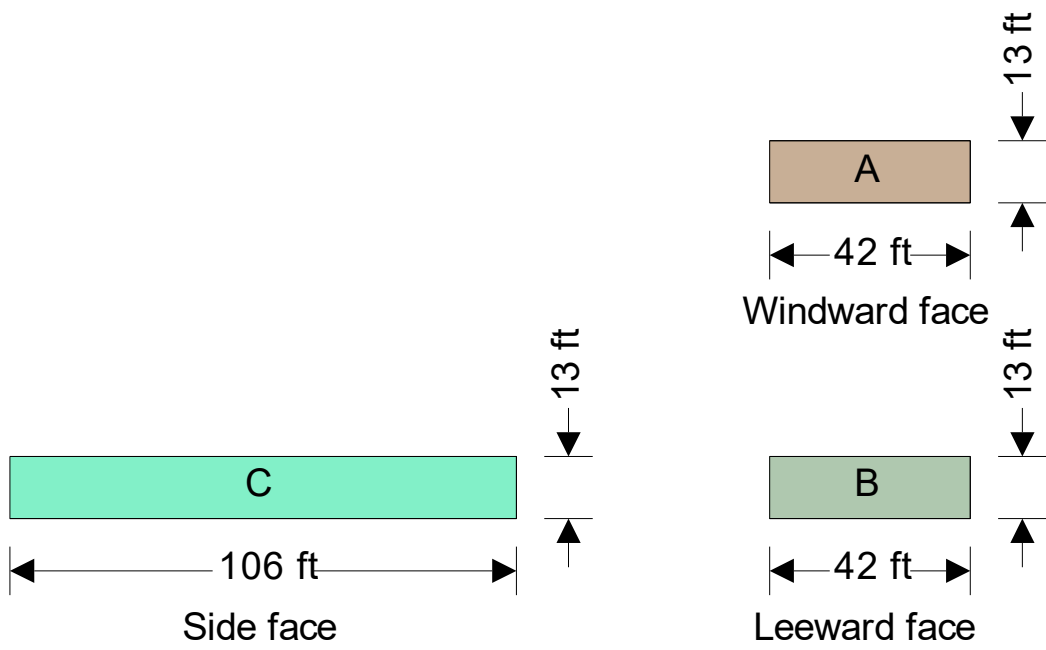


Wind - 0°
↑

Plan view - Flat roof



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Section Seismic Force		Sheet no./rev. 1	
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SEISMIC FORCES (ASCE 7-16)

Tedds calculation version 3.1.00

Site parameters

Site class	D
Mapped acceleration parameters (Section 11.4.2)	
at short period	$S_S = 0.203$
at 1 sec period	$S_1 = 0.059$
Site coefficient at 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)	$S_{MS} = F_a \times S_S = 0.325$
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1 = 0.142$

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS} = 0.217$
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1} = 0.094$

Seismic design category

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

B

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 $h_n = 15$ 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) $T_a = C_t \times (h_n)^x \times 1 \text{ sec} / (1 \text{ ft})^x = 0.152$ sec

Building fundamental period (Sect 12.8.2) $T = T_a = 0.152$ sec

Long-period transition period $T_L = 12$ 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 12.8-3) $C_{s_calc} = S_{DS} / (R / I_e) = 0.0333$

Maximum (Eq 12.8-3) $C_{s_max} = S_{D1} / ((T / 1 \text{ sec}) \times (R / I_e)) = 0.0953$

Minimum (Eq 9.5.5.2.1-3) $C_{s_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) = 0.0100$



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Seismic response coefficient $C_s = 0.0333$

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure $W = 267.0$ kips

Seismic response coefficient $C_s = 0.0333$

Seismic base shear (Eq 12.8-1) $V = C_s \times W = 8.9$ kips



Project Snow Drift Load				Job Ref.	
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SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details

Roof type Flat
Width of roof $b = 106.00$ ft

Ground snow load

Ground snow load (Figure 7.2-1) $p_g = 30.00$ lb/ft²
 Density of snow $\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3) = 17.90$ lb/ft³
 Terrain type Sect. 26.7 C
 Exposure condition (Table 7.3-1) Fully exposed
 Exposure factor (Table 7.3-1) $C_e = 0.90$
 Thermal condition (Table 7.3-2) Structures kept just above freezing
 Thermal factor (Table 7.3-2) $C_t = 1.10$
 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) $p_{f_min} = I_s \times 20$ lb/ft² = 20.00 lb/ft²
 Flat roof snow load (Sect 7.3) $p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 20.79$ lb/ft²

Left parapet

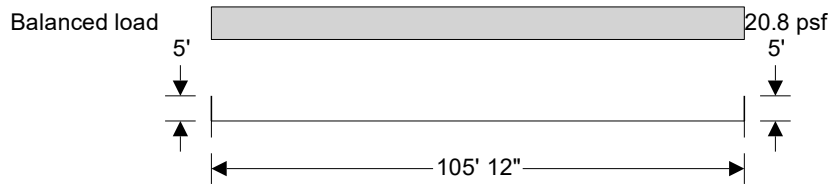
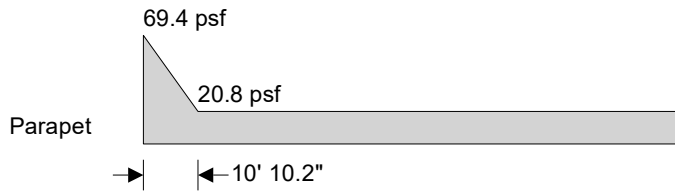
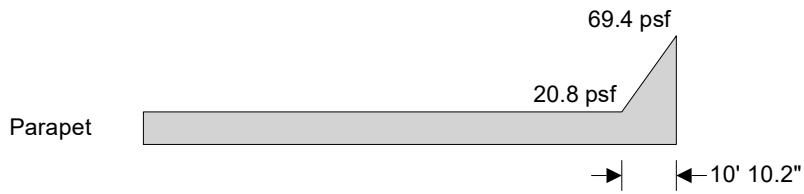
Balanced snow load height $h_b = p_f / \gamma = 1.16$ ft
 Height of left parapet $h_{pptL} = 5.00$ ft
 Height from balance load to top of left parapet $h_{c_pptL} = h_{pptL} - h_b = 3.84$ ft
 Length of roof - left parapet $l_{u_pptL} = b = 106.00$ ft
 Drift height windward drift - left parapet $h_{d_pptL} = \sqrt[4]{(I_s) \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})} = 2.71$ ft
 Drift height - left parapet $h_{d_pptL} = \min(h_{d_pptL}, h_{pptL} - h_b) = 2.71$ ft
 Drift width $W_{d_pptL} = \min(4 \times h_{d_pptL}, 8 \times (h_{pptL} - h_b), b) = 10.85$ ft
 Drift surcharge load - left parapet $p_{d_pptL} = h_{d_pptL} \times \gamma = 48.57$ lb/ft²

Right parapet

Height of right parapet $h_{pptR} = 5.00$ ft
 Height from balance load to top of right parapet $h_{c_pptR} = h_{pptR} - h_b = 3.84$ ft
 Length of roof - right parapet $l_{u_pptR} = b = 106.00$ ft
 Drift height windward drift - right parapet $h_{d_pptR} = \sqrt[4]{(I_s) \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})} = 2.71$ ft
 Drift height - right parapet $h_{d_pptR} = \min(h_{d_pptR}, h_{pptR} - h_b) = 2.71$ ft
 Drift width $W_{d_pptR} = \min(4 \times h_{d_pptR}, 8 \times (h_{pptR} - h_b), b) = 10.85$ ft
 Drift surcharge load - right parapet $p_{d_pptR} = h_{d_pptR} \times \gamma = 48.57$ lb/ft²



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Roof elevation

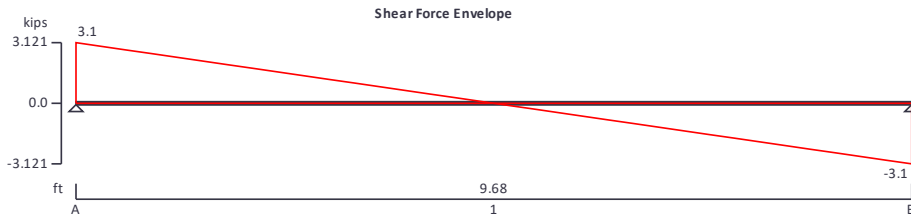
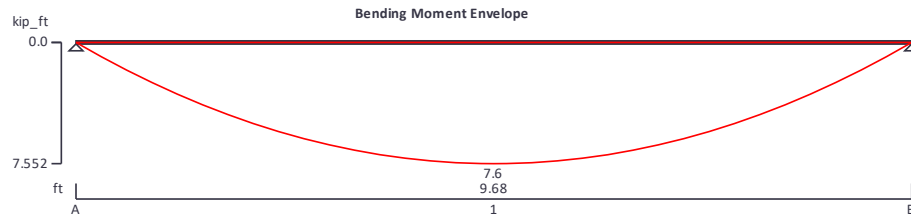
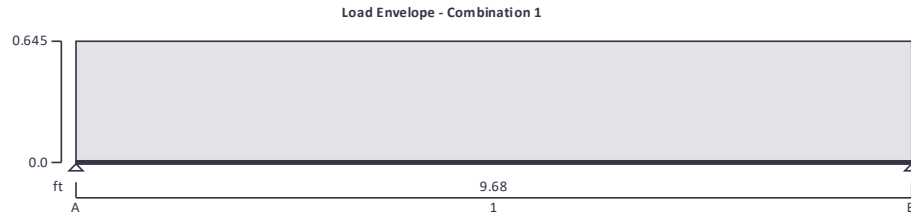


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STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

Tedds calculation version 1.7.08



Applied loading

Beam loads

- Dead self weight of beam × 1
- Dead full UDL 210 lb/ft
- Snow full UDL 420 lb/ft

Load combinations

Load combination 1

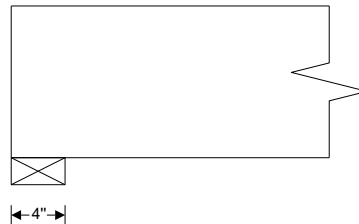
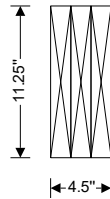
Support A	Dead × 1.00
	Live × 1.00
	Snow × 1.00
Span 1	Dead × 1.00
	Live × 1.00
	Snow × 1.00
Support B	Dead × 1.00
	Live × 1.00

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Snow \times 1.00

Analysis results

Maximum moment	$M_{max} = 7552 \text{ lb_ft}$	$M_{min} = 0 \text{ lb_ft}$
Design moment	$M = \max(\text{abs}(M_{max}), \text{abs}(M_{min})) = 7552 \text{ lb_ft}$	
Maximum shear	$F_{max} = 3121 \text{ lb}$	$F_{min} = -3121 \text{ lb}$
Design shear	$F = \max(\text{abs}(F_{max}), \text{abs}(F_{min})) = 3121 \text{ lb}$	
Total load on member	$W_{tot} = 6241 \text{ lb}$	
Reaction at support A	$R_{A_max} = 3121 \text{ lb}$	$R_{A_min} = 3121 \text{ lb}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 1088 \text{ lb}$	
Unfactored snow load reaction at support A	$R_{A_Snow} = 2033 \text{ lb}$	
Reaction at support B	$R_{B_max} = 3121 \text{ lb}$	$R_{B_min} = 3121 \text{ lb}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 1088 \text{ lb}$	
Unfactored snow load reaction at support B	$R_{B_Snow} = 2033 \text{ lb}$	



Sawn lumber section details

Nominal breadth of sections	$b_{nom} = 2 \text{ in}$
Dressed breadth of sections	$b = 1.5 \text{ in}$
Nominal depth of sections	$d_{nom} = 12 \text{ in}$
Dressed depth of sections	$d = 11.25 \text{ in}$
Number of sections in member	$N = 3$
Overall breadth of member	$b_b = N \times b = 4.5 \text{ in}$
Species, grade and size classification	Hem-Fir, No.2 grade, 2" & wider
Bending parallel to grain	$F_b = 850 \text{ lb/in}^2$
Tension parallel to grain	$F_t = 525 \text{ lb/in}^2$
Compression parallel to grain	$F_c = 1300 \text{ lb/in}^2$
Compression perpendicular to grain	$F_{c_perp} = 405 \text{ lb/in}^2$
Shear parallel to grain	$F_v = 150 \text{ lb/in}^2$
Modulus of elasticity	$E = 1300000 \text{ lb/in}^2$
Modulus of elasticity, stability calculations	$E_{min} = 470000 \text{ lb/in}^2$
Mean shear modulus	$G_{def} = E / 16 = 81250 \text{ lb/in}^2$

Member details

Service condition	Dry
Length of span	$L_{s1} = 9.68 \text{ ft}$
Length of bearing	$L_b = 4 \text{ in}$
Load duration	Ten years

Section properties

Cross sectional area of member	$A = N \times b \times d = 50.62 \text{ in}^2$
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Section modulus $S_x = N \times b \times d^2 / 6 = 94.92 \text{ in}^3$
 $S_y = d \times (N \times b)^2 / 6 = 37.97 \text{ in}^3$

Second moment of area $I_x = N \times b \times d^3 / 12 = 533.94 \text{ in}^4$
 $I_y = d \times (N \times b)^3 / 12 = 85.43 \text{ in}^4$

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

$$d_{nom} / (N \times b_{nom}) = 2.00$$

- 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 $F_{c_perp}' = F_{c_perp} \times C_t \times C_{ic_perp} \times C_b = 405 \text{ lb/in}^2$

Applied compression stress perpendicular to grain $f_{c_perp} = R_{A_max} / (N \times b \times L_b) = 173 \text{ lb/in}^2$

$$f_{c_perp} / F_{c_perp}' = 0.428$$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress

$$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = 978 \text{ lb/in}^2$$

Actual bending stress

$$f_b = M / S_x = 955 \text{ lb/in}^2$$

$$f_b / F_b' = 0.977$$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress

$$F_v' = F_v \times C_D \times C_t \times C_i = 150 \text{ lb/in}^2$$

Actual shear stress - eq.3.4-2

$$f_v = 3 \times F / (2 \times A) = 92 \text{ lb/in}^2$$

$$f_v / F_v' = 0.616$$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection

$$E' = E \times C_{ME} \times C_t \times C_{IE} = 1300000 \text{ lb/in}^2$$

Design deflection

$$\delta_{adm} = 0.003 \times L_{s1} = 0.348 \text{ in}$$

Total deflection

$$\delta_{b_s1} = 0.184 \text{ in}$$

$$\delta_{b_s1} / \delta_{adm} = 0.527$$

PASS - Total deflection is less than design deflection



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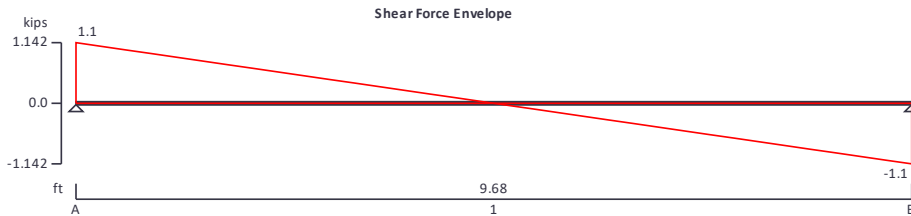
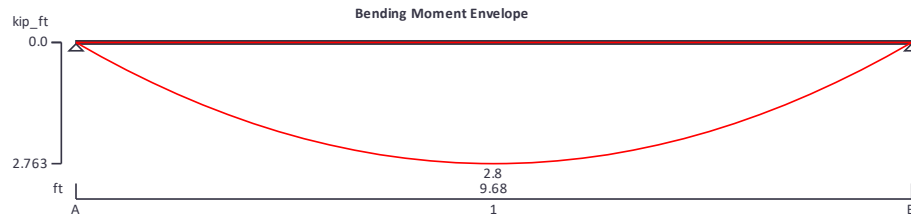
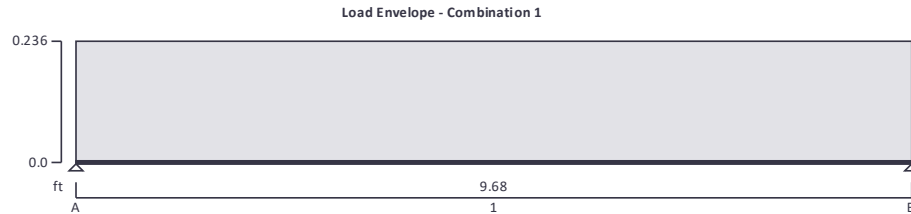


Project				Job Ref.	
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STRUCTURAL WOOD MEMBER ANALYSIS & DESIGN (NDS)

In accordance with the ANSI/AF&PA NDS-2015 using the ASD method

Tedds calculation version 1.7.08



Applied loading

Beam loads

- Dead self weight of beam × 1
- Dead full UDL 70 lb/ft
- Snow full UDL 150 lb/ft

Load combinations

Load combination 1

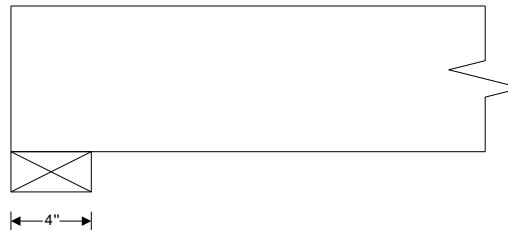
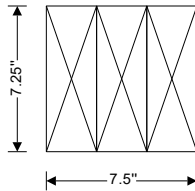
Support A	Dead × 1.00
	Live × 1.00
	Snow × 1.00
Span 1	Dead × 1.00
	Live × 1.00
	Snow × 1.00
Support B	Dead × 1.00
	Live × 1.00

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Snow \times 1.00

Analysis results

Maximum moment	$M_{max} = 2763 \text{ lb_ft}$	$M_{min} = 0 \text{ lb_ft}$
Design moment	$M = \max(\text{abs}(M_{max}), \text{abs}(M_{min})) = 2763 \text{ lb_ft}$	
Maximum shear	$F_{max} = 1142 \text{ lb}$	$F_{min} = -1142 \text{ lb}$
Design shear	$F = \max(\text{abs}(F_{max}), \text{abs}(F_{min})) = 1142 \text{ lb}$	
Total load on member	$W_{tot} = 2283 \text{ lb}$	
Reaction at support A	$R_{A_max} = 1142 \text{ lb}$	$R_{A_min} = 1142 \text{ lb}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 416 \text{ lb}$	
Unfactored snow load reaction at support A	$R_{A_Snow} = 726 \text{ lb}$	
Reaction at support B	$R_{B_max} = 1142 \text{ lb}$	$R_{B_min} = 1142 \text{ lb}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 416 \text{ lb}$	
Unfactored snow load reaction at support B	$R_{B_Snow} = 726 \text{ lb}$	



Sawn lumber section details

Nominal breadth of sections	$b_{nom} = 3 \text{ in}$
Dressed breadth of sections	$b = 2.5 \text{ in}$
Nominal depth of sections	$d_{nom} = 8 \text{ in}$
Dressed depth of sections	$d = 7.25 \text{ in}$
Number of sections in member	$N = 3$
Overall breadth of member	$b_b = N \times b = 7.5 \text{ in}$
Species, grade and size classification	Hem-Fir, No.2 grade, 2" & wider
Bending parallel to grain	$F_b = 850 \text{ lb/in}^2$
Tension parallel to grain	$F_t = 525 \text{ lb/in}^2$
Compression parallel to grain	$F_c = 1300 \text{ lb/in}^2$
Compression perpendicular to grain	$F_{c_perp} = 405 \text{ lb/in}^2$
Shear parallel to grain	$F_v = 150 \text{ lb/in}^2$
Modulus of elasticity	$E = 1300000 \text{ lb/in}^2$
Modulus of elasticity, stability calculations	$E_{min} = 470000 \text{ lb/in}^2$
Mean shear modulus	$G_{def} = E / 16 = 81250 \text{ lb/in}^2$

Member details

Service condition	Dry
Length of span	$L_{s1} = 9.68 \text{ ft}$
Length of bearing	$L_b = 4 \text{ in}$
Load duration	Ten years

Section properties

Cross sectional area of member	$A = N \times b \times d = 54.37 \text{ in}^2$
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Section modulus $S_x = N \times b \times d^2 / 6 = \mathbf{65.70 \text{ in}^3}$
 $S_y = d \times (N \times b)^2 / 6 = \mathbf{67.97 \text{ in}^3}$

Second moment of area $I_x = N \times b \times d^3 / 12 = \mathbf{238.17 \text{ in}^4}$
 $I_y = d \times (N \times b)^3 / 12 = \mathbf{254.88 \text{ in}^4}$

Adjustment factors

Load duration factor - Table 2.3.2 $C_D = \mathbf{1.00}$
 Temperature factor - Table 2.3.3 $C_t = \mathbf{1.00}$
 Size factor for bending - Table 4A $C_{Fb} = \mathbf{1.20}$
 Size factor for tension - Table 4A $C_{Ft} = \mathbf{1.20}$
 Size factor for compression - Table 4A $C_{Fc} = \mathbf{1.05}$
 Flat use factor - Table 4A $C_{fu} = \mathbf{1.15}$
 Incising factor for modulus of elasticity - Table 4.3.8

$$C_{IE} = \mathbf{1.00}$$

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

$$C_i = \mathbf{1.00}$$

Incising factor for perpendicular compression - Table 4.3.8

$$C_{ic_perp} = \mathbf{1.00}$$

Repetitive member factor - cl.4.3.9

$$C_r = \mathbf{1.15}$$

Bearing area factor - cl.3.10.4

$$C_b = \mathbf{1.00}$$

Depth-to-breadth ratio

$$d_{nom} / (N \times b_{nom}) = \mathbf{0.89}$$

- Beam is fully restrained

Beam stability factor - cl.3.3.3

$$C_L = \mathbf{1.00}$$

Bearing perpendicular to grain - cl.3.10.2

Design compression perpendicular to grain $F_{c_perp}' = F_{c_perp} \times C_t \times C_{ic_perp} \times C_b = \mathbf{405 \text{ lb/in}^2}$

Applied compression stress perpendicular to grain $f_{c_perp} = R_{A_max} / (N \times b \times L_b) = \mathbf{38 \text{ lb/in}^2}$

$$f_{c_perp} / F_{c_perp}' = \mathbf{0.094}$$

PASS - Design compressive stress exceeds applied compressive stress at bearing

Strength in bending - cl.3.3.1

Design bending stress

$$F_b' = F_b \times C_D \times C_t \times C_L \times C_{Fb} \times C_i \times C_r = \mathbf{1173 \text{ lb/in}^2}$$

Actual bending stress

$$f_b = M / S_x = \mathbf{505 \text{ lb/in}^2}$$

$$f_b / F_b' = \mathbf{0.430}$$

PASS - Design bending stress exceeds actual bending stress

Strength in shear parallel to grain - cl.3.4.1

Design shear stress

$$F_v' = F_v \times C_D \times C_t \times C_i = \mathbf{150 \text{ lb/in}^2}$$

Actual shear stress - eq.3.4-2

$$f_v = 3 \times F / (2 \times A) = \mathbf{31 \text{ lb/in}^2}$$

$$f_v / F_v' = \mathbf{0.210}$$

PASS - Design shear stress exceeds actual shear stress

Deflection - cl.3.5.1

Modulus of elasticity for deflection

$$E' = E \times C_{ME} \times C_t \times C_{IE} = \mathbf{1300000 \text{ lb/in}^2}$$

Design deflection

$$\delta_{adm} = 0.003 \times L_{s1} = \mathbf{0.348 \text{ in}}$$

Total deflection

$$\delta_{b_s1} = \mathbf{0.150 \text{ in}}$$

$$\delta_{b_s1} / \delta_{adm} = \mathbf{0.432}$$

PASS - Total deflection is less than design deflection



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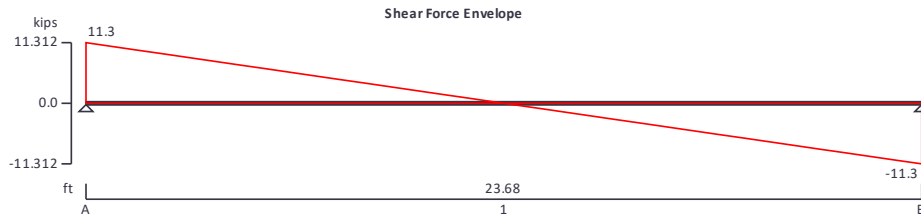
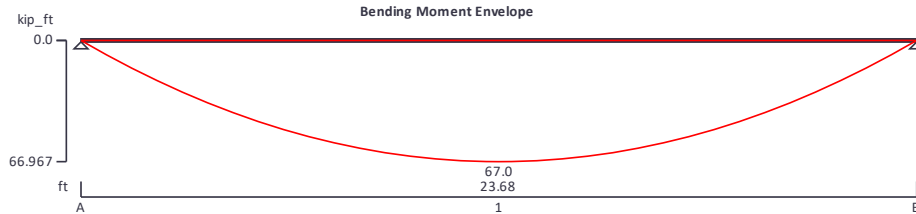
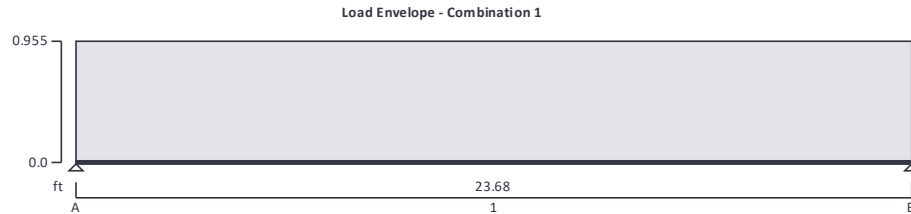


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Section H-3 type header				Sheet no./rev. 1	
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STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the LRFD method

Tedds calculation version 3.0.14



Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.21 kips/ft
	Snow full UDL 0.42 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Roof live × 1.60
		Snow × 1.60

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Support B

Dead × 1.20
Live × 1.60
Roof live × 1.60
Snow × 1.60
Dead × 1.20
Live × 1.60
Roof live × 1.60
Snow × 1.60

Analysis results

Maximum moment

$$M_{\max} = 67 \text{ kips_ft}$$

$$M_{\min} = 0 \text{ kips_ft}$$

Maximum shear

$$V_{\max} = 11.3 \text{ kips}$$

$$V_{\min} = -11.3 \text{ kips}$$

Deflection

$$\delta_{\max} = 0.7 \text{ in}$$

$$\delta_{\min} = 0 \text{ in}$$

Maximum reaction at support A

$$R_{A_{\max}} = 11.3 \text{ kips}$$

$$R_{A_{\min}} = 11.3 \text{ kips}$$

Unfactored dead load reaction at support A

$$R_{A_{\text{Dead}}} = 2.8 \text{ kips}$$

Unfactored snow load reaction at support A

$$R_{A_{\text{Snow}}} = 5 \text{ kips}$$

Maximum reaction at support B

$$R_{B_{\max}} = 11.3 \text{ kips}$$

$$R_{B_{\min}} = 11.3 \text{ kips}$$

Unfactored dead load reaction at support B

$$R_{B_{\text{Dead}}} = 2.8 \text{ kips}$$

Unfactored snow load reaction at support B

$$R_{B_{\text{Snow}}} = 5 \text{ kips}$$

Section details

Section type

W 14x26 (AISC 15th Edn (v15.0))

ASTM steel designation

A992

Steel yield stress

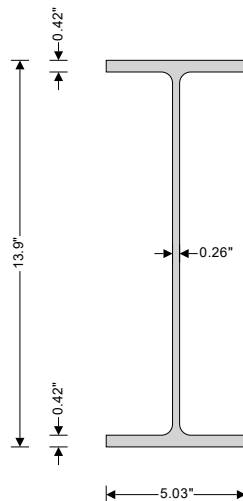
$$F_y = 50 \text{ ksi}$$

Steel tensile stress

$$F_u = 65 \text{ ksi}$$

Modulus of elasticity

$$E = 29000 \text{ ksi}$$



Resistance factors

Resistance factor for tensile yielding

$$\phi_{ty} = 0.90$$

Resistance factor for tensile rupture

$$\phi_{tr} = 0.75$$

Resistance factor for compression

$$\phi_c = 0.90$$

Resistance factor for flexure

$$\phi_b = 0.90$$



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Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio	$b_f / (2 \times t_f) = 5.99$	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 \times \sqrt{E / F_y} = 9.15$	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 \times \sqrt{E / F_y} = 24.08$	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio	$(d - 2 \times k) / t_w = 48.08$	
Limiting ratio for compact section	$\lambda_{pwf} = 3.76 \times \sqrt{E / F_y} = 90.55$	
Limiting ratio for non-compact section	$\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = 137.27$	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 11.312$ kips
Web area	$A_w = d \times t_w = 3.544$ in ²
Web plate buckling coefficient	$k_v = 5$
Web shear coefficient - eq G2-3	$C_v = 1$
Nominal shear strength – eq G2-1	$V_n = 0.6 \times F_y \times A_w \times C_v = 106.335$ kips
Resistance factor for shear	$\phi_v = 1.00$
Design shear strength	$V_c = \phi_v \times V_n = 106.335$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength	$M_r = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 66.967$ kips_ft
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Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y \times Z_x = 167.5$ kips_ft
Nominal flexural strength	$M_n = M_{nyld} = 167.500$ kips_ft
Design flexural strength	$M_c = \phi_b \times M_n = 150.750$ kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection	$\delta_{lim} = L_{s1} / 360 = 0.789$ in
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 0.653$ in

PASS - Maximum deflection does not exceed deflection limit

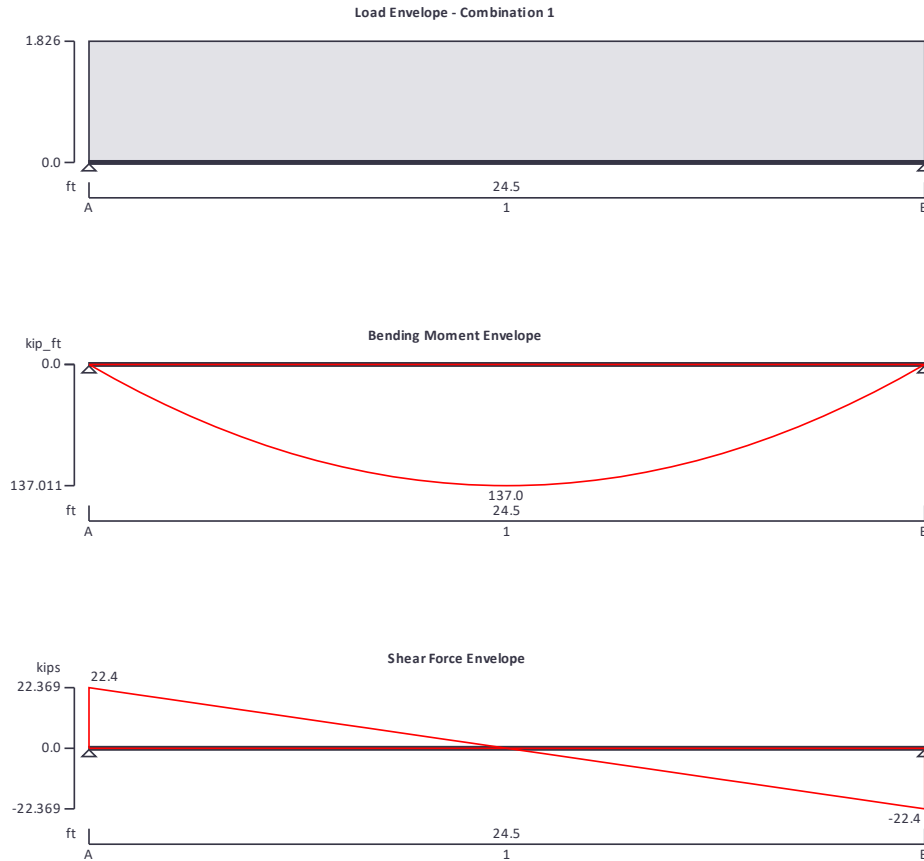


Project				Job Ref.	
Section W18x35 Beam				Sheet no./rev. 1	
Calc. by N	Date 7/27/2020	Chk'd by	Date	App'd by	Date

STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the LRFD method

Tedds calculation version 3.0.15



Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.42 kips/ft
	Snow full UDL 0.8 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Roof live × 1.60
		Snow × 1.60

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Section W18x35 Beam				Sheet no./rev. 2	
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Support B

Dead × 1.20
Live × 1.60
Roof live × 1.60
Snow × 1.60
Dead × 1.20
Live × 1.60
Roof live × 1.60
Snow × 1.60

Analysis results

Maximum moment

$$M_{\max} = 137 \text{ kips_ft}$$

$$M_{\min} = 0 \text{ kips_ft}$$

Maximum shear

$$V_{\max} = 22.4 \text{ kips}$$

$$V_{\min} = -22.4 \text{ kips}$$

Deflection

$$\delta_{\max} = 0.7 \text{ in}$$

$$\delta_{\min} = 0 \text{ in}$$

Maximum reaction at support A

$$R_{A_{\max}} = 22.4 \text{ kips}$$

$$R_{A_{\min}} = 22.4 \text{ kips}$$

Unfactored dead load reaction at support A

$$R_{A_{\text{Dead}}} = 5.6 \text{ kips}$$

Unfactored snow load reaction at support A

$$R_{A_{\text{Snow}}} = 9.8 \text{ kips}$$

Maximum reaction at support B

$$R_{B_{\max}} = 22.4 \text{ kips}$$

$$R_{B_{\min}} = 22.4 \text{ kips}$$

Unfactored dead load reaction at support B

$$R_{B_{\text{Dead}}} = 5.6 \text{ kips}$$

Unfactored snow load reaction at support B

$$R_{B_{\text{Snow}}} = 9.8 \text{ kips}$$

Section details

Section type

W 18x35

ASTM steel designation

A992

Steel yield stress

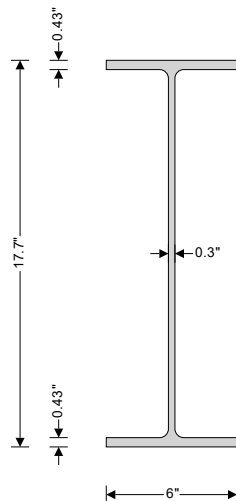
$$F_y = 50 \text{ ksi}$$

Steel tensile stress

$$F_u = 65 \text{ ksi}$$

Modulus of elasticity

$$E = 29000 \text{ ksi}$$



Resistance factors

Resistance factor for tensile yielding

$$\phi_{ty} = 0.90$$

Resistance factor for tensile rupture

$$\phi_{tr} = 0.75$$

Resistance factor for compression

$$\phi_c = 0.90$$

Resistance factor for flexure

$$\phi_b = 0.90$$



Project		Job Ref.	
Section W18x35 Beam		Sheet no./rev. 3	
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App'd by	Date	App'd by	Date

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio	$b_f / (2 \times t_f) = 7.06$	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 \times \sqrt{E / F_y} = 9.15$	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 \times \sqrt{E / F_y} = 24.08$	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio	$(d - 2 \times k) / t_w = 53.49$	
Limiting ratio for compact section	$\lambda_{pwf} = 3.76 \times \sqrt{E / F_y} = 90.55$	
Limiting ratio for non-compact section	$\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = 137.27$	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 22.369$ kips
Web area	$A_w = d \times t_w = 5.31$ in ²
Web plate buckling coefficient	$k_v = 5$
Web shear coefficient - eq G2-3	$C_v = 1$
Nominal shear strength – eq G2-1	$V_n = 0.6 \times F_y \times A_w \times C_v = 159.300$ kips
Resistance factor for shear	$\phi_v = 1.00$
Design shear strength	$V_c = \phi_v \times V_n = 159.300$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength	$M_r = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 137.011$ kips_ft
----------------------------	--

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y \times Z_x = 277.083$ kips_ft
Nominal flexural strength	$M_n = M_{nyld} = 277.083$ kips_ft
Design flexural strength	$M_c = \phi_b \times M_n = 249.375$ kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection	$\delta_{lim} = L_{s1} / 360 = 0.817$ in
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 0.688$ in

PASS - Maximum deflection does not exceed deflection limit

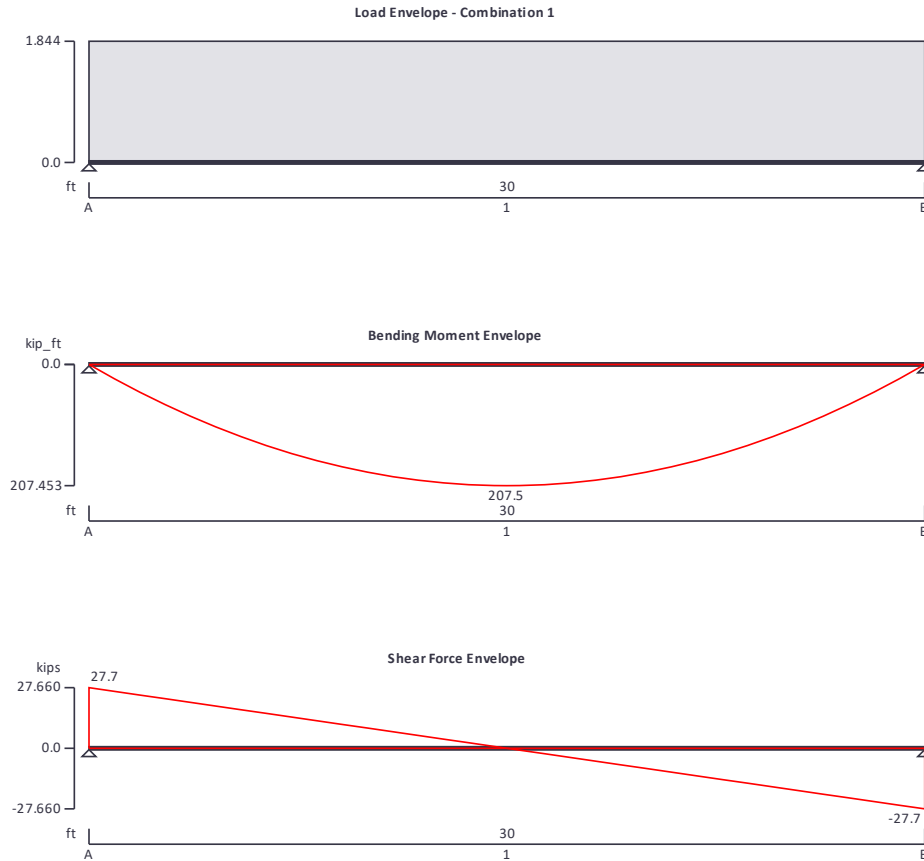


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STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

In accordance with AISC360-10 using the LRFD method

Tedds calculation version 3.0.15



Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.42 kips/ft
	Snow full UDL 0.8 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Roof live × 1.60
		Snow × 1.60

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Support B

Dead × 1.20
Live × 1.60
Roof live × 1.60
Snow × 1.60
Dead × 1.20
Live × 1.60
Roof live × 1.60
Snow × 1.60

Analysis results

Maximum moment

$$M_{\max} = 207.5 \text{ kips_ft}$$

$$M_{\min} = 0 \text{ kips_ft}$$

Maximum shear

$$V_{\max} = 27.7 \text{ kips}$$

$$V_{\min} = -27.7 \text{ kips}$$

Deflection

$$\delta_{\max} = 1 \text{ in}$$

$$\delta_{\min} = 0 \text{ in}$$

Maximum reaction at support A

$$R_{A_{\max}} = 27.7 \text{ kips}$$

$$R_{A_{\min}} = 27.7 \text{ kips}$$

Unfactored dead load reaction at support A

$$R_{A_{\text{Dead}}} = 7.1 \text{ kips}$$

Unfactored snow load reaction at support A

$$R_{A_{\text{Snow}}} = 12 \text{ kips}$$

Maximum reaction at support B

$$R_{B_{\max}} = 27.7 \text{ kips}$$

$$R_{B_{\min}} = 27.7 \text{ kips}$$

Unfactored dead load reaction at support B

$$R_{B_{\text{Dead}}} = 7.1 \text{ kips}$$

Unfactored snow load reaction at support B

$$R_{B_{\text{Snow}}} = 12 \text{ kips}$$

Section details

Section type

W 18x50 (AISC 15th Edn (v15.0))

ASTM steel designation

A992

Steel yield stress

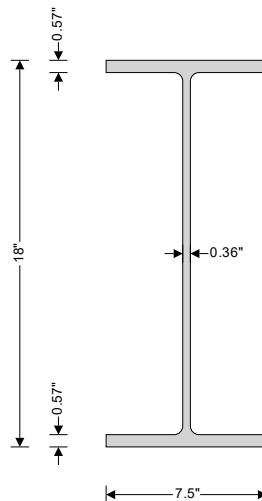
$$F_y = 50 \text{ ksi}$$

Steel tensile stress

$$F_u = 65 \text{ ksi}$$

Modulus of elasticity

$$E = 29000 \text{ ksi}$$



Resistance factors

Resistance factor for tensile yielding

$$\phi_{ty} = 0.90$$

Resistance factor for tensile rupture

$$\phi_{tr} = 0.75$$

Resistance factor for compression

$$\phi_c = 0.90$$

Resistance factor for flexure

$$\phi_b = 0.90$$



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Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio	$b_f / (2 \times t_f) = 6.58$	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 \times \sqrt{E / F_y} = 9.15$	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 \times \sqrt{E / F_y} = 24.08$	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio	$(d - 2 \times k) / t_w = 45.23$	
Limiting ratio for compact section	$\lambda_{pwf} = 3.76 \times \sqrt{E / F_y} = 90.55$	
Limiting ratio for non-compact section	$\lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = 137.27$	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 27.660$ kips
Web area	$A_w = d \times t_w = 6.39$ in ²
Web plate buckling coefficient	$k_v = 5$
Web shear coefficient - eq G2-3	$C_v = 1$
Nominal shear strength – eq G2-1	$V_n = 0.6 \times F_y \times A_w \times C_v = 191.700$ kips
Resistance factor for shear	$\phi_v = 1.00$
Design shear strength	$V_c = \phi_v \times V_n = 191.700$ kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength	$M_r = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 207.453$ kips_ft
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Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y \times Z_x = 420.833$ kips_ft
Nominal flexural strength	$M_n = M_{nyld} = 420.833$ kips_ft
Design flexural strength	$M_c = \phi_b \times M_n = 378.750$ kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, roof live and snow loads

Limiting deflection	$\delta_{lim} = L_{s1} / 360 = 1$ in
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 0.998$ in

PASS - Maximum deflection does not exceed deflection limit

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WOOD SHEAR WALL DESIGN (NDS)

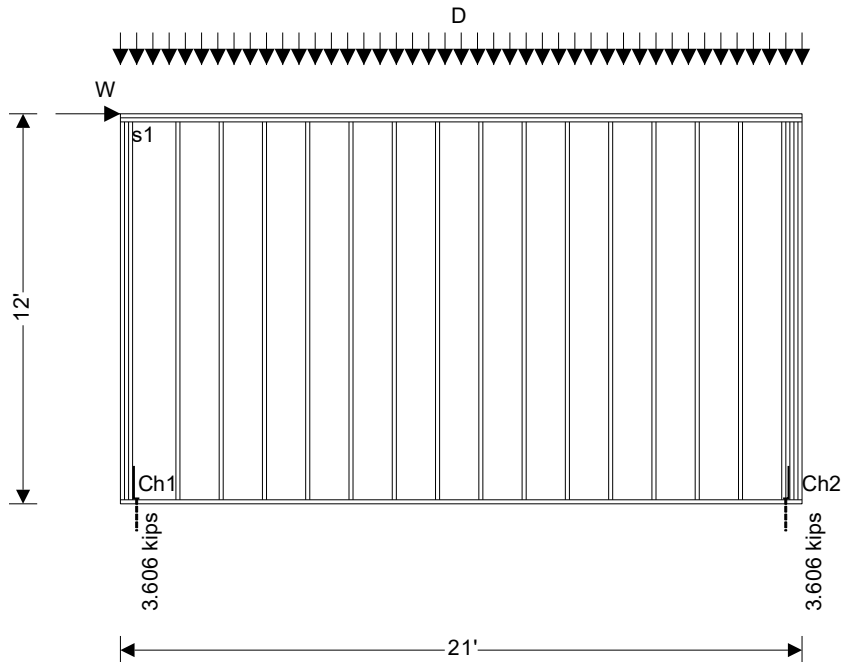
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural I wood panel sheathing on one side

Panel height $h = 12$ ft
 Panel length $b = 21$ ft
 Total area of wall $A = h \times b = 252$ ft²



Panel construction

Nominal stud size $2'' \times 6''$
 Dressed stud size $1.5'' \times 5.5''$
 Cross-sectional area of studs $A_s = 8.25$ in²
 Stud spacing $s = 16$ in
 Nominal end post size $3 \times 2'' \times 6''$
 Dressed end post size $3 \times 1.5'' \times 5.5''$
 Cross-sectional area of end posts $A_e = 24.75$ in²
 Hole diameter $\text{Dia} = 0.75$ in
 Net cross-sectional area of end posts $A_{en} = 21.375$ in²
 Nominal collector size $2 \times 2'' \times 6''$
 Dressed collector size $2 \times 1.5'' \times 5.5''$
 Service condition Dry
 Temperature 100 degF or less
 Vertical anchor stiffness $k_a = 30000$ lb/in



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

Species, grade and size classification	Hem-Fir, no.2 grade, 2" & wider
Specific gravity	G = 0.43
Tension parallel to grain	F _t = 525 lb/in²
Compression parallel to grain	F _c = 1300 lb/in²
Modulus of elasticity	E = 1300000 lb/in²
Minimum modulus of elasticity	E _{min} = 470000 lb/in²

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 3"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 = 1100 plf × min[1 - (0.5 - G), 1] = 1023 lb/ft
Nominal unit shear capacity for wind design	v _w = 1540 plf × min[1 - (0.5 - G), 1] = 1432.2 lb/ft
Apparent shear wall shear stiffness	G _a = 27 kips/in

Loading details

Dead load acting on top of panel	D = 200 lb/ft
Self weight of panel	S _{wt} = 12 lb/ft²
In plane wind load acting at head of panel	W = 12000 lbs
Wind load serviceability factor	f _{wserv} = 1.00

From IBC 2018 cl.1605.2

Load combination no.1	1.2D + 1.6(L _r or S or R) + 0.5W
Load combination no.2	1.2D + W + L _f + 0.5(L _r or S or R)
Load combination no.3	1.2D + E + L _f + 0.7S
Load combination no.4	0.9D + W
Load combination no.5	0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1	K _{Ft} = 2.70
Format conversion factor for compression – Table N1	K _{Fc} = 2.40
Format conversion factor for modulus of elasticity – Table N1	K _{FE} = 1.76
Resistance factor for tension – Table N2	φ _t = 0.80
Resistance factor for compression – Table N2	φ _c = 0.90
Resistance factor for modulus of elasticity – Table N2	φ _s = 0.85
Time effect factor – Table N3	λ = 1.00
Sheathing resistance factor	φ _D = 0.80
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	



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	$C_{ME} = 1.00$
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	$C_T = 1.00$
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times K_{FE} \times \phi_s \times C_{ME} \times C_{tE} \times C_i \times C_T = 705000 \text{ psi}$
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2 = 845 \text{ psi}$
Reference compression design value	$F_c^* = F_c \times K_{Fc} \times \phi_c \times \lambda \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i = 3089 \text{ psi}$
For sawn lumber	$c = 0.8$
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = 0.26$

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	$b = 21 \text{ ft}$
Shear wall aspect ratio	$h / b = 0.571$

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w_max} = W = 12 \text{ kips}$
Shear capacity for wind loading	$V_w = \phi_D \times v_w \times b = 24.061 \text{ kips}$ $V_{w_max} / V_w = 0.499$ PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	$h / b = 0.571$
Load combination 4	
Shear force for maximum tension	$V = W = 12 \text{ kips}$
Axial force for maximum tension	$P = (0.9 \times (D + S_{wt} \times h)) \times b_1 / 2 = 3.251 \text{ kips}$
Maximum tensile force in chord	$T = V \times h / (b) - P = 3.606 \text{ kips}$
Maximum applied tensile stress	$f_t = T / A_{en} = 169 \text{ lb/in}^2$
Design tensile stress	$F_t' = F_t \times K_{Ft} \times \phi_t \times \lambda \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1474 \text{ lb/in}^2$ $f_t / F_t' = 0.114$ PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 2

Shear force for maximum compression	$V = W = 12 \text{ kips}$
Axial force for maximum compression	$P = (1.2 \times (D + S_{wt} \times h)) \times s / 2 = 0.275 \text{ kips}$
Maximum compressive force in chord	$C = V \times h / (b) + P = 7.132 \text{ kips}$
Maximum applied compressive stress	$f_c = C / A_e = 288 \text{ lb/in}^2$
Design compressive stress	$F_c' = F_c \times K_{Fc} \times \phi_c \times \lambda \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 791 \text{ lb/in}^2$ $f_c / F_c' = 0.364$ PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1	$T_1 = 3.606 \text{ kips}$
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Chord 2

$$T_2 = 3.606 \text{ kips}$$

Wind load deflection

Design shear force

$$V_{\delta w} = f_{w_{serv}} \times W = 12 \text{ kips}$$

Deflection limit

$$\Delta_{w_allow} = h / 400 = 0.36 \text{ in}$$

Induced unit shear

$$v_{\delta w} = V_{\delta w} / b = 571.43 \text{ lb/ft}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = 4.690 \text{ kips}$$

Shear wall deflection – Eqn. 4.3-1

$$\delta_{sw} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_{\delta} / (k_a \times b) =$$

$$0.355 \text{ in}$$

$$\delta_{sw} / \Delta_{w_allow} = 0.986$$

PASS - Shear wall deflection is less than deflection limit

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WOOD SHEAR WALL DESIGN (NDS)

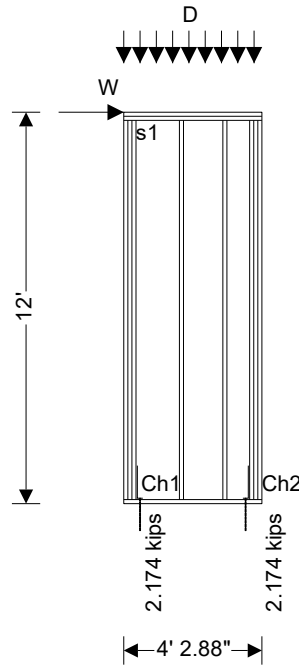
In accordance with NDS2018 load reduction factor design and the segmented shear wall method

Tedds calculation version 1.2.04

Panel details

Structural I wood panel sheathing on one side

Panel height $h = 12$ ft
 Panel length $b = 4.24$ ft
 Total area of wall $A = h \times b = 50.88$ ft²



Panel construction

Nominal stud size $2'' \times 6''$
 Dressed stud size $1.5'' \times 5.5''$
 Cross-sectional area of studs $A_s = 8.25$ in²
 Stud spacing $s = 16$ in
 Nominal end post size $3 \times 2'' \times 6''$
 Dressed end post size $3 \times 1.5'' \times 5.5''$
 Cross-sectional area of end posts $A_e = 24.75$ in²
 Hole diameter $\text{Dia} = 0.75$ in
 Net cross-sectional area of end posts $A_{en} = 21.375$ in²
 Nominal collector size $2 \times 2'' \times 6''$
 Dressed collector size $2 \times 1.5'' \times 5.5''$
 Service condition Dry
 Temperature 100 degF or less
 Vertical anchor stiffness $k_a = 30000$ lb/in



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

Species, grade and size classification	Hem-Fir, no.2 grade, 2" & wider
Specific gravity	G = 0.43
Tension parallel to grain	F _t = 525 lb/in²
Compression parallel to grain	F _c = 1300 lb/in²
Modulus of elasticity	E = 1300000 lb/in²
Minimum modulus of elasticity	E _{min} = 470000 lb/in²

Sheathing details

Sheathing material	7/16" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 3"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 = 1100 plf × min[1 - (0.5 - G), 1] = 1023 lb/ft
Nominal unit shear capacity for wind design	v _w = 1540 plf × min[1 - (0.5 - G), 1] = 1432.2 lb/ft
Apparent shear wall shear stiffness	G _a = 27 kips/in

Loading details

Dead load acting on top of panel	D = 200 lb/ft
Self weight of panel	S _{wt} = 12 lb/ft²
In plane wind load acting at head of panel	W = 1000 lbs
Wind load serviceability factor	f _{wserv} = 1.00

From IBC 2015 cl.1605.2.1 Basic load combinations

Load combination no.1	1.2D + 1.6(L _r or S or R) + 0.5W
Load combination no.2	1.2D + W + 0.5L _f + 0.5(L _r or S or R)
Load combination no.3	1.2D + E + 0.5L _f + 0.7S
Load combination no.4	0.9D + W
Load combination no.5	0.9D + E

Adjustment factors

Format conversion factor for tension – Table N1	K _{Ft} = 2.70
Format conversion factor for compression – Table N1	K _{Fc} = 2.40
Format conversion factor for modulus of elasticity – Table N1	K _{FE} = 1.76
Resistance factor for tension – Table N2	φ _t = 0.80
Resistance factor for compression – Table N2	φ _c = 0.90
Resistance factor for modulus of elasticity – Table N2	φ _s = 0.85
Time effect factor – Table N3	λ = 1.00
Sheathing resistance factor	φ _D = 0.80
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	



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$C_{ME} = 1.00$
 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 $C_T = 1.00$
 Adjusted modulus of elasticity $E_{min}' = E_{min} \times K_{FE} \times \phi_s \times C_{ME} \times C_{tE} \times C_i \times C_T = 705000$ psi
 Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2 = 845$ psi
 Reference compression design value $F_c^* = F_c \times K_{Fc} \times \phi_c \times \lambda \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i = 3089$ psi
 For sawn lumber $c = 0.8$
 Column stability factor – eqn.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c} = 0.26$

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio 3.5
 Shear wall length $b = 4.24$ ft
 Shear wall aspect ratio $h / b = 2.83$

Segmented shear wall capacity

Maximum shear force under wind loading $V_{w_max} = W = 1$ kips
 Shear capacity for wind loading $V_w = \phi_D \times v_w \times b \times (1.25 - 0.125 \times h / b_s) = 4.354$ kips
 $V_{w_max} / V_w = 0.23$
PASS - Shear capacity for wind load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio $h / b = 2.83$
 Load combination 4
 Shear force for maximum tension $V = W = 1$ kips
 Axial force for maximum tension $P = (0.9 \times (D + S_{wt} \times h)) \times b_1 / 2 = 0.656$ kips
 Maximum tensile force in chord $T = V \times h / (b) - P = 2.174$ kips
 Maximum applied tensile stress $f_t = T / A_{en} = 102$ lb/in²
 Design tensile stress $F_t' = F_t \times K_{Ft} \times \phi_t \times \lambda \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i = 1474$ lb/in²
 $f_t / F_t' = 0.069$
PASS - Design tensile stress exceeds maximum applied tensile stress

Load combination 2

Shear force for maximum compression $V = W = 1$ kips
 Axial force for maximum compression $P = (1.2 \times (D + S_{wt} \times h)) \times s / 2 = 0.275$ kips
 Maximum compressive force in chord $C = V \times h / (b) + P = 3.105$ kips
 Maximum applied compressive stress $f_c = C / A_e = 125$ lb/in²
 Design compressive stress $F_c' = F_c \times K_{Fc} \times \phi_c \times \lambda \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P = 791$ lb/in²
 $f_c / F_c' = 0.159$

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 $T_1 = 2.174$ kips



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Chord 2

$$T_2 = 2.174 \text{ kips}$$

Wind load deflection

Design shear force

$$V_{\delta w} = f_{w_{serv}} \times W = 1 \text{ kips}$$

Deflection limit

$$\Delta_{w_allow} = h / 400 = 0.36 \text{ in}$$

Induced unit shear

$$v_{\delta w} = V_{\delta w} / b = 235.85 \text{ lb/ft}$$

Anchor tension force

$$T_{\delta} = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) = 2.393 \text{ kips}$$

Shear wall deflection – Eqn. 4.3-1

$$\delta_{sw} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times T_{\delta} / (k_a \times b) = 0.354 \text{ in}$$

$$\delta_{sw} / \Delta_{w_allow} = 0.985$$

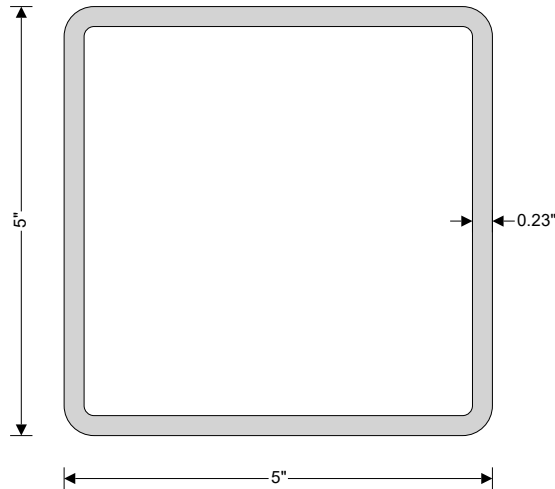
PASS - Shear wall deflection is less than deflection limit

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STEEL COLUMN DESIGN

In accordance with AISC360-10 and the LRFD method

Tedds calculation version 1.0.09



Column and loading details

Column details

Column section

HSS 5x5x1/4

Design loading

Required axial strength

$P_r = 24$ kips (Compression)

Moment about x axis at end 1

$M_{x1} = 2.0$ kips_{ft}

Moment about x axis at end 2

$M_{x2} = 2.0$ kips_{ft}

Single curvature bending about x axis

Maximum moment about x axis

$M_x = \max(\text{abs}(M_{x1}), \text{abs}(M_{x2})) = 2.0$ kips_{ft}

Maximum moment about y axis

$M_y = 0.0$ kips_{ft}

Maximum shear force parallel to y axis

$V_{ry} = 2.0$ kips

Maximum shear force parallel to x axis

$V_{rx} = 0.0$ kips

Material details

Steel grade

A500 Gr. B

Yield strength

$F_y = 46$ ksi

Ultimate strength

$F_u = 58$ ksi

Modulus of elasticity

$E = 29000$ ksi

Shear modulus of elasticity

$G = 11200$ ksi

Unbraced lengths

For buckling about x axis

$L_x = 156$ in

For buckling about y axis

$L_y = 156$ in

For torsional buckling

$L_z = 156$ in

Effective length factors

For buckling about x axis

$K_x = 1.00$

For buckling about y axis

$K_y = 1.00$



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For torsional buckling

$$K_z = 1.00$$

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f - 3 \times t = 4.301$ in

Critical web width $h = d - 3 \times t = 4.301$ in

Width to thickness ratio of flange (compression) $\lambda_{f_c} = b / t = 18.459$

Width to thickness ratio of web (compression) $\lambda_{w_c} = h / t = 18.459$

Width to thickness ratio of flange (major flexure) $\lambda_{f_{fx}} = b / t = 18.459$

Width to thickness ratio of web (major flexure) $\lambda_{w_{fx}} = h / t = 18.459$

Width to thickness ratio of flange (minor flexure) $\lambda_{f_{fy}} = h / t = 18.459$

Width to thickness ratio of web (minor flexure) $\lambda_{w_{fy}} = b / t = 18.459$

Compression

Limit for nonslender section $\lambda_{r_c} = 1.40 \times \sqrt{(E / F_y)} = 35.152$

The section is nonslender in compression

Flexure

Limit for compact flange $\lambda_{pf_f} = 1.12 \times \sqrt{(E / F_y)} = 28.121$

Limit for noncompact flange $\lambda_{rf_f} = 1.40 \times \sqrt{(E / F_y)} = 35.152$

Limit for compact web $\lambda_{pw_f} = 2.42 \times \sqrt{(E / F_y)} = 60.762$

Limit for noncompact web $\lambda_{rw_f} = 5.70 \times \sqrt{(E / F_y)} = 143.118$

The section is compact in flexure about the major axis

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = K_x \times L_x / r_x = 80.8$

Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y = 80.8$

Second order effects

Second order effects for bending about x axis (cl. App 8.1)

Coefficient C_m $C_{mx} = 0.6 + 0.4 \times M_{x1} / M_{x2} = 1.000$

Coefficient α $\alpha = 1.0$

Elastic critical buckling stress $P_{e1x} = \pi^2 \times E \times I_x / (K_{1x} \times L_x)^2 = 188.2$ kips

P- δ amplifier $B_{1x} = \max(1.0, C_{mx} / (1 - \alpha \times P_r / P_{e1x})) = 1.146$

Required flexural strength $M_{rx} = B_{1x} \times M_x = 2.3$ kips_ft

Second order effects for bending about y axis (cl. App 8.1)

Coefficient C_m $C_{my} = 1.0$

Coefficient α $\alpha = 1.0$

Elastic critical buckling stress $P_{e1y} = \pi^2 \times E \times I_y / (K_{1y} \times L_y)^2 = 188.2$ kips

P- δ amplifier $B_{1y} = \max(1.0, C_{my} / (1 - \alpha \times P_r / P_{e1y})) = 1.146$

Required flexural strength $M_{ry} = B_{1y} \times M_y = 0.0$ kips_ft

Shear strength



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Shear parallel to the minor axis (cl. G2.1)

Shear area	$A_w = 2 \times (d - 3 \times t) \times t = 2.004 \text{ in}^2$
Web plate buckling coefficient	$k_v = 5.0$
Web shear coefficient	$C_v = 1.000$
Nominal shear strength	$V_{ny} = 0.6 \times F_y \times A_w \times C_v = 55.3 \text{ kips}$

Design shear strength (cl.G1 & G2.1(a))

Resistance factor for shear	$\phi_v = 0.90$
Design shear strength	$V_{cy} = \phi_v \times V_{ny} = 49.8 \text{ kips}$

PASS - The design shear strength exceeds the required shear strength

Reduction factor for slender elements

Reduction factor for slender elements (E7)

The section does not contain any slender elements therefore:-

Slender element reduction factor	$Q = 1.0$
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Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress	$F_{ex} = (\pi^2 \times E) / (SR_x)^2 = 43.8 \text{ ksi}$
Reduction factor	$Q_x = Q = 1.000$
Flexural buckling stress about x axis	$F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y = 29.6 \text{ ksi}$
Nominal flexural buckling strength	$P_{nx} = F_{crx} \times A_g = 127.5 \text{ kips}$

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress	$F_{ey} = (\pi^2 \times E) / (SR_y)^2 = 43.8 \text{ ksi}$
Reduction factor	$Q_y = Q = 1.000$
Flexural buckling stress about y axis	$F_{cry} = Q_y \times (0.658^{Q_y \times F_y / F_{ey}}) \times F_y = 29.6 \text{ ksi}$
Nominal flexural buckling strength	$P_{ny} = F_{cry} \times A_g = 127.5 \text{ kips}$

Design compressive strength (cl.E1)

Resistance factor for compression	$\phi_c = 0.90$
Design compressive strength	$P_c = \phi_c \times \min(P_{nx}, P_{ny}) = 114.7 \text{ kips}$

PASS - The design compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F7.1)

Nominal flexural strength	$M_{nx_yld} = M_{px} = F_y \times Z_x = 29.2 \text{ kips_ft}$
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Design flexural strength about the major axis (cl. F1)

Resistance factor for flexure	$\phi_b = 0.90$
Design flexural strength	$M_{cx} = \phi_b \times M_{nx_yld} = 26.3 \text{ kips_ft}$

PASS - The design flexural strength about the major axis exceeds the required flexural strength

Combined forces

Member utilization (cl. H1.1)

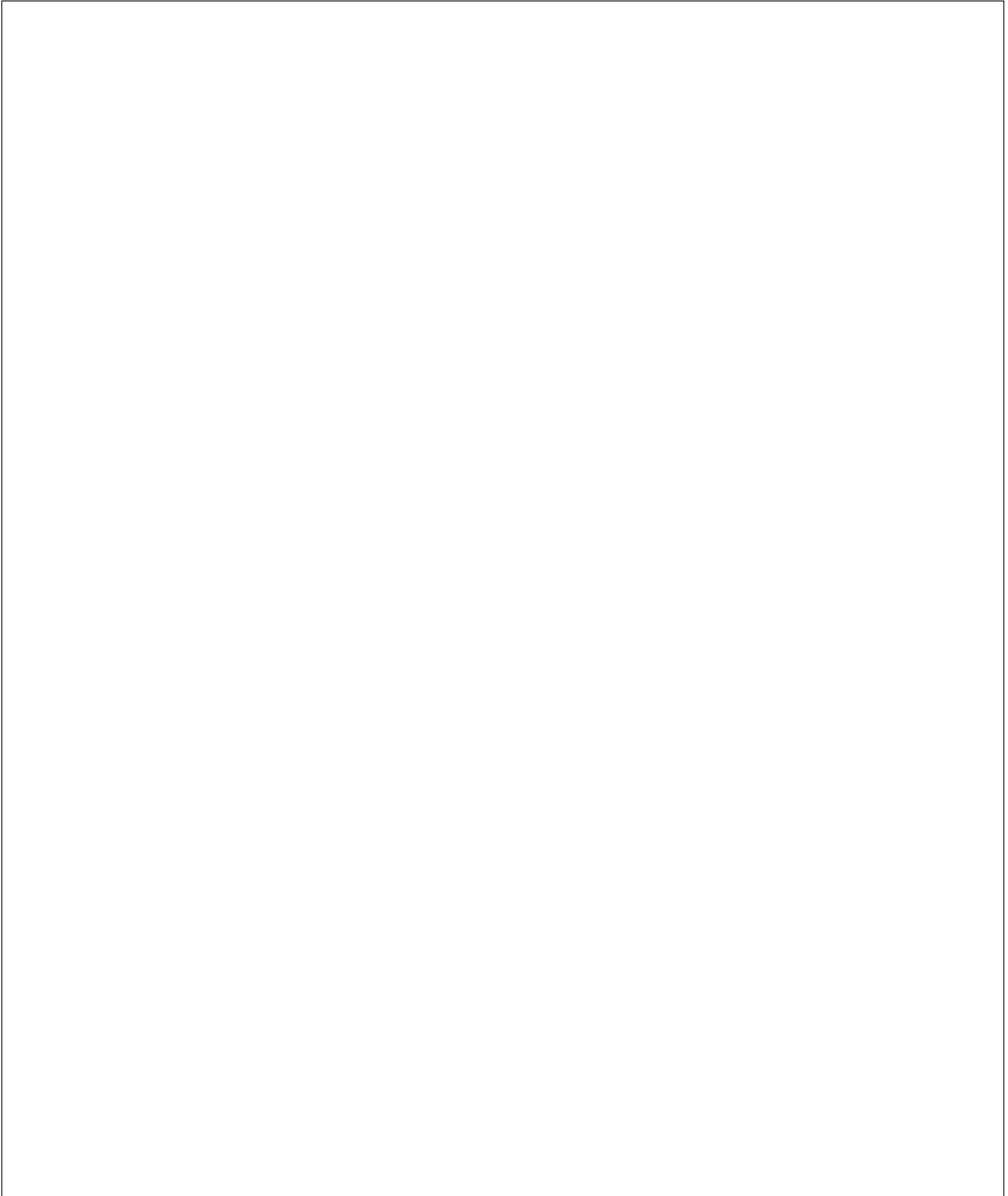
Equation H1-1a	$UR = \text{abs}(P_r) / P_c + 8 / 9 \times (M_{rx} / M_{cx} + M_{ry} / M_{cy}) = 0.287$
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PASS - The member is adequate for the combined forces



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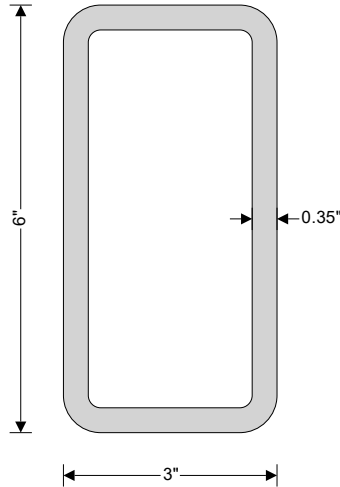


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STEEL COLUMN DESIGN

In accordance with AISC360-10 and the LRFD method

Tedds calculation version 1.0.09



Column and loading details

Column details

Column section

HSS 6x3x3/8

Design loading

Required axial strength

$P_r = 64$ kips (Compression)

Moment about x axis at end 1

$M_{x1} = 2.0$ kips_ft

Moment about x axis at end 2

$M_{x2} = 2.0$ kips_ft

Single curvature bending about x axis

Maximum moment about x axis

$M_x = \max(\text{abs}(M_{x1}), \text{abs}(M_{x2})) = 2.0$ kips_ft

Maximum moment about y axis

$M_y = 0.0$ kips_ft

Maximum shear force parallel to y axis

$V_{ry} = 2.0$ kips

Maximum shear force parallel to x axis

$V_{rx} = 0.0$ kips

Material details

Steel grade

A500 Gr. B

Yield strength

$F_y = 46$ ksi

Ultimate strength

$F_u = 58$ ksi

Modulus of elasticity

$E = 29000$ ksi

Shear modulus of elasticity

$G = 11200$ ksi

Unbraced lengths

For buckling about x axis

$L_x = 156$ in

For buckling about y axis

$L_y = 156$ in

For torsional buckling

$L_z = 156$ in

Effective length factors

For buckling about x axis

$K_x = 1.00$

For buckling about y axis

$K_y = 1.00$



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For torsional buckling

$$K_z = 1.00$$

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f - 3 \times t = 1.953$ in

Critical web width $h = d - 3 \times t = 4.953$ in

Width to thickness ratio of flange (compression) $\lambda_{f_c} = b / t = 5.596$

Width to thickness ratio of web (compression) $\lambda_{w_c} = h / t = 14.192$

Width to thickness ratio of flange (major flexure) $\lambda_{f_{fx}} = b / t = 5.596$

Width to thickness ratio of web (major flexure) $\lambda_{w_{fx}} = h / t = 14.192$

Width to thickness ratio of flange (minor flexure) $\lambda_{f_{fy}} = h / t = 14.192$

Width to thickness ratio of web (minor flexure) $\lambda_{w_{fy}} = b / t = 5.596$

Compression

Limit for nonslender section $\lambda_{r_c} = 1.40 \times \sqrt{(E / F_y)} = 35.152$

The section is nonslender in compression

Flexure

Limit for compact flange $\lambda_{pf_f} = 1.12 \times \sqrt{(E / F_y)} = 28.121$

Limit for noncompact flange $\lambda_{rf_f} = 1.40 \times \sqrt{(E / F_y)} = 35.152$

Limit for compact web $\lambda_{pw_f} = 2.42 \times \sqrt{(E / F_y)} = 60.762$

Limit for noncompact web $\lambda_{rw_f} = 5.70 \times \sqrt{(E / F_y)} = 143.118$

The section is compact in flexure about the major axis

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = K_x \times L_x / r_x = 76.5$

Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y = 133.3$

Second order effects

Second order effects for bending about x axis (cl. App 8.1)

Coefficient C_m $C_{mx} = 0.6 + 0.4 \times M_{x1} / M_{x2} = 1.000$

Coefficient α $\alpha = 1.0$

Elastic critical buckling stress $P_{e1x} = \pi^2 \times E \times I_x / (K_{1x} \times L_x)^2 = 267.0$ kips

P- δ amplifier $B_{1x} = \max(1.0, C_{mx} / (1 - \alpha \times P_r / P_{e1x})) = 1.315$

Required flexural strength $M_{rx} = B_{1x} \times M_x = 2.6$ kips_ft

Second order effects for bending about y axis (cl. App 8.1)

Coefficient C_m $C_{my} = 1.0$

Coefficient α $\alpha = 1.0$

Elastic critical buckling stress $P_{e1y} = \pi^2 \times E \times I_y / (K_{1y} \times L_y)^2 = 88.0$ kips

P- δ amplifier $B_{1y} = \max(1.0, C_{my} / (1 - \alpha \times P_r / P_{e1y})) = 3.670$

Required flexural strength $M_{ry} = B_{1y} \times M_y = 0.0$ kips_ft

Shear strength



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Shear parallel to the minor axis (cl. G2.1)

Shear area $A_w = 2 \times (d - 3 \times t) \times t = 3.457 \text{ in}^2$
 Web plate buckling coefficient $k_v = 5.0$
 Web shear coefficient $C_v = 1.000$
 Nominal shear strength $V_{ny} = 0.6 \times F_y \times A_w \times C_v = 95.4 \text{ kips}$

Design shear strength (cl.G1 & G2.1(a))

Resistance factor for shear $\phi_v = 0.90$
 Design shear strength $V_{cy} = \phi_v \times V_{ny} = 85.9 \text{ kips}$

PASS - The design shear strength exceeds the required shear strength

Reduction factor for slender elements

Reduction factor for slender elements (E7)

The section does not contain any slender elements therefore:-

Slender element reduction factor $Q = 1.0$

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = 48.9 \text{ ksi}$
 Reduction factor $Q_x = Q = 1.000$
 Flexural buckling stress about x axis $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y = 31.0 \text{ ksi}$
 Nominal flexural buckling strength $P_{nx} = F_{crx} \times A_g = 170.1 \text{ kips}$

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (SR_y)^2 = 16.1 \text{ ksi}$
 Flexural buckling stress about y axis $F_{cry} = 0.877 \times F_{ey} = 14.1 \text{ ksi}$
 Nominal flexural buckling strength $P_{ny} = F_{cry} \times A_g = 77.4 \text{ kips}$

Design compressive strength (cl.E1)

Resistance factor for compression $\phi_c = 0.90$
 Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny}) = 69.6 \text{ kips}$

PASS - The design compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F7.1)

Nominal flexural strength $M_{nx_yld} = M_{px} = F_y \times Z_x = 37.9 \text{ kips_ft}$

Design flexural strength about the major axis (cl. F1)

Resistance factor for flexure $\phi_b = 0.90$
 Design flexural strength $M_{cx} = \phi_b \times M_{nx_yld} = 34.2 \text{ kips_ft}$

PASS - The design flexural strength about the major axis exceeds the required flexural strength

Combined forces

Member utilization (cl. H1.1)

Equation H1-1a $UR = \text{abs}(P_r) / P_c + 8 / 9 \times (M_{rx} / M_{cx} + M_{ry} / M_{cy}) = 0.988$

PASS - The member is adequate for the combined forces

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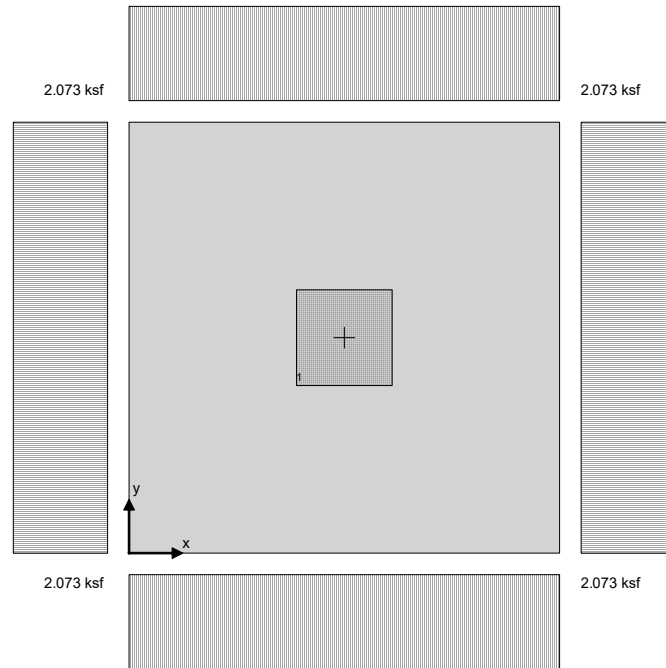
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation	$L_x = 4.5$ ft
Width of foundation	$L_y = 4.5$ ft
Foundation area	$A = L_x \times L_y = 20.25$ ft ²
Depth of foundation	$h = 18$ in
Depth of soil over foundation	$h_{soil} = 12$ in
Density of concrete	$\gamma_{conc} = 150.0$ lb/ft ³



Column no.1 details

Length of column	$l_{x1} = 12.00$ in
Width of column	$l_{y1} = 12.00$ in
position in x-axis	$x_1 = 27.00$ in
position in y-axis	$y_1 = 27.00$ in

Soil properties

Gross allowable bearing pressure	$Q_{allow_Gross} = 2.5$ ksf
Density of soil	$\gamma_{soil} = 120.0$ lb/ft ³
Angle of internal friction	$\phi_b = 30.0$ deg
Design base friction angle	$\delta_{bb} = 30.0$ deg
Coefficient of base friction	$\tan(\delta_{bb}) = 0.577$
Self weight	$F_{swt} = h \times \gamma_{conc} = 225$ psf



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Soil weight

$$F_{\text{soil}} = h_{\text{soil}} \times \gamma_{\text{soil}} = \mathbf{120 \text{ psf}}$$

Column no.1 loads

Dead load in z

$$F_{Dz1} = \mathbf{13.0 \text{ kips}}$$

Snow load in z

$$F_{Sz1} = \mathbf{22.0 \text{ kips}}$$

Footing analysis for soil and stability

Load combinations per IBC 2015

1.0D (0.395)

1.0D + 1.0L (0.395)

1.0D + 1.0Lr (0.395)

1.0D + 1.0S (0.829)

1.0D + 1.0R (0.395)

1.0D + 0.75L + 0.75Lr (0.395)

1.0D + 0.75L + 0.75S (0.721)

1.0D + 0.75L + 0.75R (0.395)

1.0D + 0.6W (0.395)

Combination 4 results: 1.0D + 1.0S

Forces on foundation

Force in z-axis

$$F_{dz} = \gamma_D \times A \times (F_{\text{swt}} + F_{\text{soil}}) + \gamma_D \times F_{Dz1} + \gamma_S \times F_{Sz1} = \mathbf{42.0 \text{ kips}}$$

Moments on foundation

Moment in x-axis, about x is 0

$$M_{dx} = \gamma_D \times (A \times (F_{\text{swt}} + F_{\text{soil}}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times X_1) + \gamma_S \times (F_{Sz1} \times X_1) = \mathbf{94.5 \text{ kip_ft}}$$

Moment in y-axis, about y is 0

$$M_{dy} = \gamma_D \times (A \times (F_{\text{swt}} + F_{\text{soil}}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) = \mathbf{94.5 \text{ kip_ft}}$$

Uplift verification

Vertical force

$$F_{dz} = \mathbf{41.986 \text{ kips}}$$

PASS - Foundation is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.073 \text{ ksf}}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.073 \text{ ksf}}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.073 \text{ ksf}}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.073 \text{ ksf}}$$

Minimum base pressure

$$q_{\text{min}} = \min(q_1, q_2, q_3, q_4) = \mathbf{2.073 \text{ ksf}}$$

Maximum base pressure

$$q_{\text{max}} = \max(q_1, q_2, q_3, q_4) = \mathbf{2.073 \text{ ksf}}$$

Allowable bearing capacity

Allowable bearing capacity

$$Q_{\text{allow}} = Q_{\text{allow_Gross}} = \mathbf{2.5 \text{ ksf}}$$

$$Q_{\text{max}} / Q_{\text{allow}} = \mathbf{0.829}$$

PASS - Allowable bearing capacity exceeds design base pressure



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FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details

Compressive strength of concrete	$f_c = 3000$ psi
Yield strength of reinforcement	$f_y = 60000$ psi
Cover to reinforcement	$c_{nom} = 3$ in
Concrete type	Normal weight
Concrete modification factor	$\lambda = 1.00$
Column type	Concrete

Analysis and design of concrete footing

Load combinations per IBC 2015

- 1.4D (0.055)
- 1.2D + 1.6L + 0.5Lr (0.049)
- 1.2D + 1.6L + 0.5S (0.081)
- 1.2D + 1.6L + 0.5R (0.049)
- 1.2D + 1.0L + 1.6Lr (0.049)
- 1.2D + 1.0L + 1.6S (0.155)
- 1.2D + 1.0L + 1.6R (0.049)
- 1.2D + 1.6Lr + 0.5W (0.049)

Combination 6 results: 1.2D + 1.0L + 1.6S

Forces on foundation

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_S \times F_{Sz1} = 59.2$ kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_S \times (F_{Sz1} \times x_1) = 133.2$ kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) = 133.2$ kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2 = 0$ in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0$ in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = 2.923$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = 2.923$ ksf

$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = 2.923$ ksf

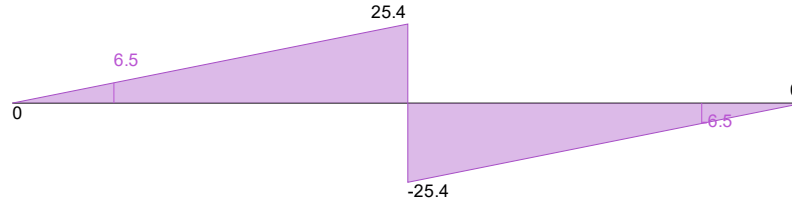
$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = 2.923$ ksf

Minimum ultimate base pressure $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 2.923$ ksf

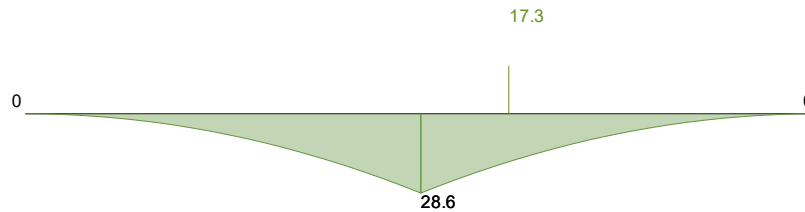
Maximum ultimate base pressure $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 2.923$ ksf

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Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u,x,max} = 17.286$ kip_ft
Tension reinforcement provided 6 No.5 bottom bars (9.4 in c/c)
Area of tension reinforcement provided $A_{sx,bot,prov} = 1.86$ in²
Minimum area of reinforcement (8.6.1.1) $A_{s,min} = 0.0018 \times L_y \times h = 1.75$ in²

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) $s_{max} = \min(2 \times h, 18 \text{ in}) = 18$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - C_{nom} - \phi_{x,bot} / 2 = 14.688$ in
Depth of compression block $a = A_{sx,bot,prov} \times f_y / (0.85 \times f'_c \times L_y) = 0.810$ in
Neutral axis factor $\beta_1 = 0.85$
Depth to neutral axis $c = a / \beta_1 = 0.953$ in
Strain in tensile reinforcement (8.3.3.1) $\epsilon_t = 0.003 \times d / c - 0.003 = 0.04321$

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity $M_n = A_{sx,bot,prov} \times f_y \times (d - a / 2) = 132.825$ kip_ft
Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\epsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.900$
Design moment capacity $\phi M_n = \phi_f \times M_n = 119.543$ kip_ft
 $M_{u,x,max} / \phi M_n = 0.145$

PASS - Design moment capacity exceeds ultimate moment load

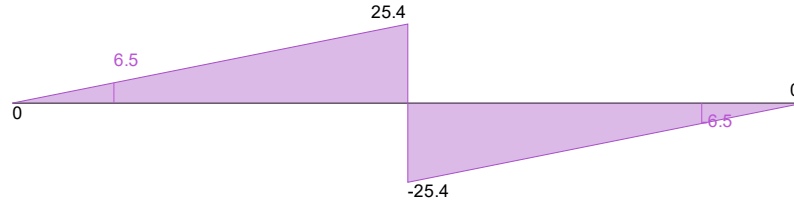
One-way shear design, x direction

Ultimate shear force $V_{u,x} = 6.526$ kips
Depth to reinforcement $d_v = h - C_{nom} - \phi_{x,bot} / 2 = 14.688$ in
Shear strength reduction factor $\phi_v = 0.75$
Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = 86.882$ kips
Design shear capacity $\phi V_n = \phi_v \times V_n = 65.162$ kips
 $V_{u,x} / \phi V_n = 0.100$

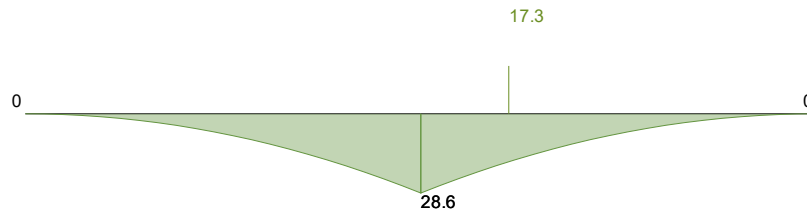
PASS - Design shear capacity exceeds ultimate shear load

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Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u,y,max} = 17.286$ kip_ft
Tension reinforcement provided	6 No.5 bottom bars (9.4 in c/c)
Area of tension reinforcement provided	$A_{sy,bot,prov} = 1.86$ in ²
Minimum area of reinforcement (8.6.1.1)	$A_{s,min} = 0.0018 \times L_x \times h = 1.75$ in ²

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18)$ in = 18 in
--	---

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - C_{nom} - \phi_{x,bot} - \phi_{y,bot} / 2 = 14.063$ in
Depth of compression block	$a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.810$ in
Neutral axis factor	$\beta_1 = 0.85$
Depth to neutral axis	$c = a / \beta_1 = 0.953$ in
Strain in tensile reinforcement (8.3.3.1)	$\epsilon_t = 0.003 \times d / c - 0.003 = 0.04125$

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	$M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = 127.013$ kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\epsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.900$
Design moment capacity	$\phi M_n = \phi_f \times M_n = 114.311$ kip_ft
	$M_{u,y,max} / \phi M_n = 0.151$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u,y} = 6.526$ kips
Depth to reinforcement	$d_v = h - C_{nom} - \phi_{x,bot} - \phi_{y,bot} / 2 = 14.063$ in
Shear strength reduction factor	$\phi_v = 0.75$
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v = 83.185$ kips
Design shear capacity	$\phi V_n = \phi_v \times V_n = 62.389$ kips
	$V_{u,y} / \phi V_n = 0.105$

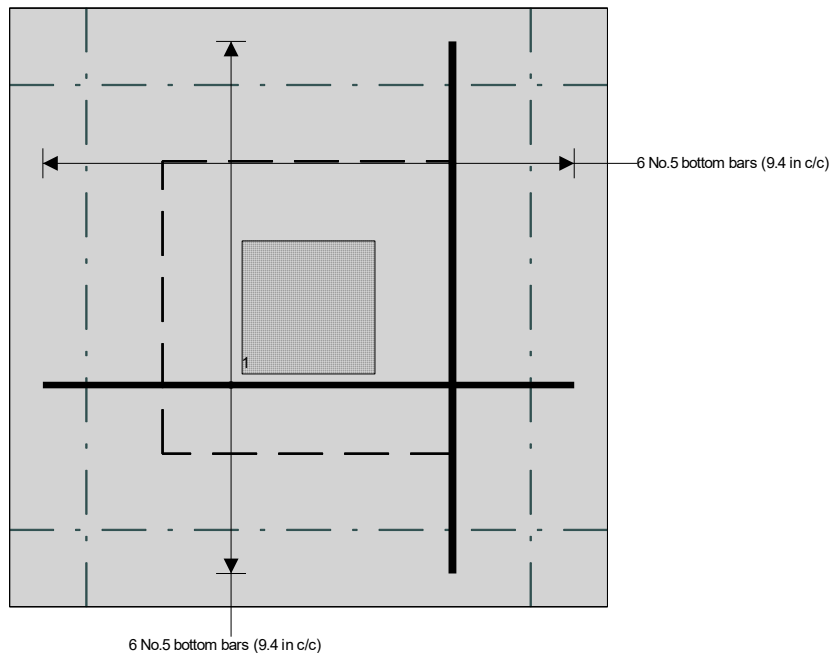
PASS - Design shear capacity exceeds ultimate shear load

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Two-way shear design at column 1

Depth to reinforcement	$d_{v2} = 14.375$ in
Shear perimeter length (22.6.4)	$l_{xp} = 26.375$ in
Shear perimeter width (22.6.4)	$l_{yp} = 26.375$ in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = 105.500$ in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} = 695.641$ in ²
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} = 551.641$ in ²
Ultimate bearing pressure at center of shear area	$q_{up,avg} = 2.923$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_S \times F_{Sz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up,avg} \times A_p = 38.537$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = 25.411$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} = 1.00$
Column location factor (22.6.5.3)	$\alpha_s = 40$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 328.634$ psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 408.066$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} = 219.089$ psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = 219.089$ psi
Shear strength reduction factor	$\phi_v = 0.75$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = 219.089$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = 164.317$ psi
	$v_{ug} / \phi v_n = 0.155$

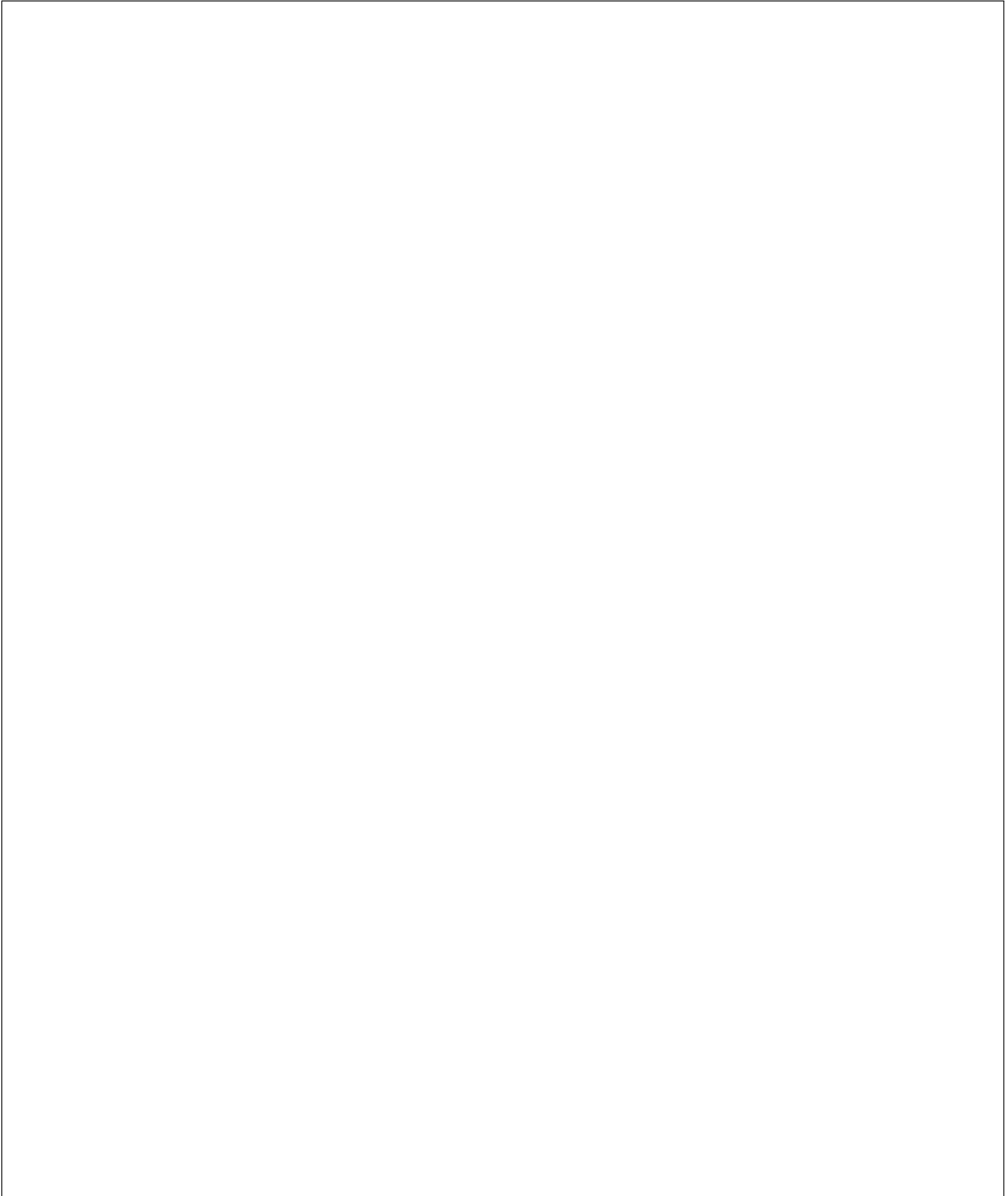
PASS - Design shear stress capacity exceeds ultimate shear stress load





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MASONRY WALL PANEL DESIGN TO MSJC-11

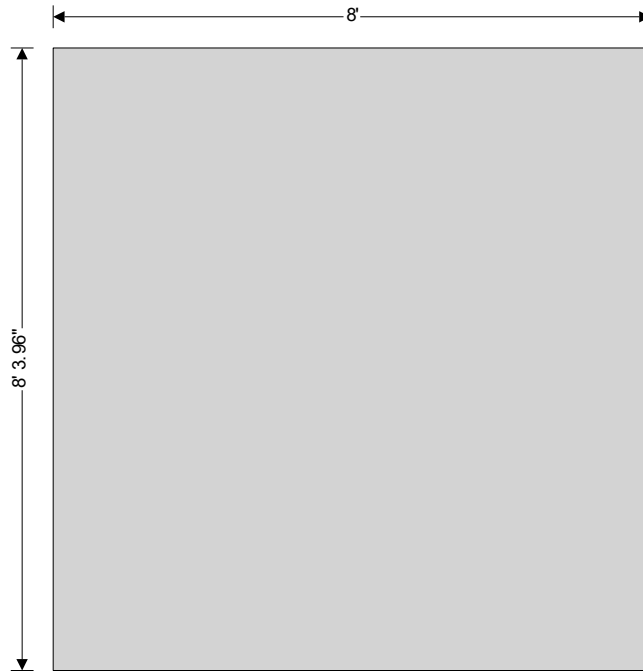
Using the allowable stress design method

Tedds calculation version 2.2.02

Masonry wall panel details

Trash Enclosure Section - Reinforced single-wythe wall, the wall is free at the top and fixed at the bottom for out of plane loads
The wall is fixed at the bottom and free at the top for in plane loads

Panel length $L = 8$ ft
Panel height $h = 8.33$ ft



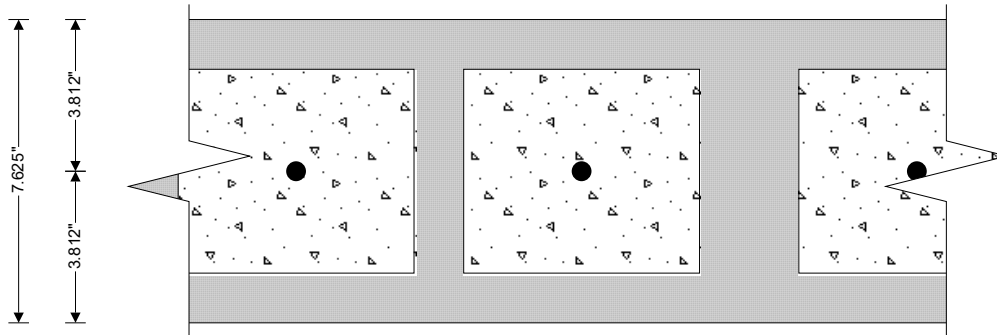
Seismic properties

Seismic design category B
 Seismic importance factor (ASCE7 Table 11.5-1) $I_e = 1$
 Design spectral response acceleration parameter, short periods (ASCE7 11.4.4) $S_{DS} = 1$
 Seismic wall classification Nonparticipating
 No prescriptive minimum seismic reinforcement
 Redundancy factor, on out-of-plane load $\rho_E = 1.0$

Construction details

Wall thickness $t = 7.625$ in

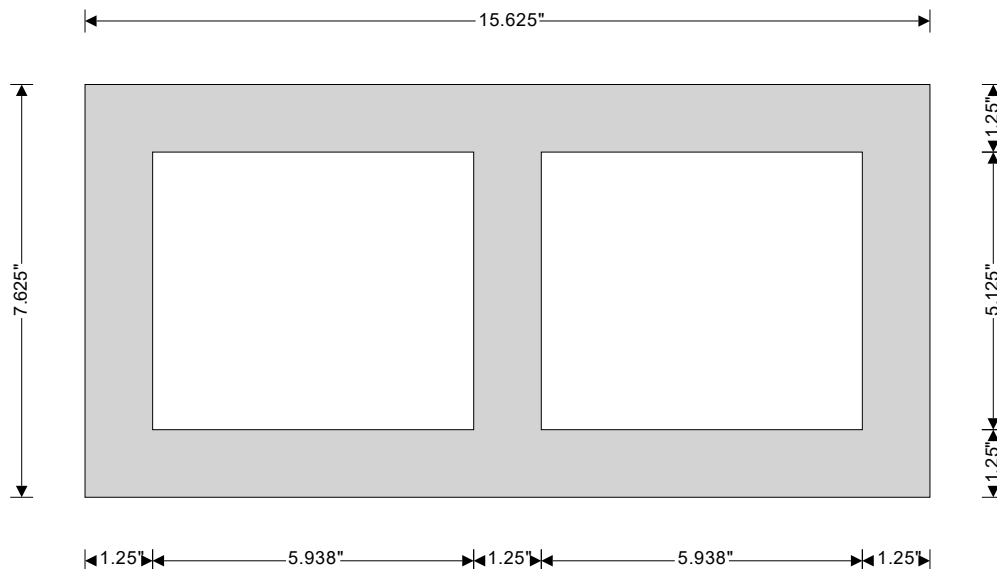
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Masonry details

Hollow concrete units fully grouted in running bond fully bedded with PCL class M mortar

Compressive strength of unit	$f_{cu} = 1900$ psi
Density of masonry units	$\gamma_{block} = 115$ lb/ft ³
Height of masonry units	$h_b = 7.625$ in
Length of masonry units	$l_b = 15.625$ in
Number of internal webs	$N_{web} = 1$
Number of end webs	$N_{end} = 2$
Internal web thickness	$t_{bw} = 1.25$ in
Face shell thickness	$t_{bf} = 1.25$ in
End web thickness	$t_{be} = 1.25$ in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = 44.76$ in ² /ft
Area of grout	$A_{grout} = [(l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = 46.74$ in ² /ft
Density of grout	$\gamma_{grout} = 140$ lb/ft ³
Self weight of wall	$WSW = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout} = 81.19$ psf





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From MSJC-11 Table 2 - Compressive strength of masonry

Net compressive strength of masonry $f_m = 1500$ psi
 Modulus of elasticity for masonry $E_m = 900 \times f_m = 1350000$ psi
 Shear modulus of masonry $G_v = 0.4 \times E_m = 540000$ psi

From MSJC-11 Table 2.2.3.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed $F_{t_norm} = 86$ psi
 Allowable flexural tensile stress parallel to bed $F_{t_para} = 106$ psi

Reinforcement details

Yield strength of reinforcement $f_y = 60000$ psi
 Allowable tensile stress in reinforcement $F_s = 32000$ psi
 Modulus of elasticity for reinforcement $E_s = 29000000$ psi
 Vertical reinforcement provided No.4 bars at 8 in centers
 Area of vertical reinforcement $A_s = \pi \times Dia^2 / (4 \times s) = 0.29$ in²/ft

Lateral out-of-plane loads

Wind load on panel $W = 30$ psf
 Wind load on parapet $W_p = 18$ psf
 Seismic load factor (ASCE7 12.11.1) $F_p = 0.4 \times S_{DS} \times I_e = 0.4$
 Seismic load from wall $E_{wall} = \max(F_p, 0.1) \times w_{sw} = 32.5$ psf
 Additional seismic load $E_{add} = 0$ psf
 Seismic lateral load on panel $E = E_{wall} + E_{add} = 32.5$ psf

Lateral in-plane loads

Wind shear load on wall $V_w = 500$ lbs

Vertical loading details

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS} = 0.2$

From IBC 2015 cl.1605.3.1 - Basic load combinations (Utilization)

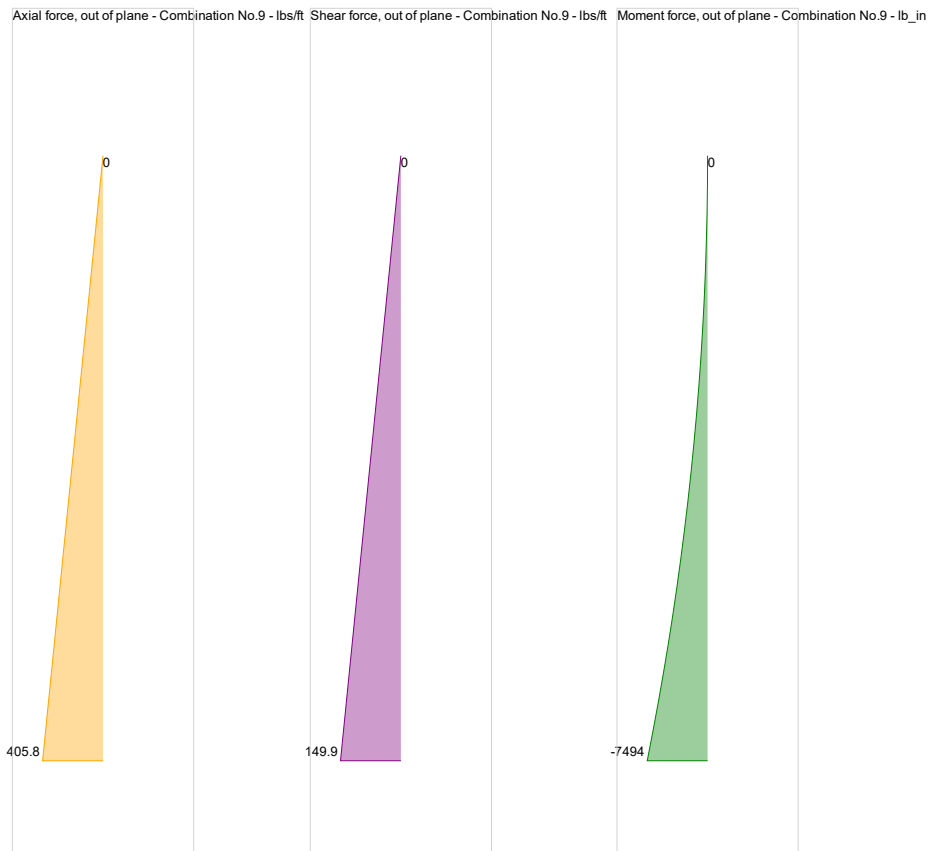
Load combination no.1 DL (0.022)
 Load combination no.5 DL + 0.6 × W (0.349)
 Load combination no.7 DL + 0.45 × W + 0.75 × LL + 0.75 × (LL_r or SL or RL) (0.262)
 Load combination no.9 0.6 × DL + 0.6 × W (0.354)

Properties of masonry section

Cross-sectional area $A = t = 91.5$ in²/ft
 Properties for walls loaded out-of-plane:
 Moment of inertia $I = t^3 / 12 = 443.3$ in⁴/ft
 Section modulus $S = I / c = 116.3$ in³/ft
 Radius of gyration $r = \sqrt{I / A} = 2.201$ in
 Effective height factor $K = 1$
 Properties for walls loaded in-plane:
 Gross moment of inertia $I_{x_gross} = t \times L^3 / 12 = 562176$ in⁴
 Gross section modulus $S_{x_gross} = I_{x_gross} / (L / 2) = 11712$ in³

Consider wall at bottom under load combination no.9

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Axial load at bottom of panel $P = 406$ lb/ft
 Compressive stress due to axial load $f_a = P / A = 4.4$ psi
 Slenderness ratio $(K \times h) / r = 45.413 < 99$
 Allowable compressive stress due to axial load $F_a = (1 / 4) \times f_m \times [1 - ((K \times h) / (140 \times r))^2] = 335.5$ psi
 $f_a / F_a = 0.013$

PASS - Allowable compressive stress exceeds compressive stress due to axial loads

Allowable compressive force $P_a = 0.25 \times f_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] = 30603$ lb/ft
 $P / P_a = 0.013$

PASS - Allowable compressive force exceeds axial load

Bending moment at bottom of panel $M = 7494$ lb_in/ft
 Depth of reinforcement $d = 3.813$ in
 Modular ratio $n = E_s / E_m = 21.481$
 Allowable compressive stress due to flexural load $F_b = (0.45) \times f_m = 675$ psi
 Balance point $k_{bal} = n / (F_s / F_b + n) = 0.312$
 Tensile strain in reinforcement $\epsilon_s = F_s / E_s = 0.001103$
 Compressive strain in masonry $\epsilon_m = \epsilon_s \times k_{bal} / (1 - k_{bal}) = 0.000500$
 Compressive stress at balance point $f_{bal} = \epsilon_m \times E_m = 675$ psi
 Tension at balance point $T_{bal} = A_s \times F_s = 9425$ lb/ft
 Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2 = 4815$ lb/ft
 Axial load at balance point $P_{bal} = C_{bal} - T_{bal} = -4610$ lb/ft
 Moment at balance point $M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3) = 16448$ lb_in/ft



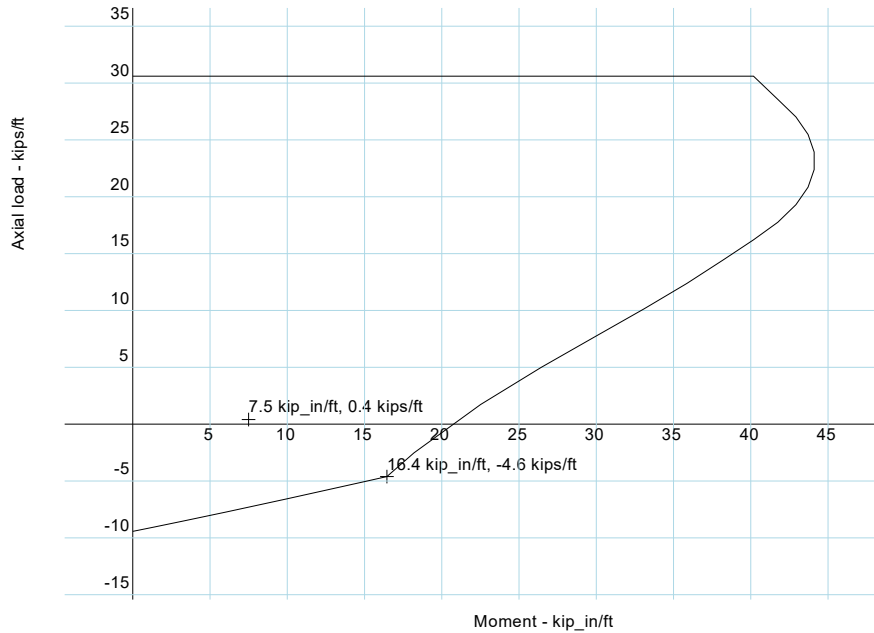
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Maximum moment from interaction diagram

$$M_c = 21175 \text{ lb_in/ft}$$

$$M / M_c = 0.354$$

PASS - Combination of applied axial load and flexure is acceptable



Allowable stress interaction diagram

Consider wall at bottom under load combination no.9

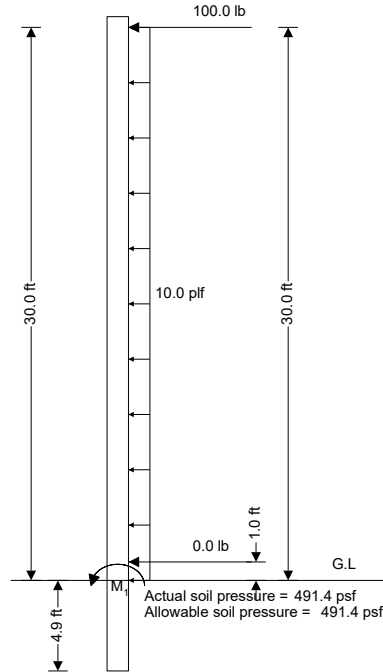
Shear force	$V = 149.9 \text{ lb/ft}$
Net shear area	$A_{nv} = d = 45.8 \text{ in}^2/\text{ft}$
Shear stress	$f_v = V / A_{nv} = 3.3 \text{ psi}$
Compressive force	$N_v = 0.60 \times P_{DL,b,out} / 1 \text{ ft} = 405.8 \text{ lb/ft}$
Moment	$M = 7494 \text{ lb_in/ft}$
Allowable masonry shear stress	$F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)), 1.0)] \times \sqrt{(f_m \times 1 \text{ psi})} + 0.25 \times N_v / A$ $= 44.7 \text{ psi}$
Allowable shear stress	$F_v = \min(F_{vm}, 2 \times \sqrt{(f_m \times 1 \text{ psi})}) = 44.7 \text{ psi}$
Shear utilization	$f_v / F_v = 0.073$

PASS - Allowable shear stress exceeds applied shear stress

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FLAGPOLE EMBEDMENT (IBC 2015)

TEDDS calculation version 1.2.00



Soil capacity data

Allowable passive pressure	$L_{sbc} = 300$ pcf
Maximum allowable passive pressure	$P_{max} = 3000$ psf
Load factor 1 (1806.1)	$LDF_1 = 1.00$
Load factor 2 (1806.3.4)	$LDF_2 = 1.0$

Pole geometry

Shape of the pole	Round
Diameter of the pole	Dia = 24 in
Laterally restrained	No

Load data

First point load	$P_1 = 100$ lbs
Distance of P ₁ from ground surface	$H_1 = 30$ ft
Second point load	$P_2 = 0$ lbs
Distance of P ₂ from ground surface	$H_2 = 1$ ft
Uniformly distributed load	$W = 10$ plf
Start distance of W from ground surface	$a = 0$ ft
End distance of W from ground surface	$a_1 = 30$ ft
Applied moment	$M_1 = 0$ lb _{ft}
Distance of M ₁ from ground surface	$H_3 = 0$ ft

Shear force and bending moment

Total shear force	$F = P_1 + P_2 + W \times (a_1 - a) = 400$ lbs
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Total bending moment at grade
 Distance of resultant lateral force

$$M_g = P_1 \times H_1 + P_2 \times H_2 + W \times (a_1 - a) \times (a + a_1) / 2 + M_1 = 7500 \text{ lb_ft}$$

$$h = \text{abs}(M_g / F) = 18.75 \text{ ft}$$

Embedment depth (1807.3.2.1)

Embedment depth provided
 Allowable lateral passive pressure
 Factor A
 Embedment depth required
 Actual lateral passive pressure

$$D = 4.91 \text{ ft}$$

$$S_1 = \min(P_{\max}, L_{\text{sbc}} \times \min(D, 12 \text{ ft}) / 3) \times \text{LDF}_1 \times \text{LDF}_2 = 491.4 \text{ psf}$$

$$A = 2.34 \times \text{abs}(F) / (S_1 \times \text{Dia}) = 1 \text{ ft}$$

$$D_1 = 0.5 \times A \times (1 + (1 + ((4.36 \times h) / A))^{0.5}) = 4.91 \text{ ft}$$

$$S_2 = (2.34 \times \text{abs}(F) \times ((4.36 \times h) + (4 \times D))) / (4 \times D^2 \times \text{Dia}) = 491.4 \text{ psf}$$