BSE STRUCTURAL ENGINEERS

Project Westlake Ace Hardware Project No. 22-365

Calc. By KMB Checked By KB Date 11/11/2022

STRUCTURAL CALCULATIONS FOR :

Westlake Ace Hardware
Lee's Summit, MO

STATE OF MISSOURI
KEVIN S. BOEHRINGER
NUMBER E-29757
PROFESSIONAL ENGINEER
12/23/22

Summary

Loads for the project referenced above were determined based on the governing International Building Code (IBC) and the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7).

All vertical/gravity loads were determined as follow: All dead loads were determined based on the building composition and all live loads were determined based on the expected occupancy for each of the spaces within the building. Snow loads were determined based on the building dimensions, the roof profile and the project location.

All lateral loads were determined as follows: All wind loading was based on the building dimensions and project location. All seismic loads were determined based on the building composition, the type of lateral stability system and the project location.

The following section of calculations covers the process used to determine the gravity and lateral loads for the project referenced above. Refer to all other sections for the application of these loads.

DEAD LOADING

DEAD LOAD CONSTRUCTION

Roof Dead Load

Material	Thickness (in)	γ (lb/ft^3)	Weight (lb/ft^2)
Roofing Material	0.000		0.5
Polystyrene	3.000	15	3.7
1.5" Type B Metal Roof Deck	1.500		1.8
Joists	0.000		3.0
Collateral/Misc.	0.000		6
Totals	**4.500**		**15.0**

LIVE LOADING

LIVE LOAD CONSTRUCTION

Roof Live Load

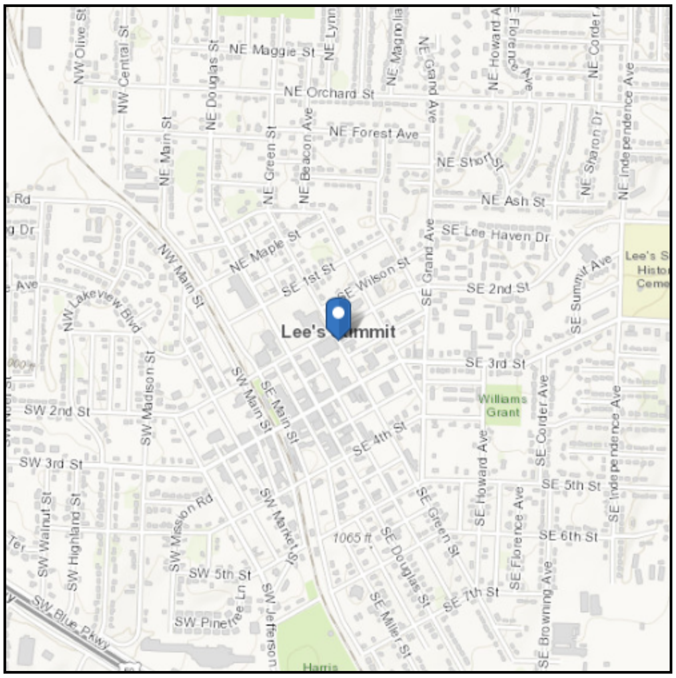
IBC 2018, Table 1607.1 **20.0**

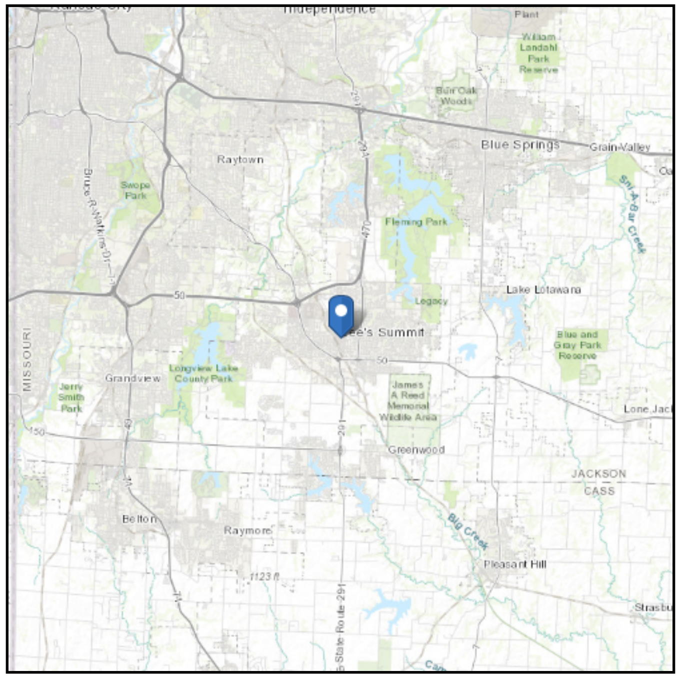
ASCE 7 Hazards Report

Address:
Lee's Summit
Missouri,

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Elevation: 1013.54 ft (NAVD 88)
Latitude: 38.91439
Longitude: -94.3755





Wind

Results:

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

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed: Thu Oct 13 2022

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

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

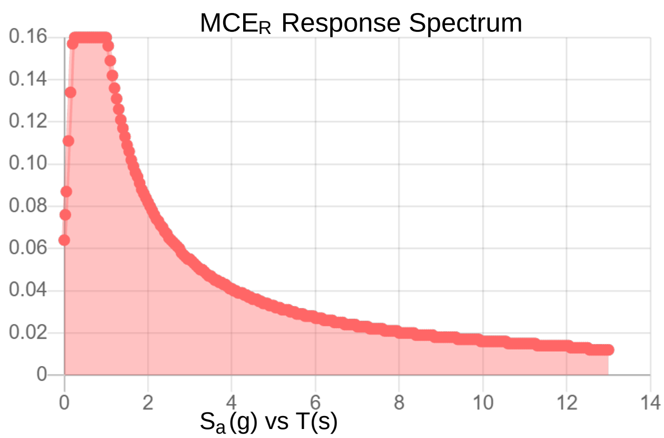
Seismic

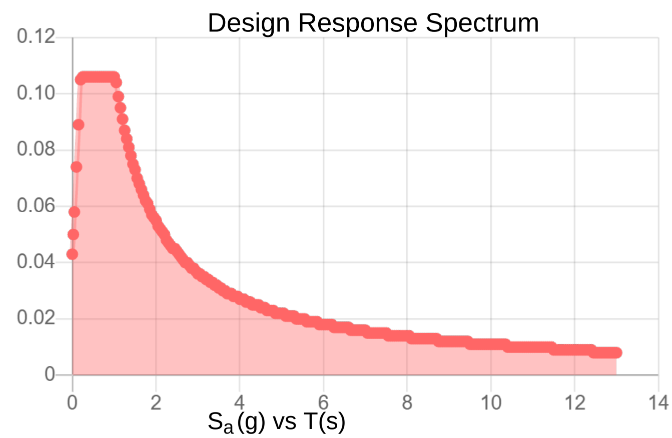
Site Soil Class: D - Default (see Section 11.4.3)

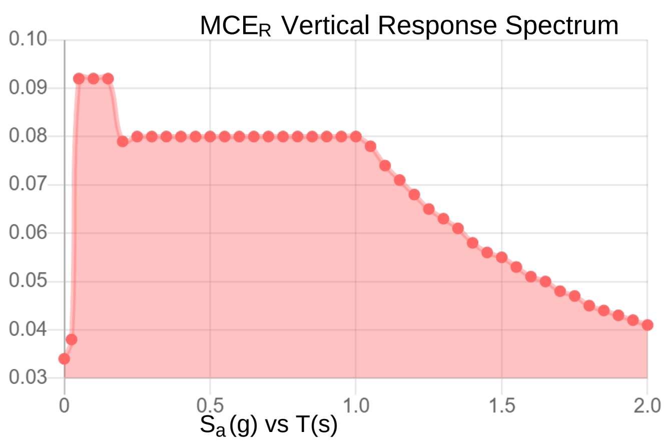
Results:

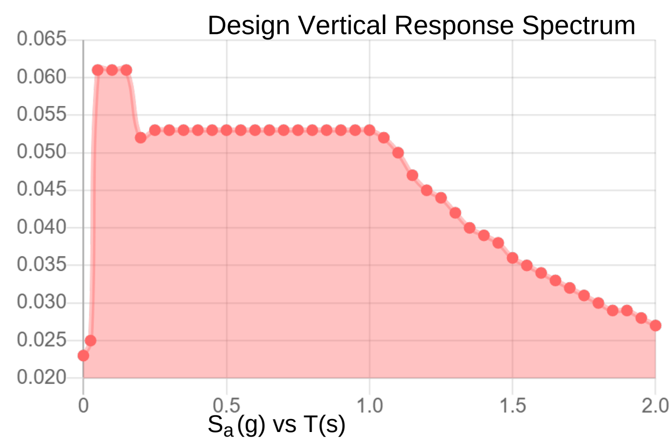
S_S :	0.1	S_{D1} :	0.109
S_1 :	0.068	T_L :	12
F_a :	1.6	PGA :	0.047
F_v :	2.4	PGA_M :	0.076
S_{MS} :	0.16	F_{PGA} :	1.6
S_{M1} :	0.164	I_e :	1
S_{DS} :	0.106	C_v :	0.7

Seismic Design Category B









Data Accessed: **Thu Oct 13 2022**

Date Source:

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

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Snow

Results:

Ground Snow Load, p_g :	20 lb/ft^2
Elevation:	1013.5 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Thu Oct 13 2022

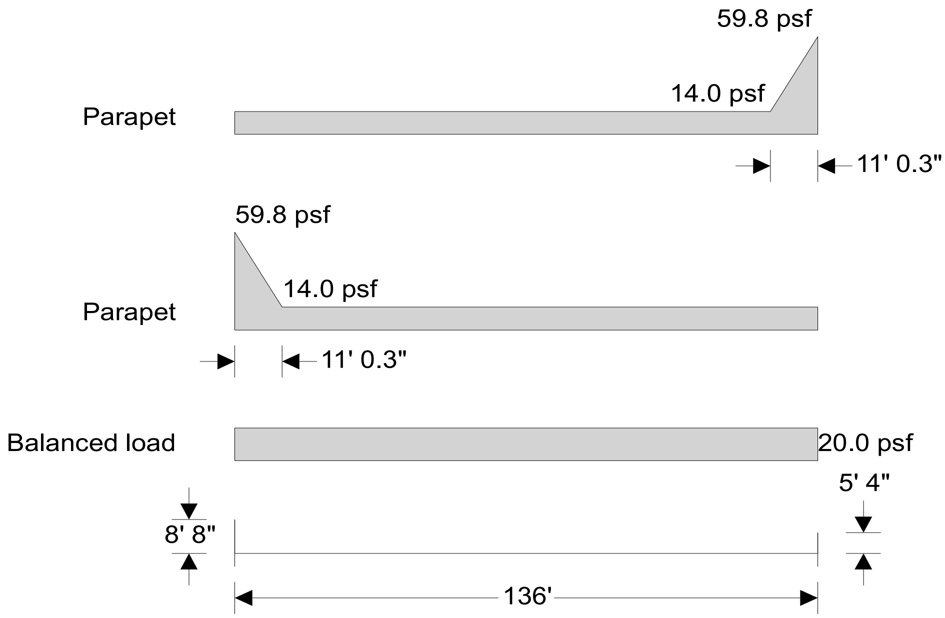
Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details	
Roof type	Flat
Width of roof	b = **136.00** ft
Ground snow load	
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal condition (Table 7.3-2)	All
Thermal factor (Table 7.3-2)	C_t = **1.00**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **14.00** lb/ft^2
Left parapet	
Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height of left parapet	h_{pptL} = **8.67** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **7.83** ft
Length of roof - left parapet	$l_{u\_pptL} = b$ = **136.00** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **2.76** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **2.76** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **11.03** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **45.76** lb/ft^2
Right parapet	
Height of right parapet	h_{pptR} = **5.33** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **4.49** ft
Length of roof - right parapet	$l_{u\_pptR} = b$ = **136.00** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **2.76** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **2.76** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **11.03** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **45.76** lb/ft^2



Roof elevation

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details

Roof type	Flat
Width of roof	b = **11.00** ft

Ground snow load

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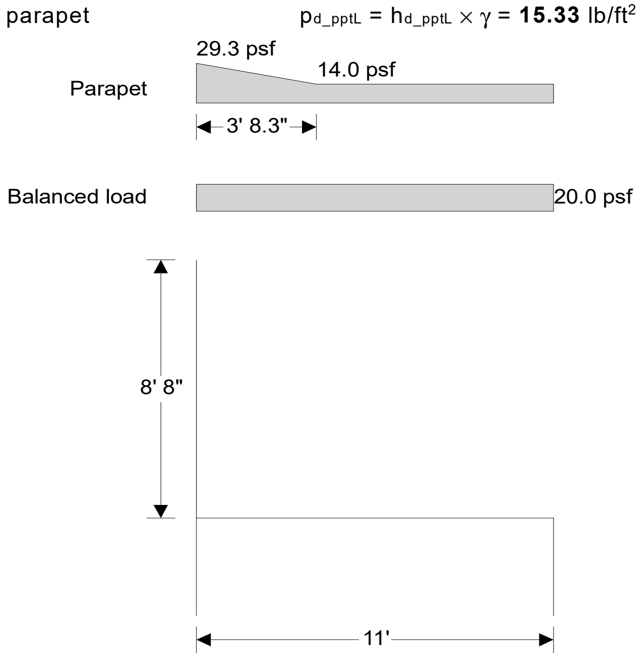
Thermal factor (Table 7.3-2)	C_t = **1.00**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times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

Left parapet

Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height of left parapet	h_{pptL} = **8.67** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **7.83** ft
Length of roof - left parapet	$l_{u\_pptL} = b$ = **11.00** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \min(\sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}), \sqrt{(I_s \times p_g \times l_{u\_pptL} / (4 \times \gamma))})$ = **0.92** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **0.92** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **3.69** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **15.33** lb/ft^2



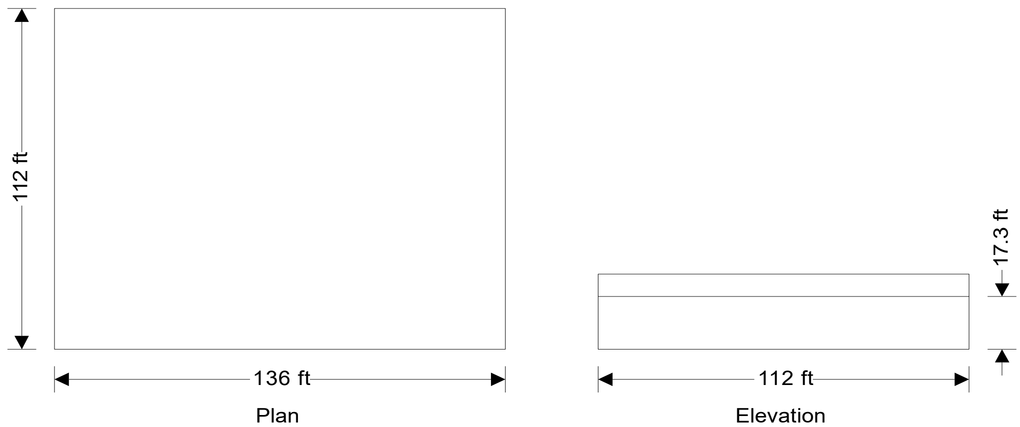
Roof elevation

WIND LOADING

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.05



Building data

Type of roof	Flat
Length of building	b = **136.00** ft
Width of building	d = **112.00** ft
Height to eaves	H = **17.34** ft
Height of parapet	h_p = **7.33** ft
Mean height	h = **17.34** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **6.94** ft
Plan length of Zone 2/2E when GC_{pf} negative	L_{Z2} = min(0.5 × d, 2.5 × H) = **43.35** ft
Plan length of Zone 3/3E encroachment on zone 2	L_{Z3} = max(0 ft,0.5 × d - L_{Z2}) = **12.65** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **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**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.87**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **22.6** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.94**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **24.2** psf

Parapet pressures and forces

Velocity pressure at top of parapet q_p = **24.23** psf

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

Combined net parapet pressure, leeward $p_{pl} = q_p \times GC_{pnl}$ = **-24.23** psf

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

Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw}$ = **36.35** psf

Wind direction 0 deg (|| to width):

Leeward parapet force $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-24.2** kips

Windward parapet force $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **36.2** kips

Wind direction 90 deg (|| to length):

Leeward parapet force $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-19.9** kips

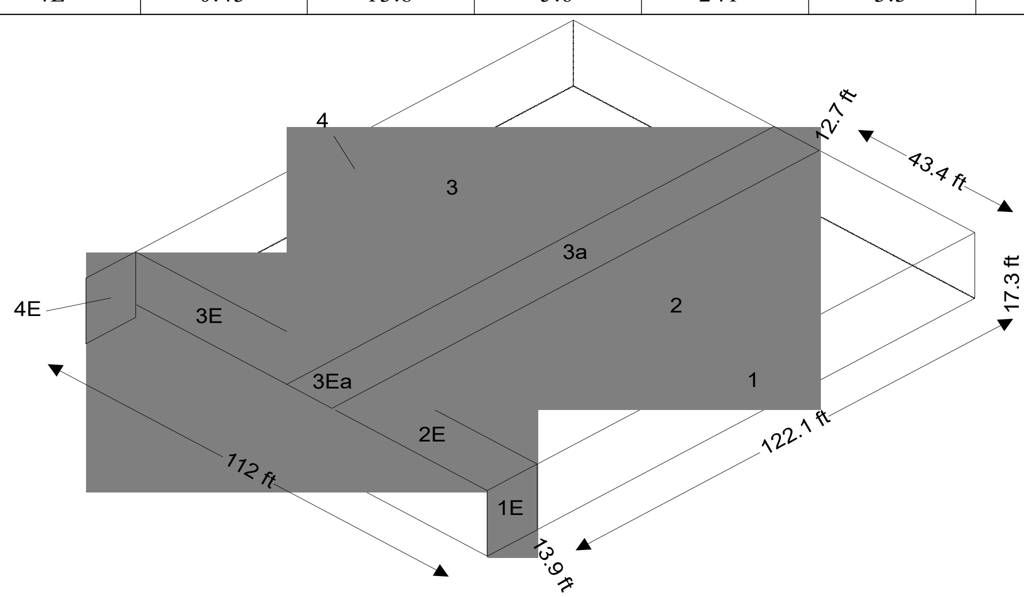
Windward parapet force $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **29.8** kips

Design wind pressures

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

Design wind pressures – Loadcase A

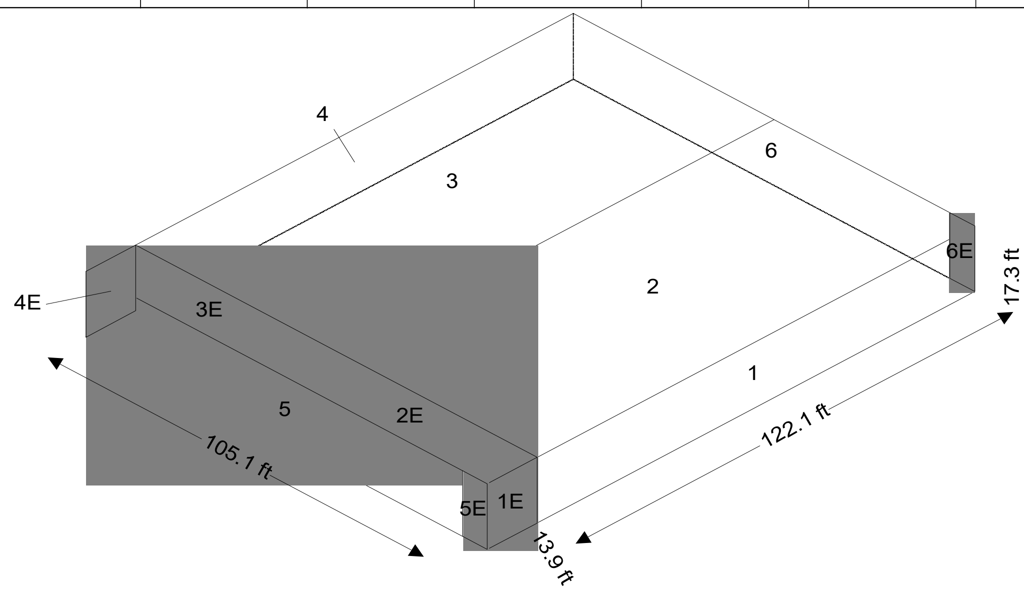
Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft²)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.0	13.1	2118	10.5	27.7
2	-0.69	-19.6	-11.5	5294	-104.0	-61.0
3a	-0.37	-12.4	-4.3	1545	-19.2	-6.6
3	-0.37	-12.4	-4.3	6839	-84.9	-29.3
4	-0.29	-10.6	-2.5	2118	-22.5	-5.3
1E	0.61	9.7	17.8	241	2.3	4.3
2E	-1.07	-28.2	-20.1	601	-17.0	-12.1
3Ea	-0.53	-16.0	-7.9	175	-2.8	-1.4
3E	-0.53	-16.0	-7.9	777	-12.5	-6.1
4E	-0.43	-13.8	-5.6	241	-3.3	-1.4



Loadcase A

Design wind pressures – Loadcase B

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-14.2	-6.1	2118	-30.1	-12.9
2	-0.69	-19.6	-11.5	6839	-134.4	-78.8
3	-0.37	-12.4	-4.3	6839	-84.9	-29.3
4	-0.45	-14.2	-6.1	2118	-30.1	-12.9
5	0.40	5.0	13.1	1822	9.1	23.9
6	-0.29	-10.6	-2.5	1822	-19.3	-4.5
1E	-0.48	-14.9	-6.8	241	-3.6	-1.6
2E	-1.07	-28.2	-20.1	777	-21.9	-15.6
3E	-0.53	-16.0	-7.9	777	-12.5	-6.1
4E	-0.48	-14.9	-6.8	241	-3.6	-1.6
5E	0.61	9.7	17.8	120	1.2	2.1
6E	-0.43	-13.8	-5.6	120	-1.7	-0.7

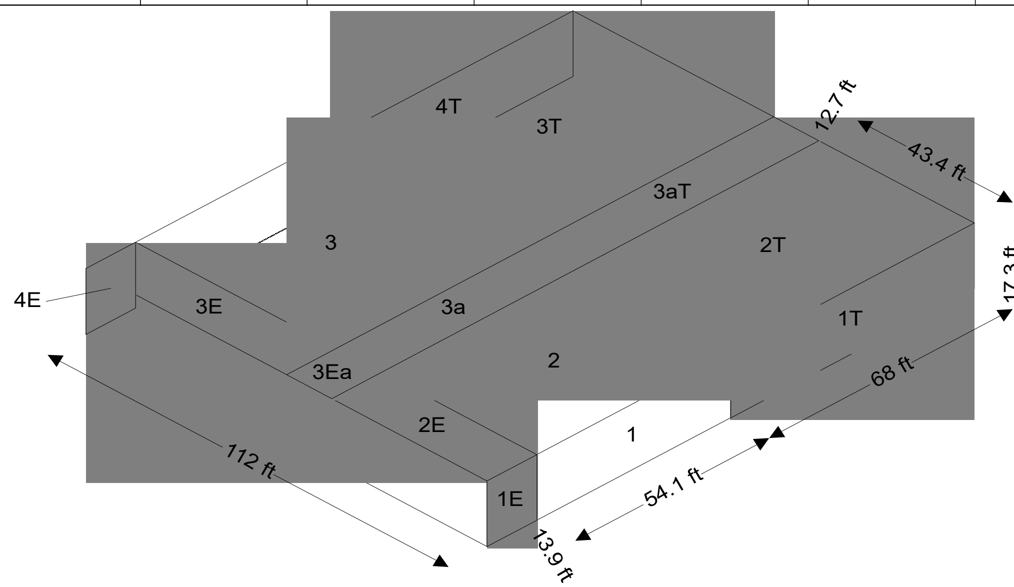


Loadcase B

Design wind pressures – Loadcase AT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	5.0	13.1	939	4.7	12.3
2	-0.69	-19.6	-11.5	2346	-46.1	-27.0
3a	-0.37	-12.4	-4.3	685	-8.5	-2.9
3	-0.37	-12.4	-4.3	3031	-37.6	-13.0
4	-0.29	-10.6	-2.5	939	-10.0	-2.3
1E	0.61	9.7	17.8	241	2.3	4.3
2E	-1.07	-28.2	-20.1	601	-17.0	-12.1

3Ea	-0.53	-16.0	-7.9	175	-2.8	-1.4
3E	-0.53	-16.0	-7.9	777	-12.5	-6.1
4E	-0.43	-13.8	-5.6	241	-3.3	-1.4
1T	-	1.2	3.3	1179	1.5	3.9
2T	-	-4.9	-2.9	2948	-14.5	-8.5
3Ta	-0.09	-6.2	2.0	860	-5.3	1.7
3T	-	-3.1	-1.1	3808	-11.8	-4.1
4T	-	-2.7	-0.6	1179	-3.1	-0.7

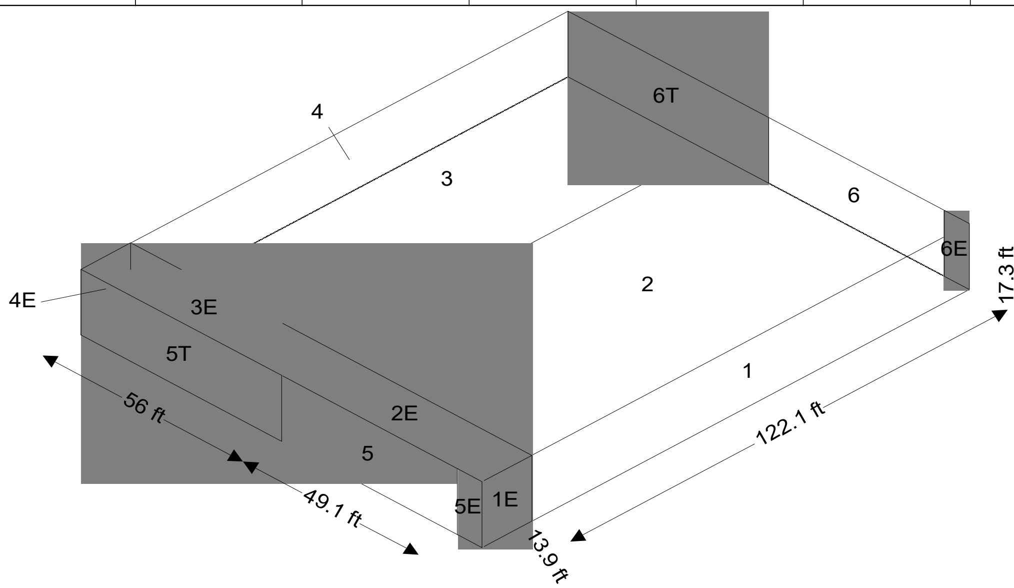


Loadcase AT

Design wind pressures – Loadcase BT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-14.2	-6.1	2118	-30.1	-12.9
2	-0.69	-19.6	-11.5	6839	-134.4	-78.8
3	-0.37	-12.4	-4.3	6839	-84.9	-29.3
4	-0.45	-14.2	-6.1	2118	-30.1	-12.9
5	0.40	5.0	13.1	911	4.5	11.9
6	-0.29	-10.6	-2.5	911	-9.7	-2.3
1E	-0.48	-14.9	-6.8	241	-3.6	-1.6
2E	-1.07	-28.2	-20.1	777	-21.9	-15.6
3E	-0.53	-16.0	-7.9	777	-12.5	-6.1
4E	-0.48	-14.9	-6.8	241	-3.6	-1.6
5E	0.61	9.7	17.8	60	0.6	1.1
6E	-0.43	-13.8	-5.6	120	-1.7	-0.7
5T	-	1.2	3.3	971	1.2	3.2

6T	-	-2.7	-0.6	971	-2.6	-0.6



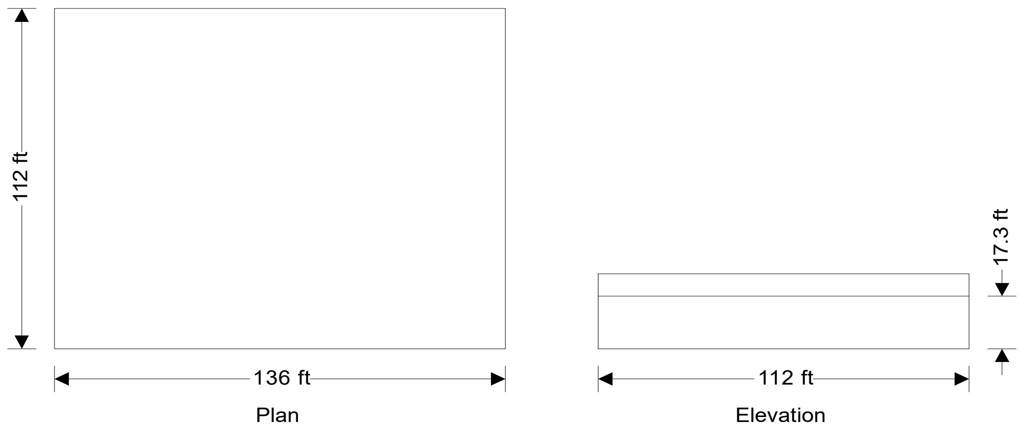
Loadcase BT

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.05



Building data

Type of roof	Flat
Length of building	b = **136.00** ft
Width of building	d = **112.00** ft
Height to eaves	H = **17.34** ft
Height of parapet	h_p = **7.33** ft
Mean height	h = **17.34** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **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**
Parapet internal pressure coef +ve (Table 26.11-1)	GC_{pi_pp} = **0.18**
Parapet internal pressure coef –ve (Table 26.11-1)	GC_{pi_np} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.87**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **22.6** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.94**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **24.2** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **22.58** psf

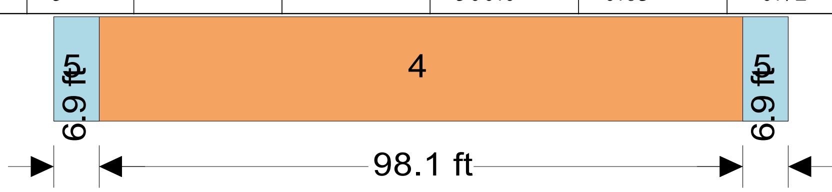
Equations used in tables

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

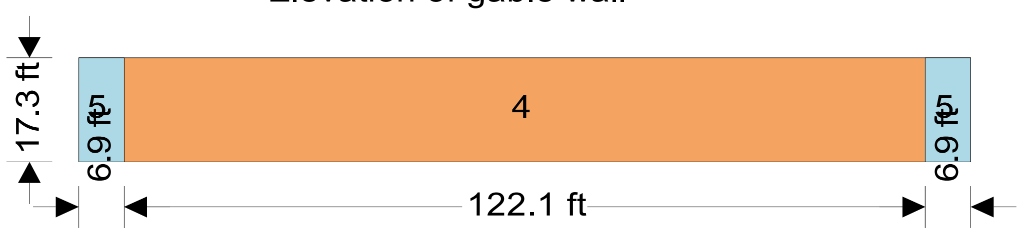
Parapet net pressure $p = q_p \times [GC_p - GC_{pi\_p}]$

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

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	4	-	-	10.0	0.90	-0.99	24.4	-26.4
20sf	4	-	-	20.0	0.85	-0.94	23.3	-25.3
50sf	4	-	-	50.0	0.79	-0.88	21.9	-23.9
>100sf	4	-	-	100.0	0.74	-0.83	20.8	-22.8
>500sf	4	-	-	500.0	0.63	-0.72	18.3	-20.3
<10sf	5	-	-	10.0	0.90	-1.26	24.4	-32.5
20	5	-	-	20.0	0.85	-1.16	23.3	-30.4
50sf	5	-	-	50.0	0.79	-1.04	21.9	-27.5
>100sf	5	-	-	100.0	0.74	-0.94	20.8	-25.3
>500sf	5	-	-	500.0	0.63	-0.72	18.3	-20.3



Elevation of gable wall



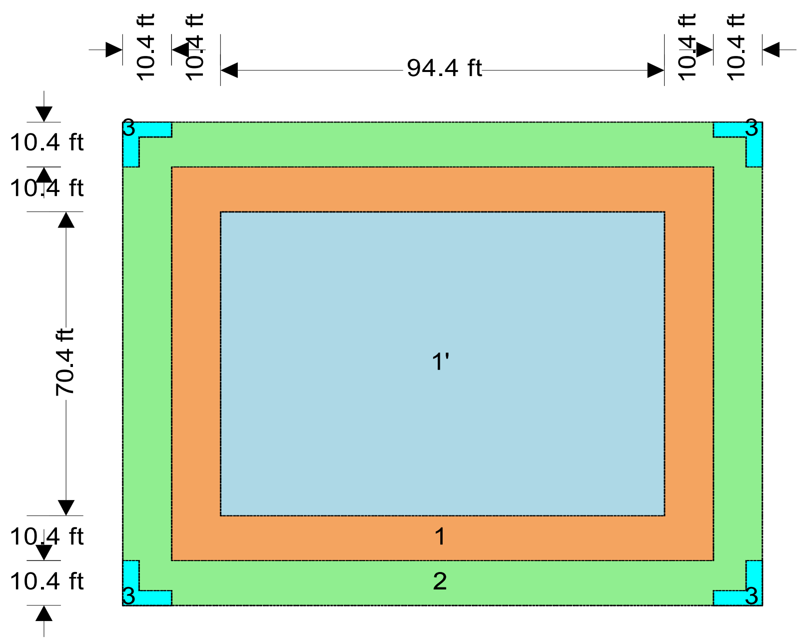
Elevation of side wall

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

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	1	-	-	10.0	0.30	-1.70	10.8 #	-42.5
20 sf	1	-	-	20.0	0.27	-1.58	10.2 #	-39.6
50 sf	1	-	-	50.0	0.23	-1.41	9.3 #	-35.9
>100 sf	1	-	-	100.0	0.20	-1.29	8.6 #	-33.1
<10 sf	2	-	-	10.0	0.90	-2.30	24.4	-56.0
20 sf	2	-	-	20.0	0.85	-2.14	23.3	-52.4

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
50 sf	2	-	-	50.0	0.79	-1.93	21.9	-47.6
>100 sf	2	-	-	100.0	0.74	-1.77	20.8	-44.0
<10 sf	3	-	-	10.0	0.90	-2.30	24.4	-56.0
20 sf	3	-	-	20.0	0.85	-2.14	23.3	-52.4
50 sf	3	-	-	50.0	0.79	-1.93	21.9	-47.6
>100 sf	3	-	-	100.0	0.74	-1.77	20.8	-44.0
<10 sf	1'	-	-	10.0	0.30	-0.90	10.8 #	-24.4
20 sf	1'	-	-	20.0	0.27	-0.90	10.2 #	-24.4
50 sf	1'	-	-	50.0	0.23	-0.90	9.3 #	-24.4
>100 sf	1'	-	-	100.0	0.20	-0.90	8.6 #	-24.4

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



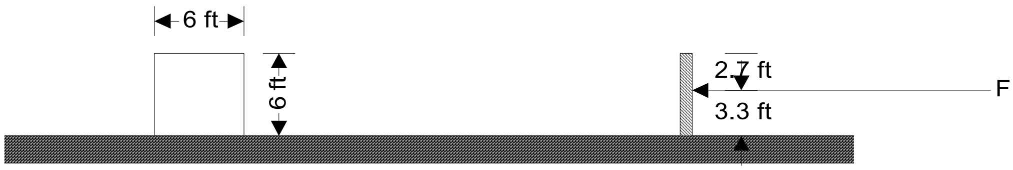
Plan on roof

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.05



Wall/sign data

Length of wall/sign	B = **6.00** ft
Height of wall/sign	s = **6.00** ft
Height to top of sign	h = **6.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	I
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** m
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$= **1.00**
Exposure category (cl 26.7.3)	C
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.4.7)	p_{min_r} = **8** lb/ft^2

Topography

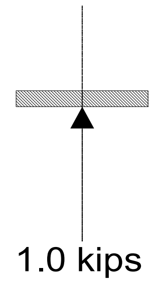
Topography factor not significant	K_{zt} = 1.0

Velocity pressure

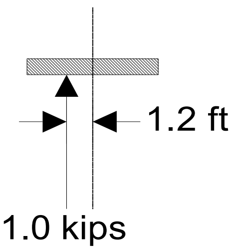
Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **22.0** psf
Area of sign	$A_f = B \times s$ = **36** ft^2
Ratio of solid area to gross area	ε = **1.00**

Wall/sign forces – Case A and B

Force coefficient (Figure 29.3-1)	C_{f_A} = **1.45**
Resultant force	$F_A = \max(16\text{psf}, q_h \times G_f \times C_{f\_A}) \times A_f$ = **1.0** kips



Plan - Case A



Plan - Case B

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.05

Wall/sign data

Length of wall/sign	B = **12.00** ft
Height of wall/sign	s = **8.00** ft
Height to top of sign	h = **8.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	I
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** m
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$= **1.00**
Exposure category (cl 26.7.3)	C
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.4.7)	p_{min_r} = **8** lb/ft^2

Topography

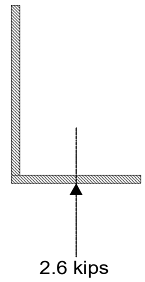
Topography factor not significant	K_{zt} = 1.0

Velocity pressure

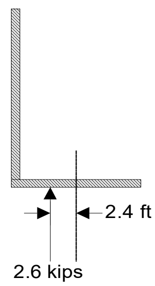
Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **22.0** psf
Area of sign	$A_f = B \times s$ = **96** ft^2
Ratio of solid area to gross area	ε = **1.00**

Wall/sign forces – Case A and B

Force coefficient (Figure 29.3-1)	C_{f_A} = **1.42**
Resultant force	$F_A = \max(16\text{psf}, q_h \times G_f \times C_{f\_A}) \times A_f$ = **2.6** kips



Plan - Case A



Plan - Case B

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 = **0.1**
at 1 sec period	S_1 = **0.068**
Site coefficientat short period (Table 11.4-1)	F_a = **1.600**
at 1 sec period (Table 11.4-2)	F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S$ = **0.160**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1$ = **0.163**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS}$ = **0.107**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1}$ = **0.109**

Seismic design category

Occupancy category (Table 1-1) II

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

A

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

B

Seismic design category B

Approximate fundamental period

Height above base to highest level of building h_n = **24** 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)	$T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$ = **0.217** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.217** sec
Long-period transition period	T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems
	9. Ordinary reinforced masonry shear walls
Response modification factor (Table 12.2-1)	R = **2**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-3)	$C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0533**
Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e))$ = **0.2508**
Minimum (Eq 9.5.5.2.1-3)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0100**
Seismic response coefficient	C_s = **0.0533**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure	W = **100.0** kips
Seismic response coefficient	C_s = **0.0533**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **5.3** kips

Summary

The gravity structure system of the project referenced above consists primarily of steel joist and beams, steel columns, and load bearing masonry walls. A new trash enclosure made of 8" masonry walls and a monument sign are also provided on the site. The locations of new framing and walls are indicated on the structural framing plans and within the attached sketches.

The following section of calculations covers the complete framing design including the design of headers. Refer to the "Loads" section of these calculations for the determination of all dead, live, roof live, and snow loads. Refer to the "Foundation" section of these calculations for the design of the continuous footings supporting the gravity framing.

Joist and Girder Designations

Joist Designation - RJ1

Dead Load =	15.00 psf	
Roof Live Load =	20.00 psf	
Snow Load =	20.00 psf	
ASD Wind (Uplift) =	33.10 psf	

Live Load Reduction

Total Trib Area =	178 ft^2	
F=	0.25	
R1 =	1	IBC Equation 16-26 thru 16-28
R2=	1	IBC Equation 16-29 thru 16-31
Reduced Live =	20.00 psf	IBC Equation 16-25

Joist Length =	29.67 ft.	
Joist Spacing =	6.00 ft.	
Joist Depth =	16 in.	(Per Vulcraft Catalog)

ASD Total Load =	210 plf
ASD Live/Snow Load =	120 plf

Joist Designation = 16K / 210 / 120 *

* = Snow Drift (if req'd). Refer to drift calculations for additonal loading

Wind Design Uplift = W - 0.6D (5 psf min.) = 24.1 psf

Joist and Girder Designations

Joist Designation - RJ2

Dead Load =	15.00 psf
Roof Live Load =	20.00 psf
Snow Load (Flat) =	14.00 psf
Wind (Uplift) =	33.10 psf

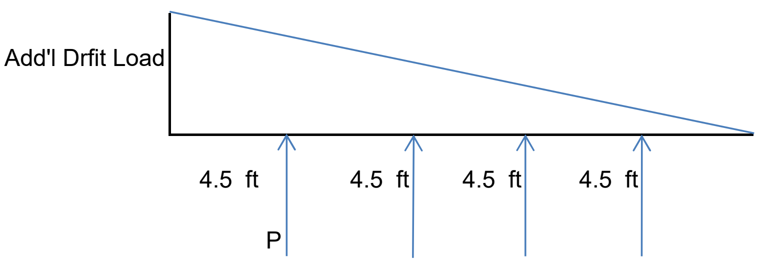
Live Load Reduction

Total Trib Area =	134 ft^2	
F=	0.25	
R1 =	1	IBC Equation 16-26 thru 16-28
R2=	1	IBC Equation 16-29 thru 16-31
Reduced Live =	20.00 psf	IBC Equation 16-25

Joist Length =	29.67 ft.	
Joist Spacing =	4.50 ft.	
Joist Depth =	16 in.	(Per Vulcraft Catalog)

Additional Drift Load

Additional Drift Load =	45.80 psf	(Total drift load minus flat snow)
Drift Length =	11.03 ft.	



P= (Add'l Drift / Drift Length)*(Drfit Length - Joist Spacing)*Joist Spacing = 27.11 psf

ASD Total Load =	252 plf
ASD Live/Snow Load =	185 plf

Joist Designation = 16K / 252 / 185 *

* = Snow Drift (if req'd). Refer to drift calculations for additonal loading

Wind Design Uplift = W - 0.6D (psf min.) = 24.10 psf

Joist and Girder Designations

Joist Designation - RJ3

Dead Load =	15.00 psf
Roof Live Load =	20.00 psf
Snow Load =	39.03 psf
ASD Wind (Uplift) =	33.10 psf

Live Load Reduction

Total Trib Area =	177 ft^2	
F=	0.25	
R1 =	1	IBC Equation 16-26 thru 16-28
R2=	1	IBC Equation 16-29 thru 16-31
Reduced Live =	20.00 psf	IBC Equation 16-25

Joist Length =	34.17 ft.	
Joist Spacing =	5.17 ft.	
Joist Depth =	18 in.	(Per Vulcraft Catalog)

ASD Total Load =	279 plf
ASD Live/Snow Load =	202 plf

Joist Designation = 18K / 279 / 202 *

* = Snow Drift (if req'd). Refer to drift calculations for additonal loading

Wind Design Uplift = W - 0.6D (5 psf min.) = 24.1 psf

Joist and Girder Designations

Joist Designation - RJ4

Dead Load =	15.00 psf	
Roof Live Load =	20.00 psf	
Snow Load =	39.03 psf	
ASD Wind (Uplift) =	33.10 psf	

Live Load Reduction

Total Trib Area =	116 ft^2	
F=	0.25	
R1 =	1	IBC Equation 16-26 thru 16-28
R2=	1	IBC Equation 16-29 thru 16-31
Reduced Live =	20.00 psf	IBC Equation 16-25

Joist Length =	22.50 ft.	
Joist Spacing =	5.17 ft.	
Joist Depth =	16 in.	(Per Vulcraft Catalog)

ASD Total Load =	279 plf
ASD Live/Snow Load =	202 plf

Joist Designation = 16K / 279 / 202 *

* = Snow Drift (if req'd). Refer to drift calculations for additonal loading

Wind Design Uplift = W - 0.6D (5 psf min.) = 24.1 psf

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
21' LENGTH (continued)						
18LH05	949	870	1424	12.9	5	1/0/0
20LH05	990	879	1485	12.5	5	1/0/0
20LH06	1146	1146	1719	14.2	6	1/0/0
20LH07	1327	1239	1991	16.4	6	1/0/0
18LH08	1373	1105	2060	17.7	6	1/0/0
20LH08	1442	1282	2163	17.5	6	1/0/0
18LH09	1562	1180	2343	19.5	7	1/0/0
18LH10	1752	1565	2628	21.7	7	1/0/0
18LH11	1955	1728	2933	25.4	7	1/0/0
20LH11	2141	2141	3212	26.1	7	1/0/0
20LH12	2367	2367	3551	28.5	8	1/0/0
18LH13	2587	2220	3881	31.7	8	1/0/0
20LH13	2847	2798	4271	32.1	8	1/0/0
22' LENGTH						
12K1	199	106	299	5.2	4	2/0/0
14K1	234	147	351	5.3	4	2/0/0
14K3	293	184	440	5.7	4	2/0/0
16K3	337	247	506	6.1	4	2/0/0
18K3	382	316	573	6.5	4	2/0/0
16K4	406	289	609	7.4	5	2/0/0
16K5	458	323	687	8.1	5	2/0/0
18K9	550	438	825	8	5	1/0/0
18LH03	675	579	1013	10.5	5	2/0/0
20LH03	715	560	1073	10.6	5	2/0/0
18LH05	886	755	1329	13.3	5	1/0/0
20LH06	1071	1007	1607	14.1	6	1/0/0
18LH07	1130	919	1695	16	6	1/0/0
20LH07	1236	1076	1854	16.1	6	1/0/0
20LH09	1524	1212	2286	19.5	7	1/0/0
18LH10	1624	1358	2436	21.4	7	1/0/0
20LH10	1645	1307	2468	21	7	1/0/0
20LH11	1988	1885	2982	25.4	7	1/0/0
18LH12	2003	1664	3005	28.9	8	1/0/0
18LH13	2391	1926	3587	31.4	8	1/0/0
20LH13	2636	2428	3954	31.7	8	1/0/0
20LH14	2879	2640	4319	37.2	8	1/0/0
23' LENGTH						
14K1	214	128	321	5.4	4	2/0/0
14K3	268	160	402	5.7	4	2/0/0
18K3	349	276	524	6.5	4	2/0/0
16K4	371	252	557	6.9	4	2/0/0
20K3	389	344	584	7	5	2/0/0
16K5	418	282	627	8	5	2/0/0

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
23' LENGTH (continued)						
20K7	550	468	825	8.2	5	1/0/0
18LH02	567	454	851	9.5	5	2/0/0
20LH02	587	444	881	9.4	5	2/0/0
20LH03	670	489	1005	10.2	5	2/0/0
18LH04	732	585	1098	11.9	5	2/0/0
18LH05	829	659	1244	13.5	5	2/0/0
20LH05	865	666	1298	13	5	2/0/0
18LH06	990	764	1485	15.4	6	1/0/0
18LH07	1053	803	1580	16.3	6	1/0/0
20LH07	1154	939	1731	16.4	6	1/0/0
20LH09	1418	1058	2127	19.4	7	1/0/0
18LH10	1508	1186	2262	21.4	7	1/0/0
20LH10	1530	1141	2295	20.9	7	1/0/0
18LH11	1683	1310	2525	25.6	7	1/0/0
18LH12	1860	1454	2790	28.4	8	1/0/0
20LH12	2045	1828	3068	28.9	8	1/0/0
18LH13	2216	1683	3324	31.3	8	1/0/0
18LH14	2420	1829	3630	37.1	8	1/0/0
20LH14	2672	2306	4008	36.8	8	1/0/0
20LH15	2895	2507	4343	40	9	1/0/0
24' LENGTH						
12K3	208	101	312	5.6	4	2/0/0
16K3	283	189	425	5.9	4	2/0/0
18K3	320	242	480	6.5	4	2/0/0
16K4	340	221	510	7	4	2/0/0
18K4	385	284	578	7.6	5	2/0/0
22K11	550	495	825	8.2	11	1/0/0
18LH03	591	444	887	10.5	5	2/0/0
20LH03	629	430	944	10	5	2/0/0
20LH04	700	546	1050	11	5	2/0/0
24LH05	827	827	1241	11.5	5	2/0/0
20LH06	942	773	1413	14.3	5	1/0/0
24LH07	1214	1214	1821	15.9	6	1/0/0
24LH08	1287	1287	1931	17.2	6	1/0/0
20LH09	1322	930	1983	19.6	7	1/0/0
20LH10	1426	1003	2139	21.2	7	1/0/0
24LH10	1784	1716	2676	22.5	7	1/0/0
24LH11	1948	1796	2922	25	7	1/0/0
24LH12	2235	2235	3353	27.7	8	1/0/0
20LH13	2276	1863	3414	31.9	8	1/0/0
24LH13	2688	2688	4032	33.2	8	1/0/0
24LH14	2936	2936	4404	36.9	8	1/0/0

28' LENGTH (continued)

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
24LH08	1008	865	1512	17	6	2/0/0
28LH08	1036	1036	1554	16.3	6	2/0/0
24LH10	1382	1074	2073	21.5	7	1/0/0
28LH10	1438	1438	2157	21.5	7	1/0/0
24LH11	1511	1124	2267	24	7	1/0/0
28LH11	1570	1570	2355	23.2	7	1/0/0
28LH12	1839	1839	2759	26.9	7	1/0/0
28LH13	2034	1944	3051	29.7	8	1/0/0
24LH13	2069	1733	3104	32.6	8	1/0/0
24LH14	2260	1885	3390	35.9	8	1/0/0
28LH14	2459	2459	3689	35.7	8	1/0/0
28LH15	2665	2665	3998	39	9	1/0/0
24LH16	2835	2263	4253	43.3	9	1/0/0

29' LENGTH

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
16K3	193	106	290	5.9	4	3/0/0
16K4	232	124	348	6.5	4	2/0/0
18K4	263	159	395	7.1	4	2/0/0
24K4	355	290	533	7.5	5	2/0/0
22K6	398	295	597	8.2	5	2/0/0
28K7	550	522	825	9.3	5	1/0/0
24LH05	621	499	932	11.5	5	2/0/0
24LH06	782	666	1173	14.4	5	2/0/0
28LH07	920	920	1380	14.9	6	2/0/0
24LH08	951	778	1427	17.1	6	2/0/0
24LH09	1216	910	1824	21.3	7	2/0/0
28LH09	1271	1271	1907	20.2	7	2/0/0
24LH10	1302	965	1953	22.6	7	2/0/0
24LH11	1423	1011	2135	24.4	7	1/0/0
28LH11	1487	1487	2231	22.9	7	1/0/0
24LH12	1633	1338	2450	28.8	8	1/0/0
28LH13	1922	1747	2883	29.5	8	1/0/0
24LH13	1946	1558	2919	33.3	8	1/0/0
24LH14	2126	1695	3189	36.1	8	1/0/0
28LH14	2324	2324	3486	36.2	8	1/0/0
28LH15	2518	2518	3777	39.1	8	1/0/0
28LH16	2962	2839	4443	44.1	9	1/0/0

30' LENGTH

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
16K4	217	112	326	6.4	4	2/0/0
20K3	227	153	341	6.6	5	3/0/0
16K5	244	126	366	7.3	4	2/0/0
16K6	266	137	399	7.8	5	2/0/0
24K6	406	319	609	8.2	5	2/0/0
24K7	453	353	680	8.9	5	2/0/0

30' LENGTH (continued)

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
30K11	550	543	825	9.5	13	1/0/0
24LH05	588	450	882	11.4	5	2/0/0
24LH06	741	601	1112	14.2	5	2/0/0
28LH06	755	755	1133	13.3	5	2/0/0
28LH07	874	874	1311	15.1	6	2/0/0
24LH08	899	702	1349	16.8	6	2/0/0
20LH10	924	585	1386	20.1	7	2/0/0
24LH09	1144	821	1716	20.7	7	2/0/0
28LH10	1291	1291	1937	20.9	7	2/0/0
28LH11	1409	1380	2114	22.6	7	1/0/0
24LH12	1540	1207	2310	27.8	8	1/0/0
28LH12	1651	1510	2477	26.8	7	1/0/0
28LH13	1818	1576	2727	29.7	8	1/0/0
24LH14	2002	1529	3003	36	8	1/0/0
28LH14	2199	2130	3299	35.7	8	1/0/0
28LH15	2383	2317	3575	39.1	8	1/0/0
28LH16	2798	2561	4197	44.3	9	1/0/0
24LH17	2819	2094	4229	49.2	9	1/0/0
20LH19	2965	1759	4448	62	11	1/0/0

31' LENGTH

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
20K3	213	138	320	6.6	5	3/0/0
16K5	228	114	342	7.2	4	3/0/0
20K6	314	198	471	8	5	2/0/0
26K5	379	314	569	8.3	5	2/0/0
24K7	424	320	636	8.8	5	2/0/0
28K8	550	480	825	10	5	2/0/0
18LH06	566	307	849	14.3	5	2/0/0
20LH06	635	386	953	14.9	6	2/0/0
24LH07	800	598	1200	15.6	6	2/0/0
24LH08	850	635	1275	17	6	2/0/0
20LH10	894	545	1341	20.1	7	2/0/0
28LH09	1143	1074	1715	19.9	7	2/0/0
28LH10	1225	1169	1838	20.8	7	2/0/0
28LH11	1337	1249	2006	22.4	7	2/0/0
24LH12	1455	1093	2183	27.5	7	1/0/0
28LH12	1566	1367	2349	26.7	7	1/0/0
28LH13	1722	1427	2583	29.5	8	1/0/0
24LH14	1889	1384	2834	35.4	8	1/0/0
28LH14	2083	1928	3125	35.6	8	1/0/0
28LH15	2257	2098	3386	38.6	8	1/0/0
28LH16	2647	2319	3971	43.7	9	1/0/0
20LH19	2775	1592	4163	61.7	11	1/0/0

ECONOMICAL JOIST GUIDE

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
32' LENGTH						
16K2	142	71	213	5.6	4	2/0/1
16K3	158	79	237	5.9	4	2/0/1
20K4	240	147	360	6.9	5	3/0/0
22K5	299	201	449	7.8	5	3/0/0
20K7	328	199	492	8.4	5	2/0/0
28K7	466	400	699	9.3	5	2/0/0
26K8	477	375	716	9.8	5	2/0/0
30K7	501	461	752	9.8	5	2/0/0
28K9	549	463	824	10.7	5	2/0/0
32LH06	655	655	983	13.1	5	2/0/0
32LH07	735	735	1103	14.1	5	2/0/0
24LH07	748	543	1122	15.4	6	2/0/0
32LH08	829	829	1244	15.2	6	2/0/0
28LH08	846	793	1269	15.8	6	2/0/0
32LH09	1099	1099	1649	19.8	7	2/0/0
28LH10	1163	1062	1745	20.8	7	2/0/0
32LH10	1184	1184	1776	20.8	7	2/0/0
28LH11	1270	1134	1905	22.8	7	2/0/0
32LH11	1296	1296	1944	22.8	7	2/0/0
32LH12	1522	1522	2283	26.4	7	1/0/0
28LH13	1633	1296	2450	29.3	8	1/0/0
32LH13	1789	1789	2684	30.8	8	1/0/0
32LH15	2050	2050	3075	34.6	8	1/0/0
28LH15	2140	1905	3210	38.5	9	1/0/0
32LH16	2691	2691	4037	43.4	9	1/0/0
28LH17	2888	2406	4332	50.3	9	1/0/0
33' LENGTH						
18K3	168	92	252	6.1	4	2/0/1
26K6	364	282	546	8.4	5	2/0/0
28K7	438	364	657	9.2	5	2/0/0
26K8	449	342	674	9.7	5	2/0/0
28K9	527	432	791	10.7	5	2/0/0
32LH06	629	629	944	13.1	5	2/0/0
32LH07	705	705	1058	14	5	2/0/0
28LH07	753	677	1130	15.3	6	2/0/0
28LH08	806	722	1209	16.2	6	2/0/0
24LH09	955	615	1433	20.2	7	2/0/0
24LH10	995	652	1493	21	7	2/0/0
24LH11	1054	683	1581	22.1	7	2/0/0
32LH10	1131	1131	1697	20.7	7	2/0/0
28LH11	1207	1033	1811	22.9	7	2/0/0
32LH12	1453	1453	2180	26.3	7	1/0/0
28LH13	1550	1181	2325	29.2	8	1/0/0

JOIST DESIG.	TOTAL LOAD (ASD)	LOAD for L/360 DEFL.	TOTAL LOAD (LRFD)	JOIST WEIGHT (lbs/ft)	MAX CHORD WIDTH (IN)	BRIDG. (H/X/EX)
33' LENGTH (continued)						
32LH13	1705	1705	2558	30.7	8	1/0/0
32LH14	1827	1827	2741	32.1	8	1/0/0
32LH15	1954	1870	2931	34	8	1/0/0
28LH15	2032	1735	3048	38.1	9	1/0/0
32LH16	2561	2553	3842	42.9	9	1/0/0
32LH17	2951	2920	4427	50.6	9	1/0/0
34' LENGTH						
18K3	158	84	237	6.1	4	2/0/1
26K5	315	237	473	8.2	5	2/0/0
24K7	352	242	528	8.8	5	2/0/0
24K8	389	264	584	9.5	5	2/0/0
28K9	496	395	744	10.6	5	2/0/0
28K10	516	410	774	10.9	5	2/0/0
24LH06	604	411	906	14.5	5	2/0/0
32LH07	678	678	1017	13.9	5	2/0/0
28LH07	718	618	1077	15.1	6	2/0/0
28LH08	768	660	1152	16	6	2/0/0
32LH09	1003	1003	1505	19.4	7	2/0/0
32LH10	1080	1080	1620	20.6	7	2/0/0
32LH11	1183	1183	1775	23	7	2/0/0
32LH12	1388	1388	2082	26.1	7	1/0/0
28LH13	1474	1079	2211	29	8	1/0/0
32LH14	1742	1653	2613	31.7	8	1/0/0
32LH15	1864	1708	2796	33.8	8	1/0/0
28LH15	1931	1585	2897	38.9	9	1/0/0
28LH16	2257	1752	3386	43.2	9	1/0/0
32LH17	2812	2668	4218	50.5	9	1/0/0
24LH19	2832	1804	4248	61.9	11	1/0/0
28LH18	2936	2286	4404	56.3	11	1/0/0
35' LENGTH						
18K3	149	77	224	6.1	4	2/0/1
28K6	349	275	524	8.5	5	2/0/0
28K7	389	305	584	9.1	5	2/0/0
30K7	418	351	627	9.4	5	2/0/0
28K8	430	333	645	9.9	5	2/0/0
28K9	468	361	702	10.6	5	2/0/0
28K10	501	389	752	11	5	2/0/0
32LH06	581	581	872	13.3	6	2/0/0
28LH06	600	502	900	13.7	5	2/0/0
32LH08	728	728	1092	14.9	6	2/0/0
32LH09	959	938	1439	19.5	7	2/0/0
32LH10	1033	1033	1550	21	7	2/0/0
28LH11	1094	865	1641	22.9	7	2/0/0

Loads
DL = 15psf
LLr = 20psf
SL = 14psf

Girder, TYP
Span = 30 ft
Trib Width = 28.835 ft

Trib Area = 30 ft * 28.835 ft = 865 ft2
LLRF = 0.6
WL (MWFRS) = 19.6psf Zone 2

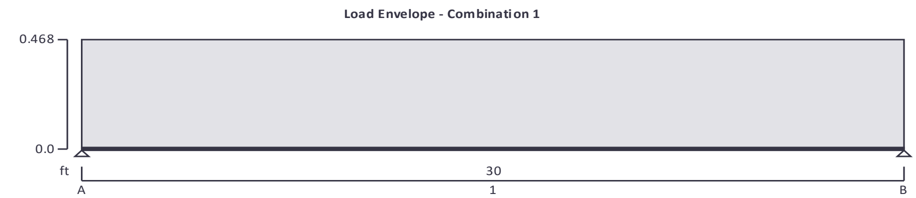
Girder Loading
DL = 15psf*28.84ft = 0.433 klf
LLr = 20psf*28.84ft*0.6 = 0.346 klf
SL = 14psf*28.84ft = 0.404 klf
WL = -19.6psf*28.84ft = -0.565 klf

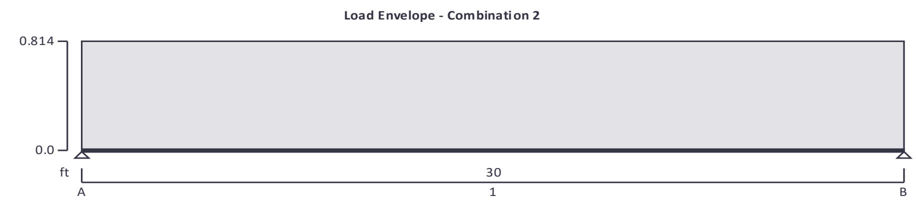
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

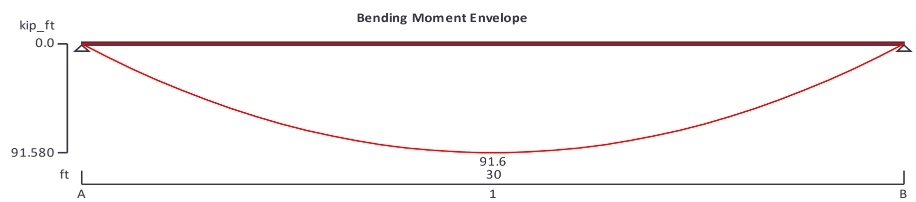
TYPICAL GIRDER (GRAVITY)

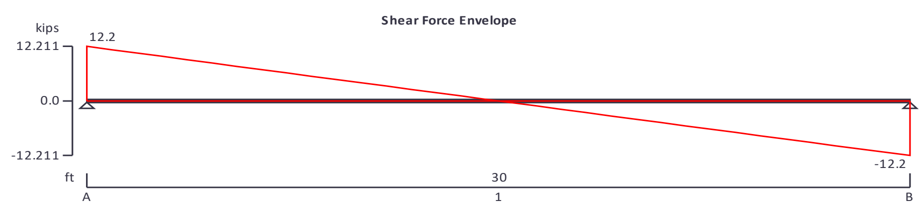
In accordance with AISC360-10 using the ASD method

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 0.433 kips/ft
	Roof live full UDL 0.346 kips/ft
	Snow full UDL 0.404 kips/ft

Load combinations

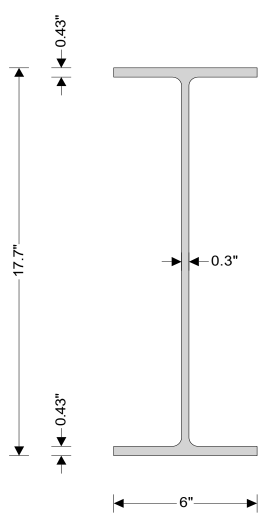
Load combination 1	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.00
Load combination 2	Support A	Dead × 1.00
		Roof live × 1.00
		Dead × 1.00
		Roof live × 1.00
	Support B	Dead × 1.00
		Roof live × 1.00

Analysis results

Maximum moment	M_{max} = **91.6** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **12.2** kips	V_{min} = **-12.2** kips
Deflection	δ_{max} = **1** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **12.2** kips	R_{A_min} = **7** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **7** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **5.2** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **6.1** kips	
Maximum reaction at support B	R_{B_max} = **12.2** kips	R_{B_min} = **7** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **7** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **5.2** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **6.1** kips	

Section details

Section type	**W 18x35 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**

Safety factor for compression Ω_c = **1.67**

Safety factor for flexure Ω_b = **1.67**

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	$V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **12.211** 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
Safety factor for shear	Ω_v = **1.50**
Allowable shear strength	$V_c = V_n / \Omega_v$ = **106.200** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **91.58** 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
Allowable flexural strength	$M_c = M_n / \Omega_b$ = **165.918** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and roof live loads

Limiting deflection	$\delta_{lim} = \min(3.33 \text{ in}, L_{s1} / 240)$ = **1.5** in
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **1.003** in

PASS - Maximum deflection does not exceed deflection limit

Project Typical Girder (Gravity) Project No. ~~22-365~~

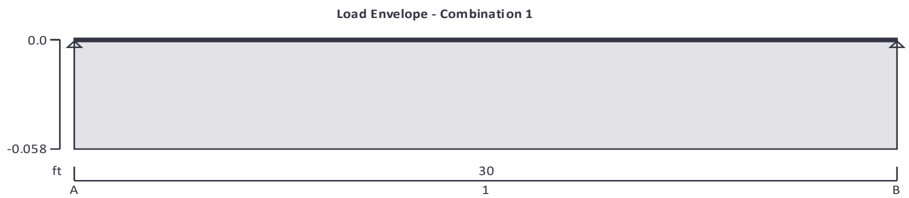
Calc. By KMB Checked By JH Date 10/24/2022

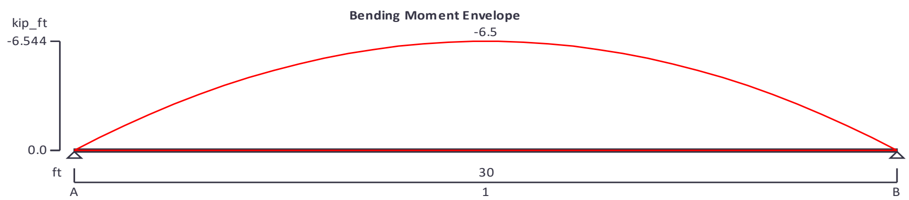
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

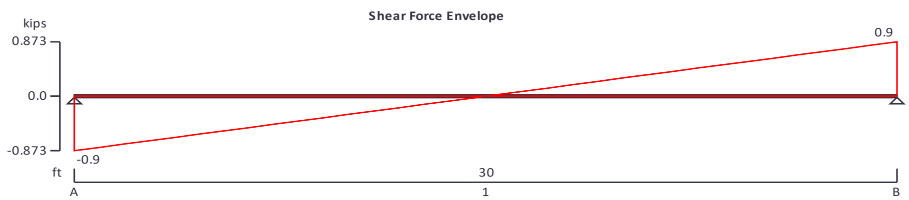
TYPICAL GIRDER (UPLIFT)

In accordance with AISC360-10 using the ASD 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 0.433 kips/ft
	Wind full UDL -0.565 kips/ft

Load combinations

Load combination 1	Support A	Dead × 0.60
		Wind × 0.60
		Dead × 0.60
		Wind × 0.60
	Support B	Dead × 0.60
		Wind × 0.60

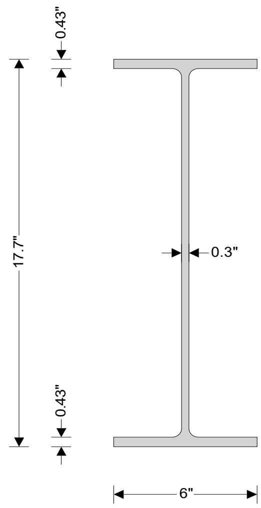
Analysis results

Maximum moment	M_{max} = **0** kips_ft	M_{min} = **-6.5** kips_ft
Maximum shear	V_{max} = **0.9** kips	V_{min} = **-0.9** kips

Deflection	δ_{max} = **0** in	δ_{min} = **0.7** in
Maximum reaction at support A	R_{A_max} = **-0.9** kips	R_{A_min} = **-0.9** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **7** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **-8.5** kips	
Maximum reaction at support B	R_{B_max} = **-0.9** kips	R_{B_min} = **-0.9** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **7** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **-8.5** kips	

Section details

Section type	**W 18x35 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

Lateral bracing

Span 1 has lateral bracing at supports only

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(abs(V_{max}), abs(V_{min})) = **0.873** 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

Safety factor for shear Ω_v = **1.50**

Allowable shear strength $V_c = V_n / \Omega_v$ = **106.200** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength M_r = max(abs(M_{s1_max}), abs(M_{s1_min})) = **6.544** 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

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{s1}$ = **360** in

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]}$ = **51.711** in

Distance between flange centroids $h_o = d - t_f$ = **17.275** in

c = **1**

$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]}$ = **1.514** in

Limiting unbraced length for inelastic LTB - eq F2-6

$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]}$ = **148.563** in

Cross-section mono-symmetry parameter R_m = **1.000**

Moment at quarter point of segment M_A = **4.908** kips_ft

Moment at center-line of segment M_B = **6.544** kips_ft

Moment at three quarter point of segment M_C = **4.908** kips_ft

Maximum moment in segment M_{abs} = **6.544** kips_ft

Lateral torsional buckling modification factor - eq F1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C]$ = **1.136**

Critical flexural stress - eq F2-4 $F_{cr} = C_b \times \pi^2 \times E / (L_b / r_{ts})^2 \times \sqrt{[1 + 0.078 \times J \times c / (S_x \times h_o) \times (L_b / r_{ts})^2]}$ = **10.361** ksi

Nominal flexural strength for lateral torsional buckling - eq F2-2 $M_{nltb} = F_{cr} \times S_x$ = **49.732** kips_ft

Nominal flexural strength M_n = min(M_{nyld}, M_{nltb}) = **49.732** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b$ = **29.779** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to wind loads

Limiting deflection δ_{lim} = min(3.33 in, L_{s1} / 240) = **1.5** in

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **0.696** in

PASS - Maximum deflection does not exceed deflection limit

Typ Girder
W18x35
V 12.2k bearing

Check Web Local Yielding and Web Local Crippling
Fy = 50ksi
tw = 0.3in
k = 1.125in
lb = 8in
d = 17.7in
tf = 0.425in
E = 29000000 psi
Qf = 1.0

Local Yielding, at member end
Rn = Fyw*tw*(2.5*k + lb)
Rn/Omega = 50ksi*0.3in*(2.5*1.125in + 8in) / 1.5 = 108k > 12.2k   OK

Local Cirppling
lb/d = 8in / 17.7in = 0.45 > 0.2
Rn = 0.4*tw^2 *[1 + (4*lb/d - 0.2)*(tw/tf)^1.5]*sqrt(E*Fyw*tf/tw)*Qf
Rn/Omega = 0.4*(0.3in)^2 *[1+(4*8in/17.7in-0.2)*(0.3/0.425)^1.5]*sqrt(29000ksi*50ksi*0.425/0.3)*1.0/2
Rn/Omega = 24.6k > 12.2k   OK

Short Girder
W12x22
V 7.6k bearing

Check Web Local Yielding and Web Local Crippling
Fy = 50ksi
tw = 0.26in
k = 0.9375in
lb = 4in
d = 12.3in
tf = 0.425in
E = 29000000 psi
Qf = 1.0

Local Yielding, at member end
Rn = Fyw*tw*(2.5*k + lb)
Rn/Omega = 50ksi*0.26in*(2.5*0.9375in + 4in) / 1.5 = 54.9k > 7.6k OK

Local Cirppling
lb/d = 4in / 12.3in = 0.325 > 0.2
Rn = 0.4*tw^2 *[1 + (4*lb/d - 0.2)*(tw/tf)^1.5]*sqrt(E*Fyw*tf/tw)*Qf
Rn/Omega =
0.4*(0.26in)^2 *[1+(4*4in/12.3in-0.2)*(0.26/0.425)^1.5]*sqrt(29000ksi*50ksi*0.425/0.26)*1.0/2
Rn/Omega = 31.7k > 7.6k OK

South Short Beam
W12x14
V 3.7k bearing

Check Web Local Yielding and Web Local Crippling
Fy = 50ksi
tw = 0.2in
k = 0.75in
lb = 4in min
d = 11.9in
tf = 0.225in
E = 29000000 psi
Qf = 1.0

Local Yielding, at member end
Rn = Fyw*tw*(2.5*k + lb)
Rn/Omega = 50ksi*0.2in*(2.5*0.75in + 4in) / 1.5 = 39k > 4k OK

Local Crippling
lb/d = 4in / 11.9in = 0.3 > 0.2
Rn = 0.4*tw^2 *[1 + (4*lb/d - 0.2)*(tw/tf)^1.5]*sqrt(E*Fyw*tf/tw)*Qf
Rn/Omega = 0.4*(0.2in)^2 *[1+(4*4in/11.9in-0.2)*(0.2/0.225)^1.5]*sqrt(29000ksi*50ksi*0.225/0.2)*1.0/2
Rn/Omega = 9.8k > 4k OK

Loads
DL = 15psf
LLr = 20psf
SL = 14psf

Girder, Short
Span = 18 ft
Trib Width = 28.835 ft

Trib Area = 18 ft * 28.835 ft = 519 ft2
LLRF = 1.2-0.001*519 = 0.681
WL (C&C) = 44 psf

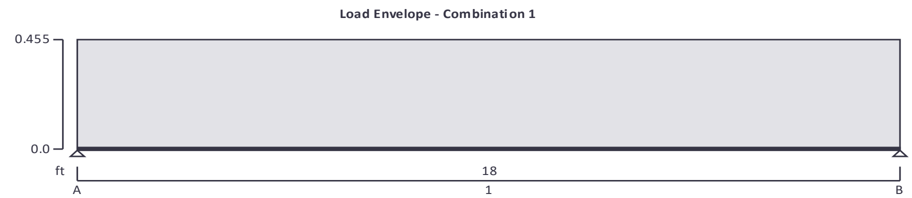
Girder Loading
DL = 15psf*28.84ft = 0.433 klf
LLr = 20psf*28.84ft*0.681 = 0.393 klf
SL = 14psf*28.84ft = 0.404 klf
WL = -44psf*28.84ft = -1.269 klf

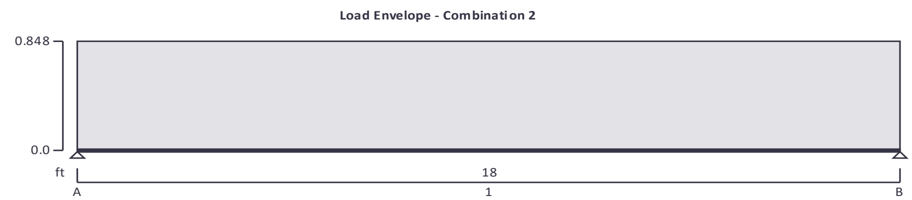
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

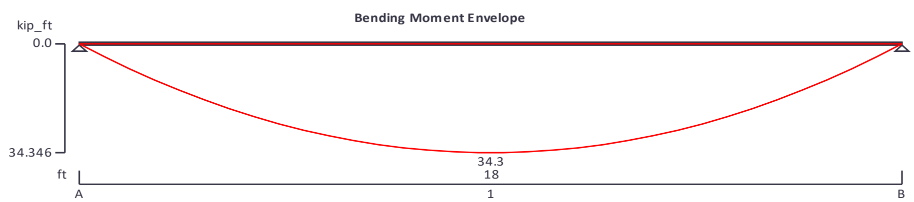
In accordance with AISC360-10 using the ASD method

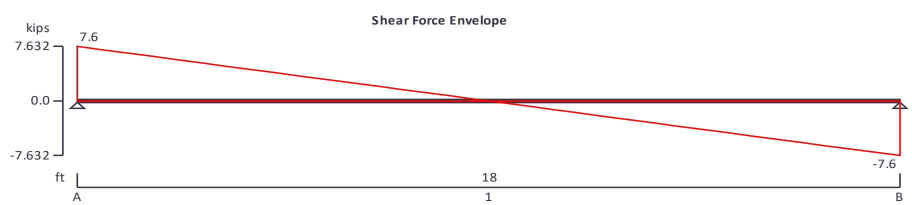
SHORT GIRDER (GRAVITY)

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 0.433 kips/ft
	Roof live full UDL 0.393 kips/ft
	Snow full UDL 0.404 kips/ft

Load combinations

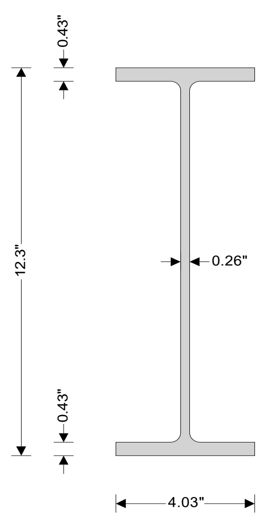
Load combination 1	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.00
Load combination 2	Support A	Dead × 1.00
		Roof live × 1.00
		Dead × 1.00
		Roof live × 1.00
	Support B	Dead × 1.00
		Roof live × 1.00

Analysis results

Maximum moment	M_{max} = **34.3** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **7.6** kips	V_{min} = **-7.6** kips
Deflection	δ_{max} = **0.4** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **7.6** kips	R_{A_min} = **4.1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **4.1** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **3.5** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **3.6** kips	
Maximum reaction at support B	R_{B_max} = **7.6** kips	R_{B_min} = **4.1** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **4.1** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **3.5** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **3.6** kips	

Section details

Section type	**W 12x22 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**

Safety factor for compression Ω_c = **1.67**

Safety factor for flexure Ω_b = **1.67**

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 $b_f / (2 \times t_f)$ = **4.74**

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 $(d - 2 \times k) / t_w$ = **41.73**

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 $V_r = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **7.632** kips

Web area $A_w = d \times t_w$ = **3.198** in^2

Web plate buckling coefficient k_v = **5**

Web shear coefficient - eq G2-3 C_v = **1**

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v$ = **95.940** kips

Safety factor for shear Ω_v = **1.50**

Allowable shear strength $V_c = V_n / \Omega_v$ = **63.960** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **34.346** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **122.083** kips_ft

Nominal flexural strength $M_n = M_{nyld}$ = **122.083** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b$ = **73.104** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and roof live loads

Limiting deflection $\delta_{lim} = \min(3.33\ \mathrm{in}, L_{s1} / 240)$ = **0.9** in

Maximum deflection span 1 $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **0.443** in

PASS - Maximum deflection does not exceed deflection limit

Project Typical Girder (Gravity) Project No. 22-365

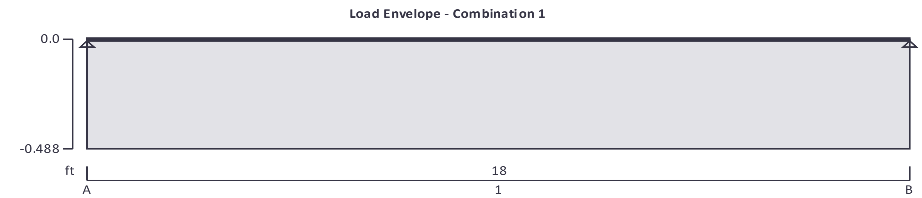
Calc. By KMB Checked By JH Date 10/24/2022

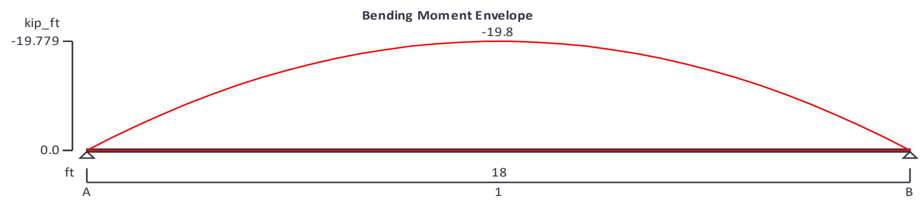
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

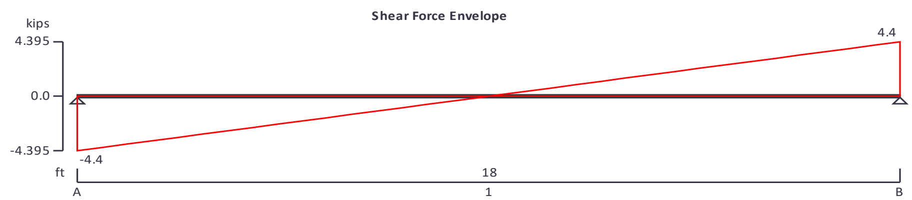
In accordance with AISC360-10 using the ASD method

SHORT GIRDER (UPLIFT)

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 0.433 kips/ft
	Wind full UDL -1.269 kips/ft

Load combinations

Load combination 1	Support A	Dead × 0.60
		Wind × 0.60
		Dead × 0.60
		Wind × 0.60
	Support B	Dead × 0.60
		Wind × 0.60

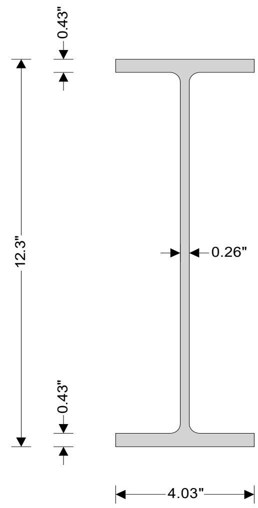
Analysis results

Maximum moment	M_{max} = **0** kips_ft	M_{min} = **-19.8** kips_ft
Maximum shear	V_{max} = **4.4** kips	V_{min} = **-4.4** kips

Deflection	δ_{max} = **0** in	δ_{min} = **0.7** in
Maximum reaction at support A	R_{A_max} = **-4.4** kips	R_{A_min} = **-4.4** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **4.1** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **-11.4** kips	
Maximum reaction at support B	R_{B_max} = **-4.4** kips	R_{B_min} = **-4.4** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **4.1** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **-11.4** kips	

Section details

Section type	**W 12x22 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

Lateral bracing

Span 1 has lateral bracing at supports only

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **4.74**	
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	$(d - 2 \times k) / t_w$ = **41.73**	
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 $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{4.395}$ kips

Web area $A_w = d \times t_w = \mathbf{3.198}$ in^2

Web plate buckling coefficient $k_v = \mathbf{5}$

Web shear coefficient - eq G2-3 $C_v = \mathbf{1}$

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v = \mathbf{95.940}$ kips

Safety factor for shear $\Omega_v = \mathbf{1.50}$

Allowable shear strength $V_c = V_n / \Omega_v = \mathbf{63.960}$ kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{19.779}$ kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x = \mathbf{122.083}$ kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{s1} = \mathbf{216}$ in

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]} = \mathbf{35.944}$ in

Distance between flange centroids $h_o = d - t_f = \mathbf{11.875}$ in

$c = \mathbf{1}$

$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]} = \mathbf{1.043}$ in

Limiting unbraced length for inelastic LTB - eq F2-6

$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]} = \mathbf{109.969}$ in

Cross-section mono-symmetry parameter $R_m = \mathbf{1.000}$

Moment at quarter point of segment $M_A = \mathbf{14.834}$ kips_ft

Moment at center-line of segment $M_B = \mathbf{19.779}$ kips_ft

Moment at three quarter point of segment $M_C = \mathbf{14.834}$ kips_ft

Maximum moment in segment $M_{abs} = \mathbf{19.779}$ kips_ft

Lateral torsional buckling modification factor - eq F1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] = \mathbf{1.136}$

Critical flexural stress - eq F2-4 $F_{cr} = C_b \times \pi^2 \times E / (L_b / r_{ts})^2 \times \sqrt{[1 + 0.078 \times J \times c / (S_x \times h_o) \times (L_b / r_{ts})^2]} = \mathbf{15.638}$ ksi

Nominal flexural strength for lateral torsional buckling - eq F2-2 $M_{nltb} = F_{cr} \times S_x = \mathbf{33.101}$ kips_ft

Nominal flexural strength $M_n = \min(M_{nyld}, M_{nltb}) = \mathbf{33.101}$ kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b = \mathbf{19.821}$ kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to wind loads

Limiting deflection $\delta_{lim} = \min(3.33 \text{ in}, L_{s1} / 240) = \mathbf{0.9}$ in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.663}$ in

PASS - Maximum deflection does not exceed deflection limit

Short Girder
W12x22
V 7.6k bearing

Check Web Local Yielding and Web Local Crippling
Fy = 50ksi
tw = 0.26in
k = 0.9375in
lb = 4in
d = 12.3in
tf = 0.425in
E = 29000000 psi
Qf = 1.0

Local Yielding, at member end
Rn = Fyw*tw*(2.5*k + lb)
Rn/Omega = 50ksi*0.26in*(2.5*0.9375in + 4in) / 1.5 = 54.9k > 7.6k OK

Local Cirppling
lb/d = 4in / 12.3in = 0.325 > 0.2
Rn = 0.4*tw^2 *[1 + (4*lb/d - 0.2)*(tw/tf)^1.5]*sqrt(E*Fyw*tf/tw)*Qf
Rn/Omega =
0.4*(0.26in)^2 *[1+(4*4in/12.3in-0.2)*(0.26/0.425)^1.5]*sqrt(29000ksi*50ksi*0.425/0.26)*1.0/2
Rn/Omega = 31.7k > 7.6k OK

Loads
DL = 15psf
LLr = 20psf
SL = 14psf

Column, Interior
Trib Area = 30 ft * 28.835 ft = 865 ft2
LLRF = 0.6
WL (MWFRS) = 19.6psf Zone 2

Girder Loading
DL = 15psf*865ft2 = 13.0 k
LLr = 20psf*865ft2*0.6 = 10.4 k
SL = 14psf*865ft2 = 12.1 k

P = 13.0k + max(10.4k, 12.1k) = 25.1k

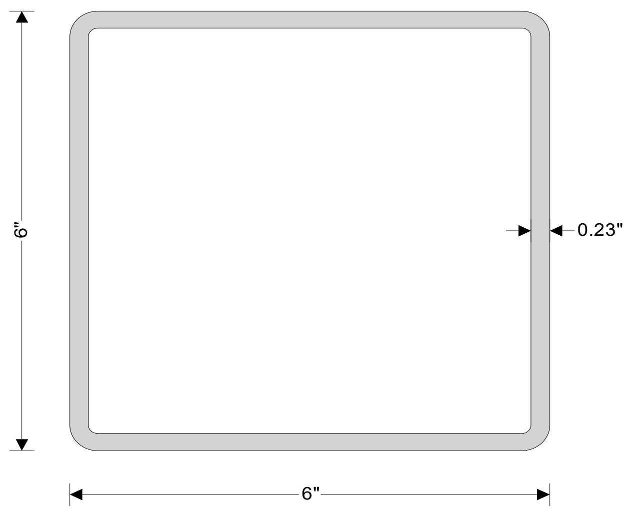
Uplift (ASD)
WL = -19.6psf*865ft2*0.6 = -10.2 k

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

INTERIOR COL

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section **HSS 6x6x1/4**

Design loading

Required axial strength P_r = **25** kips (Compression)

Maximum moment about x axis M_x = **0.0** kips_ft

Maximum moment about y axis M_y = **0.0** kips_ft

Maximum shear force parallel to y axis V_{ry} = **0.0** kips

Maximum shear force parallel to x axis V_{rx} = **0.0** kips

Material details

Steel grade **A500 Gr. B**

Yield strength F_y = **46** ksi

Ultimate strength F_u = **58** ksi

Modulus of elasticity E = **29000** ksi

Shear modulus of elasticity G = **11200** ksi

Unbraced lengths

For buckling about x axis L_x = **216** in

For buckling about y axis L_y = **216** in

For torsional buckling L_z = **216** in

Effective length factors

For buckling about x axis K_x = **1.00**

For buckling about y axis K_y = **1.00**

For torsional buckling K_z = **1.00**

Effective unbraced lengths

For buckling about x axis $L_{cx} = L_x \times K_x$ = **216** in

For buckling about y axis $L_{cy} = L_y \times K_y$ = **216** in

For torsional buckling $L_{cz} = L_z \times K_z$ = **216** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width	$b = b_f - 3 \times t =$ **5.301** in
Critical web width	$h = d - 3 \times t =$ **5.301** in
Width to thickness ratio of flange (compression)	$\lambda_{f\_c} = b / t =$ **22.751**
Width to thickness ratio of web (compression)	$\lambda_{w\_c} = h / t =$ **22.751**
Width to thickness ratio of flange (major flexure)	$\lambda_{f\_fx} = b / t =$ **22.751**
Width to thickness ratio of web (major flexure)	$\lambda_{w\_fx} = h / t =$ **22.751**
Width to thickness ratio of flange (minor flexure)	$\lambda_{f\_fy} = h / t =$ **22.751**
Width to thickness ratio of web (minor flexure)	$\lambda_{w\_fy} = b / t =$ **22.751**

Compression

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

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis	$SR_x = L_{cx} / r_x =$ **92.3**
Slenderness ratio about y axis	$SR_y = L_{cy} / r_y =$ **92.3**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress	$F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **33.6** ksi
Flexural buckling stress	$F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **25.9** ksi
Nominal compressive strength for flexural buckling	$P_{nx} = F_{crx} \times A =$ **135.9** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress	$F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **33.6** ksi
Flexural buckling stress	$F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y =$ **25.9** ksi
Nominal compressive strength for flexural buckling	$P_{ny} = F_{cry} \times A =$ **135.9** kips

Allowable compressive strength (cl. E1)

Safety factor for compression	$\Omega_c =$ **1.67**
Allowable compressive strength	$P_c = \min(P_{nx}, P_{ny}) / \Omega_c =$ **81.4** kips

PASS - The allowable compressive strength exceeds the required compressive strength

Loads
DL = 15psf
LLr = 20psf
SL = 59.8psf (Peak Drift Load)

Column, Exterior
Trib Width = (34.17ft + 45ft)/2 = 39.6ft
Trib Area = 39.6 ft * 10.33 ft / 2 = 205 ft2

WL (C&C) = -44 psf Zone 2

Girder Loading
DL = 15psf*205ft2 = 3.1 k
LLr = 20psf*205ft2 = 4.1 k
SL = 59.8psf*205ft2 = 12.3 k

P = 3.1k + max(4.1k, 12.3k) = 15.4 k

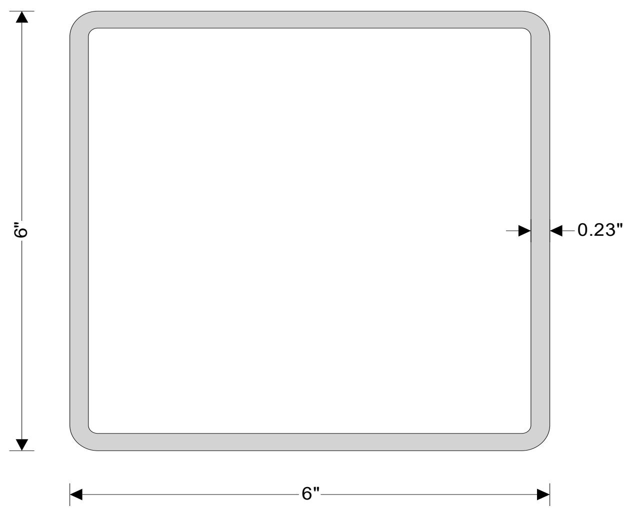
Uplift (ASD)
WL = -44psf*205ft2*0.6 = -5.4 k

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

EXTERIOR COL

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section **HSS 6x6x1/4**

Design loading

Required axial strength P_r = **15** kips (Compression)

Maximum moment about x axis M_x = **0.0** kips_ft

Maximum moment about y axis M_y = **0.0** kips_ft

Maximum shear force parallel to y axis V_{ry} = **0.0** kips

Maximum shear force parallel to x axis V_{rx} = **0.0** kips

Material details

Steel grade **A500 Gr. B**

Yield strength F_y = **46** ksi

Ultimate strength F_u = **58** ksi

Modulus of elasticity E = **29000** ksi

Shear modulus of elasticity G = **11200** ksi

Unbraced lengths

For buckling about x axis L_x = **216** in

For buckling about y axis L_y = **216** in

For torsional buckling L_z = **216** in

Effective length factors

For buckling about x axis K_x = **1.00**

For buckling about y axis K_y = **1.00**

For torsional buckling K_z = **1.00**

Effective unbraced lengths

For buckling about x axis $L_{cx} = L_x \times K_x$ = **216** in

For buckling about y axis $L_{cy} = L_y \times K_y$ = **216** in

For torsional buckling $L_{cz} = L_z \times K_z$ = **216** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width	$b = b_f - 3 \times t =$ **5.301** in
Critical web width	$h = d - 3 \times t =$ **5.301** in
Width to thickness ratio of flange (compression)	$\lambda_{f\_c} = b / t =$ **22.751**
Width to thickness ratio of web (compression)	$\lambda_{w\_c} = h / t =$ **22.751**
Width to thickness ratio of flange (major flexure)	$\lambda_{f\_fx} = b / t =$ **22.751**
Width to thickness ratio of web (major flexure)	$\lambda_{w\_fx} = h / t =$ **22.751**
Width to thickness ratio of flange (minor flexure)	$\lambda_{f\_fy} = h / t =$ **22.751**
Width to thickness ratio of web (minor flexure)	$\lambda_{w\_fy} = b / t =$ **22.751**

Compression

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

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis	$SR_x = L_{cx} / r_x =$ **92.3**
Slenderness ratio about y axis	$SR_y = L_{cy} / r_y =$ **92.3**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress	$F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **33.6** ksi
Flexural buckling stress	$F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **25.9** ksi
Nominal compressive strength for flexural buckling	$P_{nx} = F_{crx} \times A =$ **135.9** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress	$F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **33.6** ksi
Flexural buckling stress	$F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y =$ **25.9** ksi
Nominal compressive strength for flexural buckling	$P_{ny} = F_{cry} \times A =$ **135.9** kips

Allowable compressive strength (cl. E1)

Safety factor for compression	$\Omega_c =$ **1.67**
Allowable compressive strength	$P_c = \min(P_{nx}, P_{ny}) / \Omega_c =$ **81.4** kips

PASS - The allowable compressive strength exceeds the required compressive strength

Loads
DL = 15psf
LLr = 20psf
SL = 59.8psf (Peak Drift Load)
DL = 15psf*10.67ft EIFS /2 = 80.0 plf

South Beam, Long
Span = 45ft max
Trib Width = 10.33 ft / 4 = 2.6 ft
Trib Area = 45 ft * 2.6 ft = 117 ft2

WL (C&C) = -44 psf Zone 2

Beam Loading
DL = 15psf*2.6ft + 80.0plf = 119 plf
LLr = 20psf*2.6ft = 52 plf
SL = 59.8psf*2.6ft = 155.5 plf
WL = -44psf*2.6ft = -114.4 plf

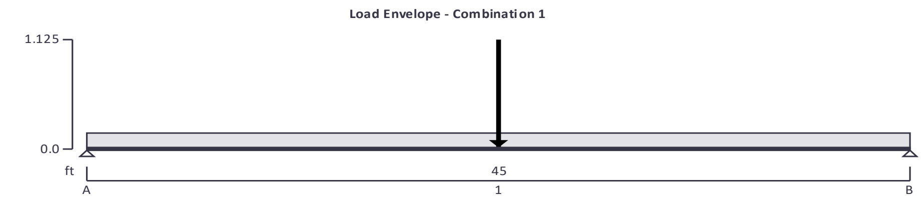
Point Load, Reaction from South Beam, Short
DL = 2.25k / 2 = 1.125k
LLr = 3k / 2 = 1.5k
SL = 5k / 2 = 2.5k
WL = -6.6k / 2 = -3.3k

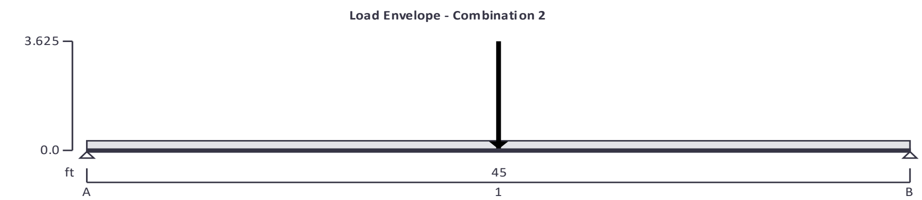
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

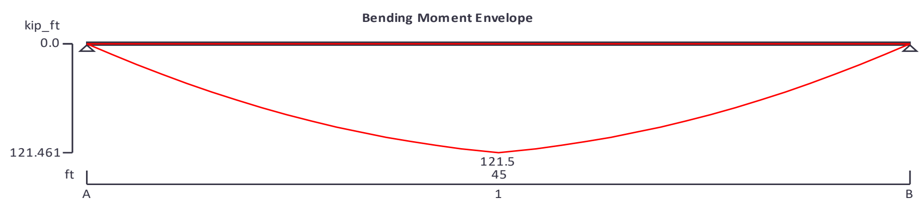
In accordance with AISC360-10 using the ASD method

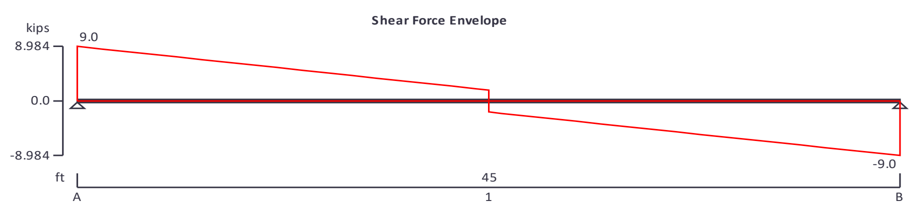
SOUTH BEAM 2 (GRAVITY)

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 0.119 kips/ft
	Snow full UDL 0.156 kips/ft
	Roof live full UDL 0.052 kips/ft
	Dead point load 1.125 kips at 270.00 in

	Snow point load 2.5 kips at 270.00 in	
	Dead full UDL 0 kips/ft	
	Live point load 1.5 kips at 270.00 in	

Load combinations

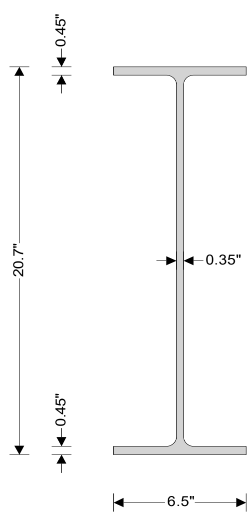
Load combination 1	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.00
Load combination 2	Support A	Dead × 1.00
		Snow × 1.00
		Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **121.5** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **9** kips	V_{min} = **-9** kips
Deflection	δ_{max} = **1** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **9** kips	R_{A_min} = **4.2** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **4.2** kips	
Unfactored live load reaction at support A	R_{A_Live} = **0.8** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **1.2** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **4.7** kips	
Maximum reaction at support B	R_{B_max} = **9** kips	R_{B_min} = **4.2** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **4.2** kips	
Unfactored live load reaction at support B	R_{B_Live} = **0.8** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **1.2** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **4.7** kips	

Section details

Section type	**W 21x44 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

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	$b_f / (2 \times t_f)$ = **7.22**	
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	$(d - 2 \times k) / t_w$ = **53.71**	
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	$V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **8.984** kips
Web area	$A_w = d \times t_w$ = **7.245** in^2
Web plate buckling coefficient	k_v = **5**
Web shear coefficient - eq G2-3	C_v = **1**
Nominal shear strength – eq G2-1	$V_n = 0.6 \times F_y \times A_w \times C_v$ = **217.350** kips
Safety factor for shear	Ω_v = **1.50**
Allowable shear strength	$V_c = V_n / \Omega_v$ = **144.900** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **121.461** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y \times Z_x$ = **397.5** kips_ft
Nominal flexural strength	$M_n = M_{nyld}$ = **397.500** kips_ft
Allowable flexural strength	$M_c = M_n / \Omega_b$ = **238.024** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and roof live loads

Limiting deflection	$\delta_{lim} = \min(3.33 \text{ in}, L_{s1} / 360)$ = **1.5** in
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.963** in

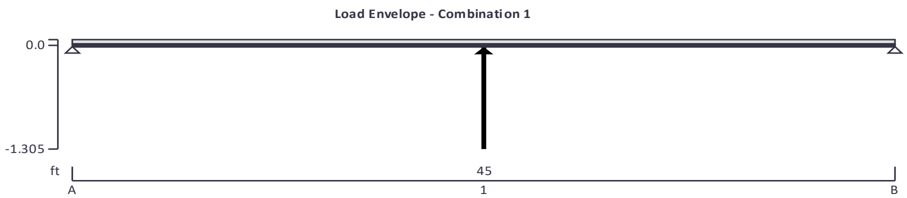
PASS - Maximum deflection does not exceed deflection limit

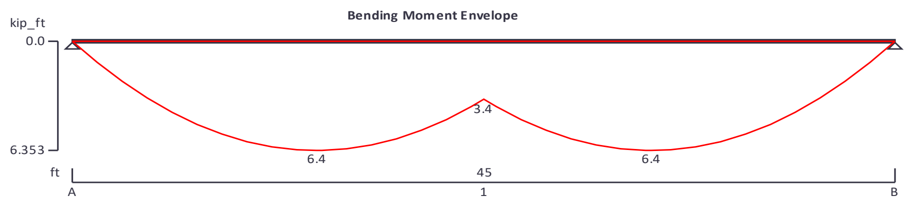
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

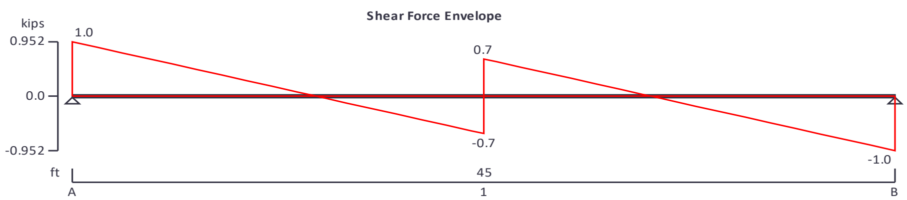
In accordance with AISC360-10 using the ASD method

SOUTH BEAM 2 (UPLIFT)

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 0.039 kips/ft
	Dead point load 1.125 kips at 270.00 in
	Dead full UDL 0.15 kips/ft
	Wind point load -3.3 kips at 270.00 in
	Wind full UDL -0.114 kips/ft

Load combinations

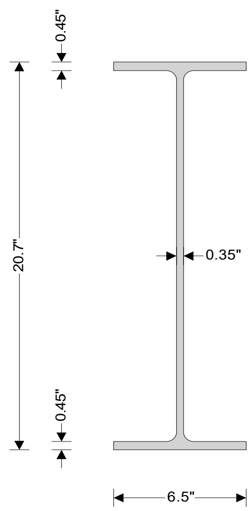
Load combination 1	Support A	Dead × 0.60
		Wind × 0.60
		Dead × 0.60
		Wind × 0.60
	Support B	Dead × 0.60
		Wind × 0.60

Analysis results

Maximum moment	M_{max} = **6.4** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **1** kips	V_{min} = **-1** kips
Deflection	δ_{max} = **1** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **1** kips	R_{A_min} = **1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **5.8** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **-4.2** kips	
Maximum reaction at support B	R_{B_max} = **1** kips	R_{B_min} = **1** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **5.8** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **-4.2** kips	

Section details

Section type	**W 21x44 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

Lateral bracing

Span 1 has lateral bracing at supports only

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **7.22**	
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	$(d - 2 \times k) / t_w$ = **53.71**
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(abs(V_{max}), abs(V_{min})) = **0.952** kips

Web area $A_w = d \times t_w$ = **7.245** in^2

Web plate buckling coefficient k_v = **5**

Web shear coefficient - eq G2-3 C_v = **1**

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v$ = **217.350** kips

Safety factor for shear Ω_v = **1.50**

Allowable shear strength $V_c = V_n / \Omega_v$ = **144.900** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength M_r = max(abs(M_{s1_max}), abs(M_{s1_min})) = **6.353** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **397.5** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{s1}$ = **540** in

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]}$ = **53.407** in

Distance between flange centroids $h_o = d - t_f$ = **20.25** in

c = **1**

$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]}$ = **1.6** in

Limiting unbraced length for inelastic LTB - eq F2-6

$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]}$ = **155.967** in

Cross-section mono-symmetry parameter R_m = **1.000**

Moment at quarter point of segment M_A = **6.196** kips_ft

Moment at center-line of segment M_B = **3.367** kips_ft

Moment at three quarter point of segment M_C = **6.196** kips_ft

Maximum moment in segment M_{abs} = **6.353** kips_ft

Lateral torsional buckling modification factor - eq F1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C]$ = **1.194**

Critical flexural stress - eq F2-4 $F_{cr} = C_b \times \pi^2 \times E / (L_b / r_{ts})^2 \times \sqrt{[1 + 0.078 \times J \times c / (S_x \times h_o) \times (L_b / r_{ts})^2]}$ = **6.802** ksi

Nominal flexural strength for lateral torsional buckling - eq F2-2 $M_{nltb} = F_{cr} \times S_x$ = **46.255** kips_ft

Nominal flexural strength M_n = min(M_{nyld}, M_{nltb}) = **46.255** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b$ = **27.697** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and roof live loads

Limiting deflection δ_{lim} = min(3.33 in, L_{s1} / 360) = **1.5** in

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **1.031** in

PASS - Maximum deflection does not exceed deflection limit

Loads
DL = 15psf
LLr = 20psf
SL = 33.34psf (Avg drift load)

South Beam, Short
Span = 10.33ft
Trib Width = (34.17 ft + 45 ft /2) / 2 = 28.4 ft
Trib Area = 10.33 ft * 28.4 ft /2 = 147 ft2

WL (C&C) = -44 psf Zone 2

Beam Loading, Point Load at midspan
DL = 15psf*150ft2 = 2.25k
LLr = 20psf*150ft2 = 3k
SL = 33.34psf*150ft2 = 5 k
WL = -44psf*150ft2 = -6.6k

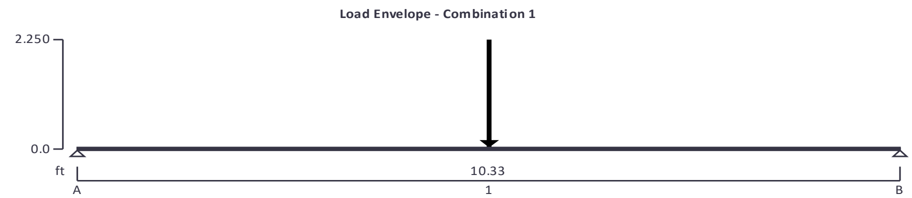
Reactions
DL = 2.25k / 2 = 1.125k
LLr = 3k / 2 = 1.5k
SL = 5k / 2 = 2.5k
WL = -6.6k / 2 = -3.3k

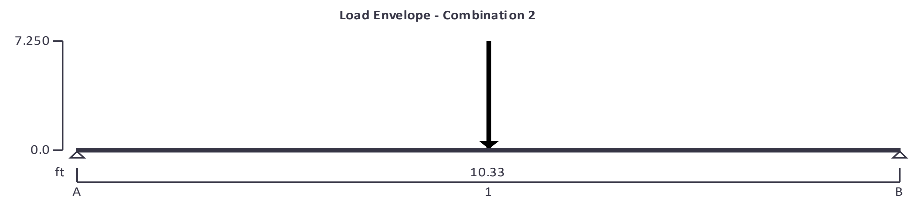
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

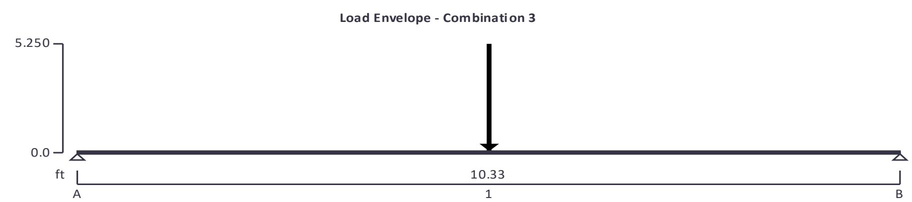
In accordance with AISC360-10 using the ASD method

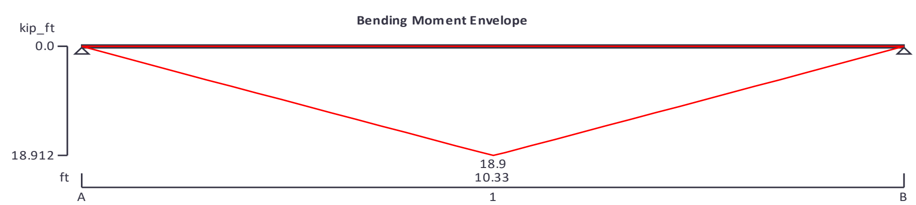
SOUTH BEAM 1 (GRAVITY)

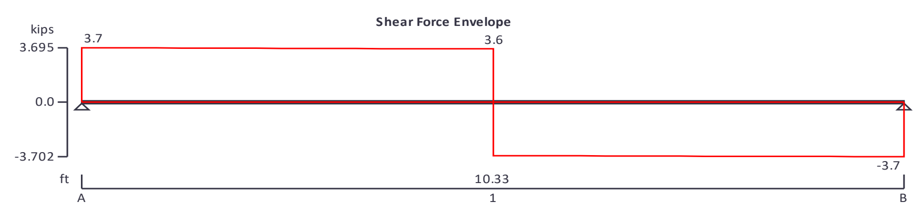
Tedds calculation version 3.0.15











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 point load 2.25 kips at 62.04 in	
	Snow point load 5 kips at 62.04 in	
	Roof live point load 3 kips at 62.04 in	

Load combinations

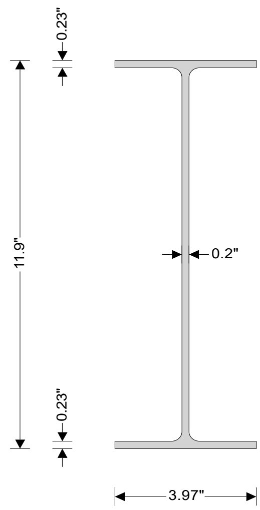
Load combination 1	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.00
Load combination 2	Support A	Dead × 1.00
		Snow × 1.00
		Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.00
		Roof live × 1.00
		Dead × 1.00
		Roof live × 1.00
	Support B	Dead × 1.00
		Roof live × 1.00

Analysis results

Maximum moment	M_{max} = **18.9** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **3.7** kips	V_{min} = **-3.7** kips
Deflection	δ_{max} = **0.1** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **3.7** kips	R_{A_min} = **1.2** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **1.2** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **1.5** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **2.5** kips	
Maximum reaction at support B	R_{B_max} = **3.7** kips	R_{B_min} = **1.2** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **1.2** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **1.5** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **2.5** kips	

Section details

Section type	**W 12x14 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

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	$b_f / (2 \times t_f)$ = **8.82**	
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	$(d - 2 \times k) / t_w$ = **54.25**	
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	$V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **3.702** kips
Web area	$A_w = d \times t_w$ = **2.38** in^2
Web plate buckling coefficient	k_v = **5**
Web shear coefficient - eq G2-3	C_v = **1**
Nominal shear strength – eq G2-1	$V_n = 0.6 \times F_y \times A_w \times C_v$ = **71.400** kips
Safety factor for shear	Ω_v = **1.67**
Allowable shear strength	$V_c = V_n / \Omega_v$ = **42.754** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength	$M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **18.912** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x =$ **72.5** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **72.500** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b =$ **43.413** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and roof live loads

Limiting deflection $\delta_{lim} = \min(3.33 \text{ in}, L_{s1} / 360) =$ **0.344** in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.082** in

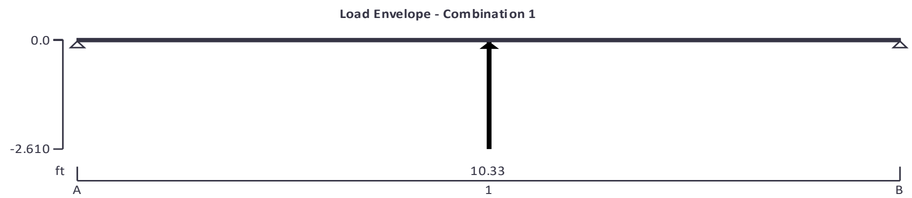
PASS - Maximum deflection does not exceed deflection limit

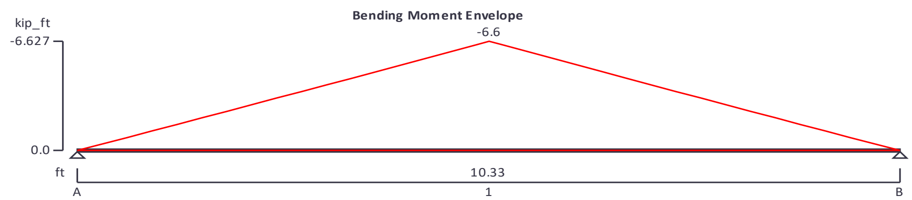
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

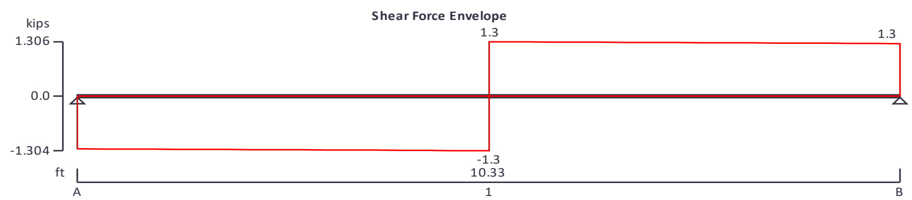
In accordance with AISC360-10 using the ASD method

SOUTH BEAM 1 (UPLIFT)

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 point load 2.25 kips at 62.04 in
	Wind point load -6.6 kips at 62.04 in

Load combinations

Load combination 1	Support A	Dead × 0.60
		Wind × 0.60
		Dead × 0.60
		Wind × 0.60
	Support B	Dead × 0.60
		Wind × 0.60

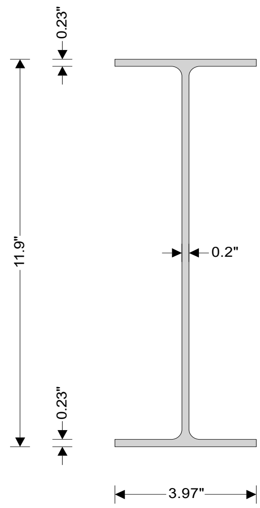
Analysis results

Maximum moment	M_{max} = **0** kips_ft	M_{min} = **-6.6** kips_ft
Maximum shear	V_{max} = **1.3** kips	V_{min} = **-1.3** kips

Deflection	δ_{max} = **0** in	δ_{min} = **0.1** in
Maximum reaction at support A	R_{A_max} = **-1.3** kips	R_{A_min} = **-1.3** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **1.2** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **-3.3** kips	
Maximum reaction at support B	R_{B_max} = **-1.3** kips	R_{B_min} = **-1.3** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **1.2** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **-3.3** kips	

Section details

Section type	**W 12x14 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A992**
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

Lateral bracing

Span 1 has lateral bracing at supports only

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **8.82**	
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	$(d - 2 \times k) / t_w$ = **54.25**	
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 $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **1.306** kips

Web area $A_w = d \times t_w$ = **2.38** in^2

Web plate buckling coefficient k_v = **5**

Web shear coefficient - eq G2-3 C_v = **1**

Nominal shear strength – eq G2-1 $V_n = 0.6 \times F_y \times A_w \times C_v$ = **71.400** kips

Safety factor for shear Ω_v = **1.67**

Allowable shear strength $V_c = V_n / \Omega_v$ = **42.754** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **6.627** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **72.5** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{s1}$ = **123.96** in

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{E / F_y}$ = **31.917** in

Distance between flange centroids $h_o = d - t_f$ = **11.675** in

c = **1**

$r_{ts} = \sqrt{\sqrt{I_y \times C_w} / S_x}$ = **0.961** in

Limiting unbraced length for inelastic LTB - eq F2-6

$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{(J \times c / (S_x \times h_o)) + \sqrt{(J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2}}$ = **92.803** in

Cross-section mono-symmetry parameter R_m = **1.000**

Moment at quarter point of segment M_A = **3.282** kips_ft

Moment at center-line of segment M_B = **6.621** kips_ft

Moment at three quarter point of segment M_C = **3.288** kips_ft

Maximum moment in segment M_{abs} = **6.627** kips_ft

Lateral torsional buckling modification factor - eq F1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C]$ = **1.320**

Critical flexural stress - eq F2-4 $F_{cr} = C_b \times \pi^2 \times E / (L_b / r_{ts})^2 \times \sqrt{1 + 0.078 \times J \times c / (S_x \times h_o) \times (L_b / r_{ts})^2}$ = **28.065** ksi

Nominal flexural strength for lateral torsional buckling - eq F2-2 $M_{nltb} = F_{cr} \times S_x$ = **34.847** kips_ft

Nominal flexural strength $M_n = \min(M_{nyld}, M_{nltb})$ = **34.847** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b$ = **20.867** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and wind loads

Limiting deflection $\delta_{lim} = \min(3.33 \text{ in}, L_{s1} / 360)$ = **0.344** in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.102** in

PASS - Maximum deflection does not exceed deflection limit

South Short Beam
W12x14
V 3.7k bearing

Check Web Local Yielding and Web Local Crippling
Fy = 50ksi
tw = 0.2in
k = 0.75in
lb = 4in min
d = 11.9in
tf = 0.225in
E = 29000000 psi
Qf = 1.0

Local Yielding, at member end
Rn = Fyw*tw*(2.5*k + lb)
Rn/Omega = 50ksi*0.2in*(2.5*0.75in + 4in) / 1.5 = 39k > 4k   OK

Local Crippling
lb/d = 4in / 11.9in = 0.3 > 0.2
Rn = 0.4*tw^2 *[1 + (4*lb/d - 0.2)*(tw/tf)^1.5]*sqrt(E*Fyw*tf/tw)*Qf
Rn/Omega = 0.4*(0.2in)^2 *[1+(4*4in/11.9in-0.2)*(0.2/0.225)^1.5]*sqrt(29000ksi*50ksi*0.225/0.2)*1.0/2
Rn/Omega = 9.8k > 4k   OK

Loads
DL = 15psf
LLr = 20psf
SL = 14psf
DL Wall = 47psf

Entrance Header
Span = 22ft
Trib Width = 2.6 ft + 3 ft = 5.6 ft

Uniform Loading 1
DL = (20.67ft-10ft)*47psf wall + 15psf*5.6ft roof = 0.586 klf
LLr = 15psf*5.6ft = 0.084 klf
SL = 14psf*5.6ft = 0.078 klf

Point Load 10.67ft above beam
Disperses at 45deg, over 21ft length

Reactions from South Beam, Short
DL = 2.25k / 2 = 1.125k
LLr = 3k / 2 = 1.5k
SL = 5k / 2 = 2.5k
Reactions from 15ft of Girder
DL = 0.433 klf*15ft = 6.50k
LLr = 0.346 klf*15ft = 5.19k
SL = 0.404 klf*15ft = 6.06k

Uniform Loading 2
DL = 1.125k + 6.5k = 7.625k /21ft = 0.363klf
LLr = 1.5k + 5.19k = 6.69k /21ft = 0.319klf
SL = 2.5k + 6.06k = 8.56k /21ft = 0.408klf

Out of Plane Wind Load
-22.8psf * 10ft /2 = -0.114 klf

Bentley

Current Date: 10/24/2022 4:36 PM
Units system: English
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Steel Code Check

Report: Comprehensive

Members: Hot-rolled
Design code: AISC 360-2016 ASD

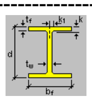
Member : 1
Design status : OK

DESIGN WARNINGS

Section information

Section name: W 16X40 **(US)**

Dimensions



bf	=	7.000	[in]	Width
d	=	16.000	[in]	Depth
k	=	0.907	[in]	Distance k
k1	=	0.813	[in]	Distance k1
tf	=	0.505	[in]	Flange thickness
tw	=	0.305	[in]	Web thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	11.800	
Moment of Inertia (local axes) (I)	[in4]	518.000	28.900
Moment of Inertia (principal axes) (I')	[in4]	518.000	28.900
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	6.626	1.565
Radius of gyration (principal axes) (r')	[in]	6.626	1.565
Saint-Venant torsion constant. (J)	[in4]	0.794	
Section warping constant. (Cw)	[in6]	1730.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	64.700	8.250
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	64.700	8.250
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	64.700	8.250
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	64.700	8.250
Plastic section modulus (local axis) (Z)	[in3]	73.000	12.700
Plastic section modulus (principal axis) (Z')	[in3]	73.000	12.700
Polar radius of gyration. (ro)	[in]	6.808	
Area for shear (Aw)	[in2]	7.070	4.880
Torsional constant. (C)	[in3]	1.480	

Material : A992 Gr50

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	65.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	22.00

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
4.00	22.00
4.00	
4.00	
4.00	
4.00	
0.67	
1.33	

Laterally unbraced length

	Length [ft]			Effective length factor	
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
22.00	22.00	22.00	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✓

Axial tension

Ratio	: 0.00		
Capacity	: 353.29 [Kip]	Reference	: Cl.D2
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(P_n/Ω)	[Kip]	353.29	Cl.D2

AXIAL COMPRESSION DESIGN ✓

Compression in the major axis 33

Ratio	: 0.00		
Capacity	: 298.83 [Kip]	Reference	: Cl.E3
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.93	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	46.51	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.4.1a.Case5
Factored flexural buckling strength(P_{n33}/Ω)	[Kip]	298.83	Cl.E3
Elastic critical buckling stress (F_{e33})	[Kip/in2]	180.28	Eq.E3-4
Effective area of the cross section based on the effective width (A_{eff33})	[in2]	11.21	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	44.52	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	499.05	Eq.E7-1

Compression in the minor axis 22

Ratio	: 0.00		
Capacity	: 62.33 [Kip]	Reference	: Cl.E3
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.93	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.4.1a.Case1
Stiffened element classification	--	Slender	
Stiffened element slenderness (λ)	--	46.51	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.4.1a.Case5
Factored flexural buckling strength(P_{n22}/Ω)	[Kip]	62.33	Cl.E3
Unbraced length (L_{22})	[ft]	22.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	168.69	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	10.06	Eq.E3-4
Effective area of the cross section based on the effective width (A_{eff22})	[in2]	11.80	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	8.82	Eq.E3-3
Factored torsional or flexural-torsional buckling strength(P_{n11}/Ω)	[Kip]	172.47	Cl.E4
Unbraced length (L_{11})	[ft]	22.00	Cl.E2
Effective area of the cross section based on the effective width (A_{eff11})	[in2]	11.80	
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	288.02	Eq.E7-1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	: 0.44		
Capacity	: 182.14 [Kip*ft]	Reference	: Cl.F2.1
Demand	: 80.65 [Kip*ft]	Ctrl Eq.	: D3 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.93	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	46.51	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(M_n/Ω)	[Kip*ft]	182.14	Cl.F2.1
Factored lateral-torsional buckling strength(M_n/Ω)	[Kip*ft]	78.60	Cl.F2.2

Limiting laterally unbraced length for yielding (L_p)	[ft]	5.53	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	1.86	Eq.F2-7
Critical stress (F_{cr})	[Kip/in2]	24.35	Eq.F2-4
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	131.27	Eq.F2-3

Bending about minor axis, M22

Ratio	:	0.13			
Capacity	:	31.69 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	4.14 [Kip*ft]	Ctrl Eq.	:	D6 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.93	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	46.51	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(M_n/Ω)	[Kip*ft]	31.69	Cl.F6.1

DESIGN FOR SHEAR ✓

Shear in major axis 33

Ratio	:	0.01			
Capacity	:	127.01 [Kip]	Reference	:	Cl.G1
Demand	:	0.75 [Kip]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	127.01	Cl.G1
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	212.10	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.13			
Capacity	:	97.60 [Kip]	Reference	:	Cl.G1
Demand	:	12.70 [Kip]	Ctrl Eq.	:	D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	97.60	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	146.40	Eq.G2-1

COMBINED ACTIONS DESIGN ✓

Combined flexure and axial

Ratio	:	0.50			
Ctrl Eq.	:	D8 at 50.00%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force	--	0.50	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	182.14	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	31.69	Cl.H1.1
Available axial strength (P_c)	[Kip]	353.29	Cl.H1.1

Bentley

Current Date: 10/24/2022 4:36 PM
Units system: English
File name: G:\2022\22-365 Westlake Ace Hardware - Lee's Summit, MO\Calculations\B - Roof Framing and Columns\Main Entrance Header.retx

Analysis result

Maximum relative deflections

Remark.- Magnitude of deflections in absolute value.

CONDITION **D1=DL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.29639	(L/891)	50.00000	0.00000	(< L/10000)	0.00000

CONDITION **D2=DL+LLr**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.41392	(L/638)	50.00000	0.00000	(< L/10000)	0.00000

CONDITION **D3=DL+SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.43695	(L/604)	50.00000	0.00000	(< L/10000)	0.00000

CONDITION **D4=DL+0.75LLr**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.38454	(L/687)	50.00000	0.00000	(< L/10000)	0.00000

CONDITION **D5=DL+0.75SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.40181	(L/657)	50.00000	0.00000	(< L/10000)	0.00000

CONDITION **D6=DL+0.6Wz**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.29639	(L/891)	50.00000	0.38987	(L/677)	50.00000

CONDITION **D7=DL+0.45Wz+0.75LLr**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.38454	(L/687)	50.00000	0.29241	(L/903)	50.00000

CONDITION **D8=DL+0.45Wz+0.75SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.40181	(L/657)	50.00000	0.29241	(L/903)	50.00000

CONDITION **D9=DL+0.75SL**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.40181 (L/657)	50.00000	0.00000 (< L/10000)	50.00000

CONDITION **D10=0.6DL+0.6Wz**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.17783 (L/1485)	50.00000	0.38987 (L/677)	50.00000

CONDITION **S1=DL**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.29639 (L/891)	50.00000	0.00000 (< L/10000)	50.00000

CONDITION **S2=DL+LLr**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.41392 (L/638)	50.00000	0.00000 (< L/10000)	50.00000

CONDITION **S3=DL+SL**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.43695 (L/604)	50.00000	0.00000 (< L/10000)	50.00000

CONDITION **S4=DL+0.75LLr**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.38454 (L/687)	50.00000	0.00000 (< L/10000)	50.00000

CONDITION **S5=DL+0.75SL**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.40181 (L/657)	50.00000	0.00000 (< L/10000)	50.00000

CONDITION **S6=DL+0.6Wz**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.29639 (L/891)	50.00000	0.38987 (L/677)	50.00000

CONDITION **S7=DL+0.75SL**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.40181 (L/657)	50.00000	0.00000 (< L/10000)	50.00000

CONDITION **S8=0.6DL+0.6Wz**

Member	**Defl. (2) [in]**	**@(%)**	**Defl. (3) [in]**	**@(%)**
1	0.17783 (L/1485)	50.00000	0.38987 (L/677)	50.00000

HSS Canopy Support
Braced 6' SPA

4 Key Locations
Grid 1: (16'-1/4" - 12'-0") = 4.02ft wall and (20'-8" - 16'-1/4") = 4.65ft parapet
Grid 2.2: (16'-9 3/4" - 12'-0") = 4.81ft wall and (24'-0" - 16'-9 3/4") = 7.19ft parapet
Grid 3.8: (17'-10 1/4" - 12'-0") = 5.85ft wall and (24'-0" - 17'-10 1/4") = 6.15ft parapet
Grid 5: (18'-7 3/4" - 12'-0") = 6.65ft wall and (20'-8" - 18'-7 3/4") = 2.02ft parapet

Wind Load over wall and parapet, 50 ft2 area
+23.3psf / -25.3psf wall
+36.2psf / -24.2psf parapet

EIFS DL
12ft * 15psf /2 = 90plf
Canopy Underside DL
10ft 4in /2 * 15psf = 77.5plf

DL = 90plf + 77.5plf = 167.5plf

Brace Wind Reaction
X = wall length, Y = parapet length
W = wall wind load, P = parapet wind load
R*X + W*X^2/2 = P*Y^2/2
R = P*Y^2 / 2*X - W*X/2

Grid 1: P*(4.65ft)^2 / (2*4.02ft) - W*4.02ft / 2 = 2.69P - 2.01W = 50.5plf / -14.3plf
Grid 2.2: P*(7.19ft)^2 / (2*4.81ft) - W*4.81ft / 2 = 5.37P - 2.4W = 138.5plf / -69.2plf   Governs
Grid 3.8: P*(6.15ft)^2 / (2*5.85ft) - W*8.85ft / 2 = 3.23P - 4.4W = 14.4plf / -33.2plf
Grid 5: P*(2.02ft)^2 / (2*6.65ft) - W*6.65ft / 2 = 0.31P - 3.3W = -65plf / 76.0plf

6ft*138.5plf = 0.83k horz

P*cos(47.5) = 0.83k
0.83k / cos(47.5+3) = 1.3k < 1.5k

Horz force goes into roof diaphragm
Vert force goes into roof joists

V = 1.3k*sin(47.5+3) = 1.0k uplift @ 6ft

Current Date: 11/4/2022 11:21 AM
Units system: English
File name: G:\2022\22-365 Westlake Ace Hardware - Lee's Summit, MO\Calculations\B - Roof Framing and Columns\HSS Canopy.retx

Steel Code Check

Report: Comprehensive

Members: Hot-rolled
Design code: AISC 360-2016 ASD

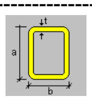
Member	:	**1 (HSS)**
Design status	:	**OK**

DESIGN WARNINGS

Section information

Section name: HSS_RECT 14X6X1_2 **(US)**

Dimensions



a	=	14.000	[in]	Height
b	=	6.000	[in]	Width
T	=	0.465	[in]	Thickness

Properties

Section properties	**Unit**	**Major axis**	**Minor axis**
Gross area of the section. (Ag)	[in2]	17.200	
Moment of Inertia (local axes) (I)	[in4]	402.000	105.000
Moment of Inertia (principal axes) (I')	[in4]	402.000	105.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.834	2.471
Radius of gyration (principal axes) (r')	[in]	4.834	2.471
Saint-Venant torsion constant. (J)	[in4]	279.000	
Section warping constant. (Cw)	[in6]	364.130	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	57.400	35.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	57.400	35.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	57.400	35.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	57.400	35.100
Plastic section modulus (local axis) (Z)	[in3]	73.600	40.400
Plastic section modulus (principal axis) (Z')	[in3]	73.600	40.400
Polar radius of gyration. (ro)	[in]	5.434	
Area for shear (Aw)	[in2]	4.283	11.723
Torsional constant. (C)	[in3]	69.274	

Material : A500 GrB rectangular

Properties	**Unit**	**Value**
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	45.00

Distance between member lateral bracing points

Length (Lb) [ft] Top	Bottom
4.00	45.00
4.00	
4.00	
4.00	
4.00	
0.67	
24.33	

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
45.00	6.00	45.00	1.0	1.0	1.0
	6.00			1.0	
	6.00			1.0	
	6.00			1.0	
	6.00			1.0	
	6.00			1.0	
	6.00			1.0	
	3.00			1.0	

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✓

Axial tension

Ratio	: 0.00		
Capacity	: 473.77 [Kip]	Reference	: Cl.D2
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(P_n/Ω)	[Kip]	473.77	Cl.D2

AXIAL COMPRESSION DESIGN ✓

Compression in the major axis 33

Ratio	: 0.00		
Capacity	: 204.68 [Kip]	Reference	: Cl.E3
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	9.90	
Unstiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	27.11	
Stiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Factored flexural buckling strength(P_{n33}/Ω)	[Kip]	204.68	Cl.E3
Elastic critical buckling stress (F_{e33})	[Kip/in2]	22.94	Eq.E3-4
Effective area of the cross section based on the effective width (A_{eff33})	[in2]	17.20	
Critical stress for flexural buckling (F_{cr33})	[Kip/in2]	19.87	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	341.82	Eq.E3-1

Compression in the minor axis 22

Ratio	: 0.00		
Capacity	: 447.47 [Kip]	Reference	: Cl.E3
Demand	: 0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	9.90	
Unstiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	27.11	
Stiffened element limiting slenderness (λ_r)	--	35.15	Table.4.1a.Case6
Factored flexural buckling strength(P_{n22}/Ω)	[Kip]	447.47	Cl.E3
Unbraced length (L_{22})	[ft]	6.00	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	29.14	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	337.05	Eq.E3-4
Effective area of the cross section based on the effective width (A_{eff22})	[in2]	17.20	
Critical stress for flexural buckling (F_{cr22})	[Kip/in2]	43.45	Eq.E3-2

FLEXURAL DESIGN

Bending about major axis, M33

Ratio	: 0.50		
Capacity	: 168.94 [Kip*ft]	Reference	: Cl.F7.1
Demand	: 84.80 [Kip*ft]	Ctrl Eq.	: D1 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	9.90	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	35.15	
Limiting slenderness for compact unstiffened element (λ_p)	--	28.12	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	27.11	
Limiting slenderness for noncompact stiffened element (λ_r)	--	143.12	
Limiting slenderness for compact stiffened element (λ_p)	--	60.76	
Factored yielding strength(M_n/Ω)	[Kip*ft]	168.94	Cl.F7.1
Factored lateral-torsional buckling strength(M_n/Ω)	[Kip*ft]	168.94	Cl.F7.4
Limiting laterally unbraced length for yielding (L_p)	[ft]	15.88	Eq.F7-12
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	282.13	Eq.F7-10

Bending about minor axis, M22

Ratio	:	0.32			
Capacity	:	92.73 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	30.07 [Kip*ft]	Ctrl Eq.	:	D6 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	9.90	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	35.15	
Limiting slenderness for compact unstiffened element (λ_p)	--	28.12	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	27.11	
Limiting slenderness for noncompact stiffened element (λ_r)	--	143.12	
Limiting slenderness for compact stiffened element (λ_p)	--	60.76	
Factored yielding strength about a geometric axis(M_n/Ω)	[Kip*ft]	92.73	Cl.F7.1

DESIGN FOR SHEAR

Shear in major axis 33

Ratio	:	0.04			
Capacity	:	70.78 [Kip]	Reference	:	Cl.G1
Demand	:	2.67 [Kip]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	70.78	Cl.G1
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	118.20	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.04			
Capacity	:	193.74 [Kip]	Reference	:	Cl.G1
Demand	:	7.54 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(V_n/Ω)	[Kip]	193.74	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	323.55	Eq.G4-1

TORSION DESIGN

Torsion

Ratio	:	0.00			
Capacity	:	95.41 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(T_n/Ω)	[Kip*ft]	95.41	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	27.60	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	159.33	Eq.H3-1

COMBINED ACTIONS DESIGN

Combined flexure and axial

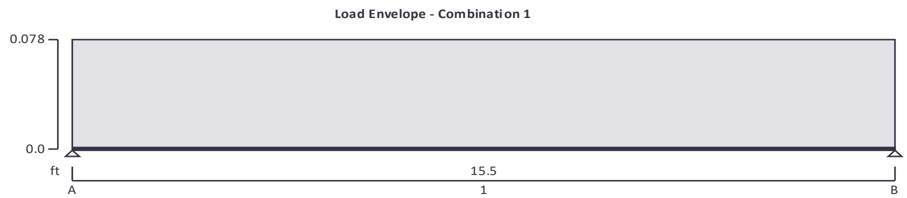
Ratio	:	0.83			
Ctrl Eq.	:	D6 at 50.00%	Reference	:	Eq.H1-1b

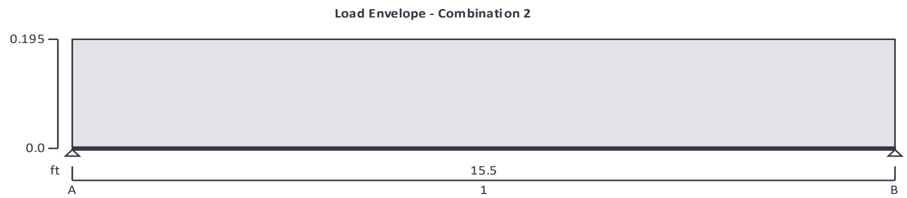
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force	--	0.83	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	168.94	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	92.73	Cl.H1.1
Available axial strength (P_c)	[Kip]	473.77	Cl.H1.1

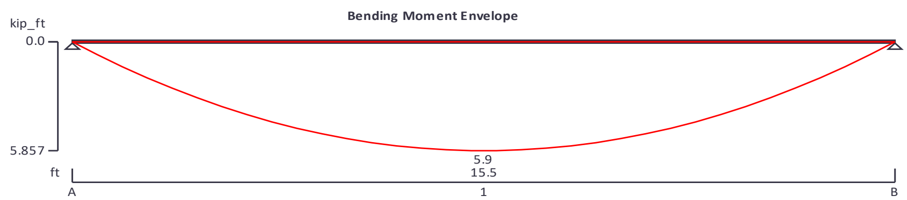
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

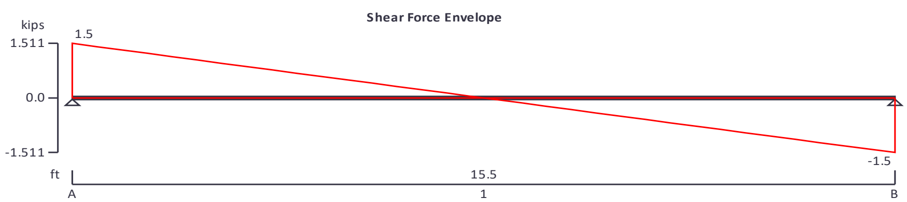
In accordance with AISC360-10 using the ASD method

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 0.06 kips/ft
	Snow full UDL 0.117 kips/ft
	Roof live full UDL 0.08 kips/ft

Load combinations

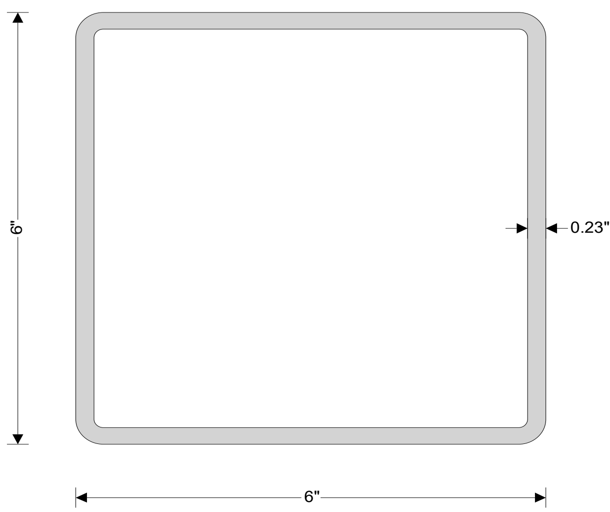
Load combination 1	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.00
Load combination 2	Support A	Dead × 1.00
		Snow × 1.00
		Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **5.9** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **1.5** kips	V_{min} = **-1.5** kips
Deflection	δ_{max} = **0.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **1.5** kips	R_{A_min} = **0.6** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **0.6** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **0.6** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **0.9** kips	
Maximum reaction at support B	R_{B_max} = **1.5** kips	R_{B_min} = **0.6** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **0.6** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **0.6** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **0.9** kips	

Section details

Section type	**HSS 6x6x1/4 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A500 Gr.B 46**
Steel yield stress	F_y = **46** ksi
Steel tensile stress	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding	Ω_{ty} = **1.67**
Safety factor for tensile rupture	Ω_{tr} = **2.00**

Safety factor for compression	Ω_c = **1.67**	
Safety factor for flexure	Ω_b = **1.67**	

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 17)

Width to thickness ratio	$(b_f - 3 \times t) / t$ = **22.75**	
Limiting ratio for compact section	$\lambda_{pff} = 1.12 \times \sqrt{[E / F_y]}$ = **28.12**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.40 \times \sqrt{[E / F_y]}$ = **35.15**	Compact

Classification of web in flexure - Table B4.1b (case 19)

Width to thickness ratio	$(d - 3 \times t) / t$ = **22.75**	
Limiting ratio for compact section	$\lambda_{pwf} = 2.42 \times \sqrt{[E / F_y]}$ = **60.76**	
Limiting ratio for non-compact section	$\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **143.12**	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **1.511** kips
Web area	$A_w = 2 \times (d - 3 \times t) \times t$ = **2.47** in^2
Web plate buckling coefficient	k_v = **5**
Web shear coefficient - eq G2-3	C_v = **1**
Nominal shear strength – eq G2-1	$V_n = 0.6 \times F_y \times A_w \times C_v$ = **68.179** kips
Safety factor for shear	Ω_v = **1.67**
Allowable shear strength	$V_c = V_n / \Omega_v$ = **40.826** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **5.857** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1	$M_{nyld} = M_p = F_y \times Z_x$ = **42.933** kips_ft
Nominal flexural strength	$M_n = M_{nyld}$ = **42.933** kips_ft
Allowable flexural strength	$M_c = M_n / \Omega_b$ = **25.709** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and roof live loads

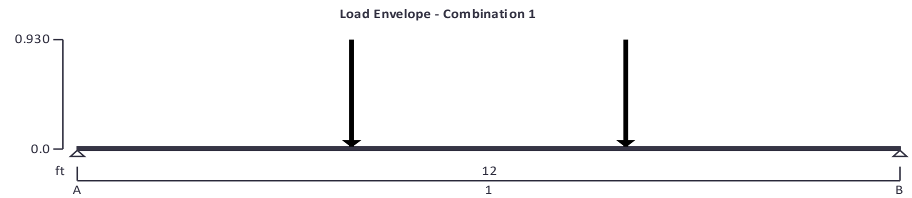
Limiting deflection	$\delta_{lim} = \min(3.33 \text{ in}, L_{s1} / 360)$ = **0.517** in
Maximum deflection span 1	$\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min}))$ = **0.247** in

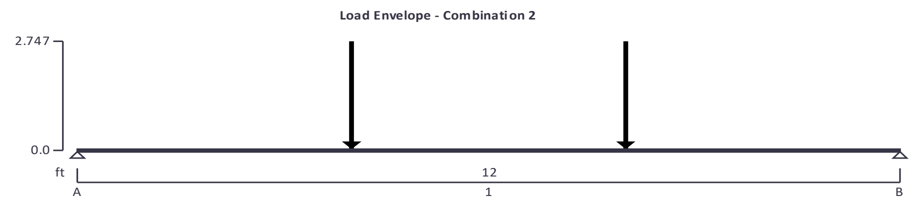
PASS - Maximum deflection does not exceed deflection limit

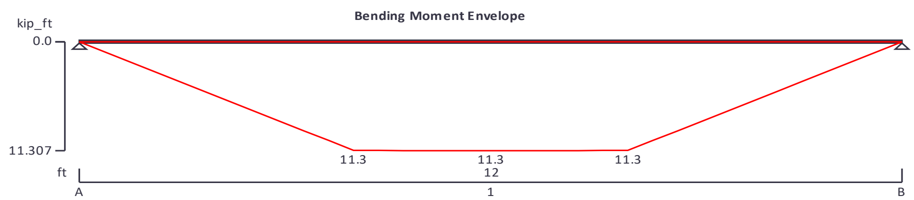
STEEL BEAM ANALYSIS & DESIGN (AISC360-10)

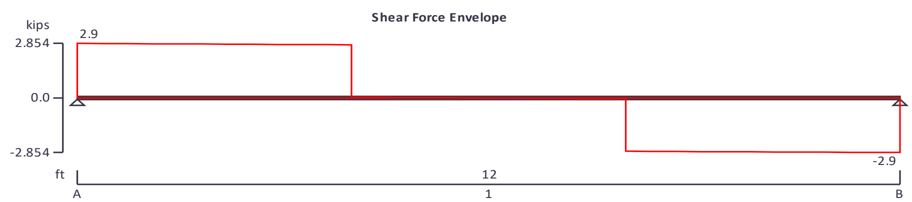
In accordance with AISC360-10 using the ASD method

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 point load 0.93 kips at 48.00 in
	Snow point load 1.817 kips at 48.00 in
	Roof live point load 1.24 kips at 48.00 in
	Dead point load 0.93 kips at 96.00 in

Snow point load 1.817 kips at 96.00 in

Roof live point load 1.24 kips at 96.00 in

Load combinations

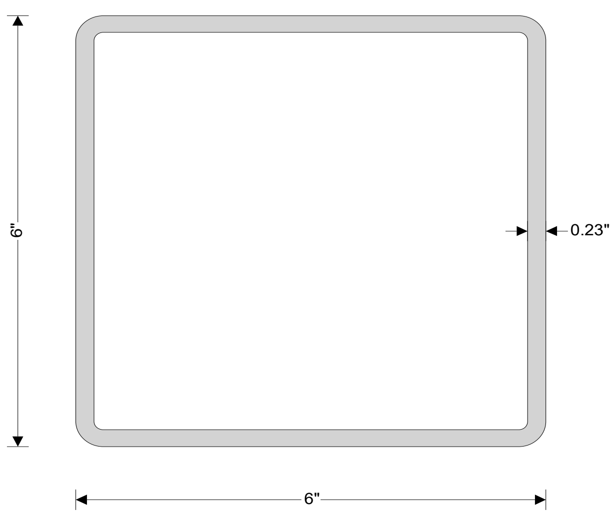
Load combination 1	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.00
Load combination 2	Support A	Dead × 1.00
		Snow × 1.00
		Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **11.3** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **2.9** kips	V_{min} = **-2.9** kips
Deflection	δ_{max} = **0.3** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **2.9** kips	R_{A_min} = **1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **1** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **1.2** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **1.8** kips	
Maximum reaction at support B	R_{B_max} = **2.9** kips	R_{B_min} = **1** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **1** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **1.2** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **1.8** kips	

Section details

Section type	**HSS 6x6x1/4 (AISC 15th Edn (v15.0))**
ASTM steel designation	**A500 Gr.B 46**
Steel yield stress	F_y = **46** ksi
Steel tensile stress	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi



Safety factors

Safety factor for tensile yielding Ω_{ty} = **1.67**

Safety factor for tensile rupture	Ω_{tr} = **2.00**
Safety factor for compression	Ω_c = **1.67**
Safety factor for flexure	Ω_b = **1.67**

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 17)

Width to thickness ratio	$(b_f - 3 \times t) / t$ = **22.75**	
Limiting ratio for compact section	$\lambda_{pff} = 1.12 \times \sqrt{[E / F_y]}$ = **28.12**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.40 \times \sqrt{[E / F_y]}$ = **35.15**	Compact

Classification of web in flexure - Table B4.1b (case 19)

Width to thickness ratio	$(d - 3 \times t) / t$ = **22.75**	
Limiting ratio for compact section	$\lambda_{pwf} = 2.42 \times \sqrt{[E / F_y]}$ = **60.76**	
Limiting ratio for non-compact section	$\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **143.12**	Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **2.854** kips
Web area	$A_w = 2 \times (d - 3 \times t) \times t$ = **2.47** in^2
Web plate buckling coefficient	k_v = **5**
Web shear coefficient - eq G2-3	C_v = **1**
Nominal shear strength – eq G2-1	$V_n = 0.6 \times F_y \times A_w \times C_v$ = **68.179** kips
Safety factor for shear	Ω_v = **1.67**
Allowable shear strength	$V_c = V_n / \Omega_v$ = **40.826** kips

PASS - Allowable shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **11.307** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1	$M_{nyld} = M_p = F_y \times Z_x$ = **42.933** kips_ft
Nominal flexural strength	$M_n = M_{nyld}$ = **42.933** kips_ft
Allowable flexural strength	$M_c = M_n / \Omega_b$ = **25.709** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and roof live loads

Limiting deflection	$\delta_{lim} = \min(3.33 \text{ in}, L_{s1} / 360)$ = **0.4** in
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.287** in

PASS - Maximum deflection does not exceed deflection limit

Loads
DL = 15psf
LLr = 20psf
SL = 29.3psf (Peak Drift Load)

Column, Rear Canopy
Trib Area = 15.5 ft * 12 ft / 4 = 46.5 ft2

WL (C&C) = -47.6 psf Zone 2

Girder Loading
DL = 15psf*47ft2 = 0.71 k
LLr = 20psf*47ft2 = 0.94 k
SL = 29.3psf*47ft2 = 1.38 k

P = 0.71k + max(0.94k, 1.38k) = 2.1 k

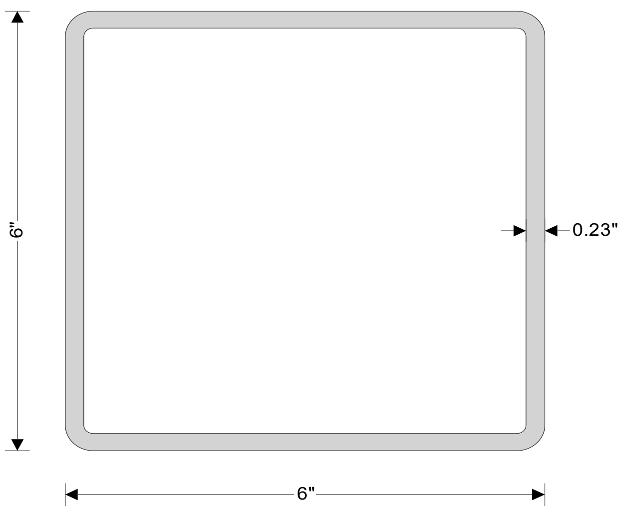
Uplift (ASD)
WL = -47.6psf*47ft2*0.6 = -1.34 k

BP Force
F = 0.6*47ft2 * (-47.6psf + 15psf) = 919lb = 1.0k

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the ASD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section **HSS 6x6x1/4**

Design loading

Required axial strength	P_r = **2** kips (Compression)
Maximum moment about x axis	M_x = **0.0** kips_ft
Maximum moment about y axis	M_y = **0.0** kips_ft
Maximum shear force parallel to y axis	V_{ry} = **0.0** kips
Maximum shear force parallel to x axis	V_{rx} = **0.0** kips

Material details

Steel grade	**A500 Gr. B**
Yield strength	F_y = **46** ksi
Ultimate strength	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **216** in
For buckling about y axis	L_y = **216** in
For torsional buckling	L_z = **216** in

Effective length factors

For buckling about x axis	K_x = **1.00**
For buckling about y axis	K_y = **1.00**
For torsional buckling	K_z = **1.00**

Effective unbraced lengths

For buckling about x axis	$L_{cx} = L_x \times K_x$ = **216** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **216** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **216** in

Section classification

Section classification for local buckling (cl. B4)

Critical flange width	$b = b_f - 3 \times t =$ **5.301** in
Critical web width	$h = d - 3 \times t =$ **5.301** in
Width to thickness ratio of flange (compression)	$\lambda_{f\_c} = b / t =$ **22.751**
Width to thickness ratio of web (compression)	$\lambda_{w\_c} = h / t =$ **22.751**
Width to thickness ratio of flange (major flexure)	$\lambda_{f\_fx} = b / t =$ **22.751**
Width to thickness ratio of web (major flexure)	$\lambda_{w\_fx} = h / t =$ **22.751**
Width to thickness ratio of flange (minor flexure)	$\lambda_{f\_fy} = h / t =$ **22.751**
Width to thickness ratio of web (minor flexure)	$\lambda_{w\_fy} = b / t =$ **22.751**

Compression

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

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis	$SR_x = L_{cx} / r_x =$ **92.3**
Slenderness ratio about y axis	$SR_y = L_{cy} / r_y =$ **92.3**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress	$F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **33.6** ksi
Flexural buckling stress	$F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **25.9** ksi
Nominal compressive strength for flexural buckling	$P_{nx} = F_{crx} \times A =$ **135.9** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress	$F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **33.6** ksi
Flexural buckling stress	$F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y =$ **25.9** ksi
Nominal compressive strength for flexural buckling	$P_{ny} = F_{cry} \times A =$ **135.9** kips

Allowable compressive strength (cl. E1)

Safety factor for compression	$\Omega_c =$ **1.67**
Allowable compressive strength	$P_c = \min(P_{nx}, P_{ny}) / \Omega_c =$ **81.4** kips

PASS - The allowable compressive strength exceeds the required compressive strength

Summary

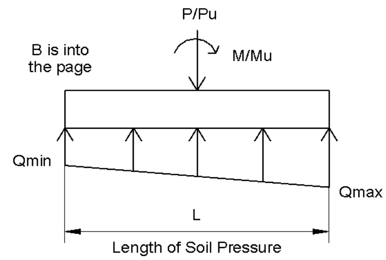
The gravity structure system of the project referenced above consists primarily of steel joist and beams, steel columns, and load bearing masonry walls. All masonry walls are supported by continuous foundations and all columns are supported by spread footings. A new trash enclosure and monument sign made of 8" masonry walls are supported by continuous foundations. The locations of all footings are indicated on the structural framing plans and within the attached sketches.

The following section of calculations covers the complete design of the foundation system for project referenced above. Refer to the "Loads" section of these calculations for the determination of all dead, live, roof live, and snow loads.

BSE Structural Engineers, LLC ▪ 11320 West 79th Street ▪ Lenexa, Kansas 66214 ▪ 913.492.7400 ▪ www.BSEstructural.com

Footing Designation: F1

Footing Location: Interior



General Information:

Footing Length, L =	4	ft			
Footing Width, B =	4	ft			
Footing Depth, H =	18	in			
Location =	Interior	in			
Steel Depth, d =	14.25	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in			
Slab Depth Above Footing =	4	in			
Area of Footing =	16	ft^2			(B*L)
Soil Bearing Pressure =	2	ksf			
Allowable or Effective SBC?	Allowable				
Concrete Strength =	3	ksi			
	B Direction			L Direction	
Column Size =	6.00 in		X	6.00 in	
Base Plate Size =	12.00 in		X	12.00 in	
Critical Section =	9.00 in		X	9.00 in	

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	13	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	4.4	k	Live =	1.6	
Applied Live Load =	12.1	k	Uplift=	1.6	
ASD Total Load, P =	29.5	k			
LRFD Total Load, Pu =	40.24	k	***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accc**		
ASD Uplift Load =	10.2	k			
LRFD Uplift Load =	16.32	k			

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.844	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.844	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(ASD M / Pu)
Kern =	0.667	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	2.515	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	2.515	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	2.515	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.031	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	10.37	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	56.20	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	23.25	in	(Critical Section B + d)
b2 =	23.25	in	(Column Height L + d)
b0 =	93.00	in	(2*b1 + 2*b2)
Vu2 =	30.80	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	326.64	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	442.55	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	217.76	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	10.2	k	(From Above)
Required Dead Load =	17.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	17.4	k	
Additional Slab Used =	5	ft	(Length of Additional Slab Past Edge of Footing in Each Direction
Additional Slab Area =	180	ft^2	From Each Edge of Footing)
Additional Slab Weight =	9	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	26.40	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.54	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.003	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.011	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	4		
Bar Spacing =	12	in	
As Provided =	0.20	in^2/ft	= 5 Bars in B Direction
			= 5 Bars in L Direction

Bottom Steel:

Mu =	1.34	k-ft / ft	(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.007	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0001		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0002		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.028	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	4		
Bar Spacing =	12	in	
As Provided =	0.20	in^2/ft	= 5 Bars in B Direction
			= 5 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.324	in^2/ft	(T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.20	in^2/ft	
As Provided Bott =	0.20	in^2/ft	
As Provided Total =	0.40	in^2/ft	
T&S Steel Provided?	**YES**		

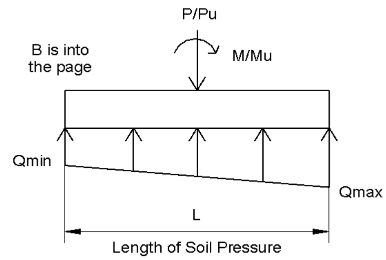
Final Footing Design:

Footing Width, B =	**4**	ft
Footing Length L =	**4**	ft
Footing Depth, H =	**18**	in

Top Steel =	**#4 bars**	**@12 inches O.C.**
Bottom Steel =	**#4 bars**	**@12 inches O.C.**

Footing Designation: F2

Footing Location: Exterior



General Information:

Footing Length, L =	4.5	ft		
Footing Width, B =	4.5	ft		
Footing Depth, H =	34	in		
Location =	Corner	in		
Steel Depth, d =	30.0625	in		(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in		
Slab Depth Above Footing =	4	in		
Area of Footing =	20.25	ft^2		(B*L)
Soil Bearing Pressure =	2	ksf		
Allowable or Effective SBC?	Allowable			
Concrete Strength =	3	ksi		
	B Direction			L Direction
Column Size =	6.00 in		X	6.00 in
Base Plate Size =	12.00 in		X	12.00 in
Critical Section =	9.00 in		X	9.00 in

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	3.1	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	8.859375	k	Live =	1.6	
Applied Live Load =	12.3	k	Uplift=	1.67	
ASD Total Load, P =	24.259375	k			
LRFD Total Load, Pu =	34.03125	k	***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accc**		
ASD Uplift Load =	5.4	k			
LRFD Uplift Load =	9.018	k			

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.750	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.198	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.198	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(ASD M / Pu)
Kern =	0.750	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.681	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.681	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.681	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	0.622	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	4.71	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	133.37	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	39.06	in	(Critical Section B + d)
b2 =	39.06	in	(Column Height L + d)
b0 =	156.25	in	(2*b1 + 2*b2)
Vu2 =	16.22	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1157.76	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1128.43	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	771.84	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	5.4	k	(From Above)
Required Dead Load =	9.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	11.959375	k	
Additional Slab Used =	0	ft	(Length of Additional Slab Past Edge of Footing in Each Direction
Additional Slab Area =	0	ft^2	From Each Edge of Footing)
Additional Slab Weight =	0	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	11.96	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.09	k-ft / ft		(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529			(m = fy/(0.85*f'c))
Ru =	0.000	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027			(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.001	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in^2/ft	=	6 Bars in B Direction
			=	6 Bars in L Direction

Bottom Steel:

Mu =	0.33	k-ft / ft		(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	23.529			(m = fy/(0.85*f'c))
Ru =	0.000	ksi		(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027			(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.003	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in^2/ft	=	6 Bars in B Direction
			=	6 Bars in L Direction

Temperature & Shrinkage Steel:

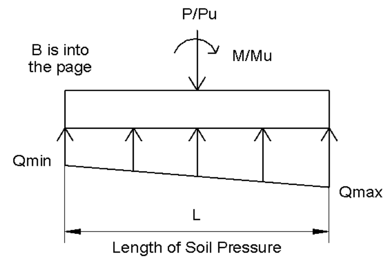
Minimum Steel =	0.5208	in^2/ft	(T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

Footing Width, B =	**4.5**	ft
Footing Length L =	**4.5**	ft
Footing Depth, H =	**34**	in
Top Steel =	**#5 bars**	**@12 inches O.C.**
Bottom Steel =	**#5 bars**	**@12 inches O.C.**

Footing Designation: F3

Footing Location: Rear Canopy



General Information:

Footing Length, L =	4	ft				
Footing Width, B =	4	ft				
Footing Depth, H =	34	in				
Location =	Corner	in				
Steel Depth, d =	30.0625	in		(H - 3 in - 1.5*Bar Dia.)		
Typical Slab Depth =	4	in				
Slab Depth Above Footing =	4	in		Trib Area	402	ft2
Area of Footing =	16	ft^2		DL	15	psf
Soil Bearing Pressure =	2	ksf		LLR	20	psf
Allowable or Effective SBC?	Allowable			WL	24.4	psf
Concrete Strength =	3	ksi				
	B Direction			L Direction		
Column Size =	6.00 in		X	6.00 in		
Base Plate Size =	12.00 in		X	12.00 in		
Critical Section =	9.00 in		X	9.00 in		

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	0.71	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	7	k	Live =	1.6	
Applied Live Load =	0.94	k	Uplift=	1.67	
ASD Total Load, P =	8.65	k			
LRFD Total Load, Pu =	10.756	k	***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accc**		
ASD Uplift Load =	1.34	k			
LRFD Uplift Load =	2.2378	k			

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.67	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	0.541	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.541	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(ASD M / Pu)
Kern =	0.667	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	0.672	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.672	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	0.672	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	0.372	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	1.00	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	118.55	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	39.06	in	(Critical Section B + d)
b2 =	39.06	in	(Column Height L + d)
b0 =	156.25	in	(2*b1 + 2*b2)
Vu2 =	3.63	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1157.76	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1128.43	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	771.84	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	1.34	k	(From Above)
Required Dead Load =	2.23	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	7.71	k	
Additional Slab Used =	0	ft	(Length of Additional Slab Past Edge of Footing in <u>Each Direction</u>
Additional Slab Area =	0	ft^2	From <u>Each Edge of Footing</u>)
Additional Slab Weight =	0	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	7.71	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.01	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.000	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	12	in	
As Provided =	0.31	in^2/ft	= 5 Bars in B Direction
			= 5 Bars in L Direction

Bottom Steel:

Mu =	0.05	k-ft / ft	(Mu= Qcrit*0.5*Lcrit^2 + (Qmax-Qcrit)*0.5*(2/3)*Lcrit^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.000	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	12	in	
As Provided =	0.31	in^2/ft	= 5 Bars in B Direction
			= 5 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.5208	in^2/ft	(T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

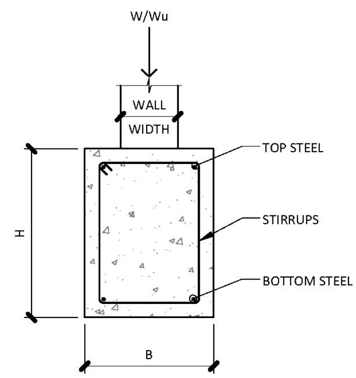
Final Footing Design:

Footing Width, B =	**4**	ft
Footing Length L =	**4**	ft
Footing Depth, H =	**34**	in

Top Steel =	**#5 bars**	**@12 inches O.C.**
Bottom Steel =	**#5 bars**	**@12 inches O.C.**

Grade Beam Design

Grade Beam Location: Perimeter



General Information:

Footing Width, B =	18	in	
Footing Depth, H =	34	in	
Steel Depth, d =	30.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	8	in	
Soil Bearing Pressure =	2	ksf	
Allowable or Effective SBC?	Allowable		
Footing Concrete Strength =	3	ksi	
Wall Concrete Strength =	3	ksi	

Loading:

Vertical Loads:

Applied Dead Load =	0.223	klf	
Wall Weight =	47	psf	
Wall Height =	24.67	ft	
Total Wall Weight =	1.15949	klf	
Footing Weight =	0.6375	klf	
Applied Live Load =	0.429	klf	
ASD Total Load, W =	2.44899	klf	
LRFD Total Load, Wu =	3.110388	klf	

LRFD Factors: (ASCE 7 Combo)

Dead =	1.2
Live =	1.6

ASD Soil Pressures:

Required Footing Width =	1.224495	ft	(Footing Width = W / Soil Bearing Pressure)
Actual Soil Bearing Pressure =	1632.66	psf	(Actual Soil Bearing Pressure = W / Footing Width)
Chosen Footing Width =	1.5	ft	
Assumed Footing Span =	2	ft	
Is Footing Width Adequate?	**YES**		

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFDl Bearing Pressure =	2073.592	psf	
h=	32	in	(h = H - 2 in.)
Cantilever =	5	in	(Cantilever = B/2 - Wall Width/2)
Vu1 =	0.86	k/ft	(LRFD Bearing Pressure* Cantilever)
ΦVn =	21.03	k/ft	(ACI 318-11 Equation 22-9, ΦVn = 0.75*(4/3)*sqrt(f'c)
Adequate in One-Way Shear?	**YES**		

Plain Concrete Flexure Check: Cantilevered Side of Footing

h=	32	in	(h = H - 2 in.)
Cantilever =	5	in	(Cantilever = B/2 - Wall Width/2)
Mu =	0.18	k-ft/ft	(Actual Soil Bearing Pressure* Cantilever)
S=	3072.00	in^3	(S = 12*h^2 / 4)
ΦMn =	63.10	k-ft/ft	(ACI 318-11 Equation 22-2, ΦMn = 0.9*5*sqrt(f'c)*S /
Adequate in Flexure?	**YES**		

One-Way Shear Check: For Spanning "X" Distance Listed

Vu1 =	3.11	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	44.46	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*(
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Oth

Use #3 Stirrups at 18 in. O.C. (Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	159.12	klf	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wal
Adequate in Bearing?	**YES**		

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

Mu =	1.56	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.001	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy &
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ R
A's Required =	0.015	in^2	(As = Governing ρ*B*d)
Bar # =	5		
Number of Bars =	2	bars	
As Provided =	0.62	in^2	
Is Steel Adequate ?	**YES**		

Top Steel:

Bar # =	5	
Number of Bars =	2	bars
As Provided =	0.62	in^2

Temperature & Shrinkage Steel:

Minimum Steel =	1.1016	in^2/ft	(T&S Steel = 0.0018*B*H)
As Provided Top =	0.62	in^2/ft	
As Provided Bott =	0.62	in^2/ft	
As Provided Total =	1.24	in^2/ft	
T&S Steel Provided?	**YES**		

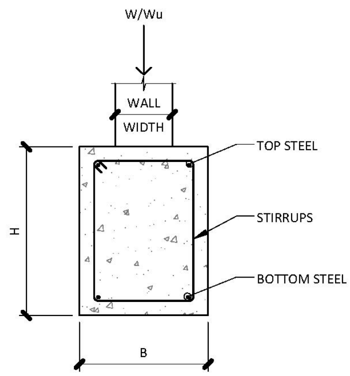
Final Footing Design:

Footing Width, B =	**18**	in
Footing Depth, H =	**34**	in

Top Steel =	**(2) #5 bars**
Bottom Steel =	**(2) #5 bars**
Stirrups =	**#3 Stirrups at 18 in. O.C.**

Grade Beam Design

Grade Beam Location: Perimeter w/ Double Wall



General Information:

Footing Width, B =	26	in	
Footing Depth, H =	34	in	
Steel Depth, d =	30.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	16	in	
Soil Bearing Pressure =	2	ksf	
Allowable or Effective SBC?	Allowable		
Footing Concrete Strength =	3	ksi	
Wall Concrete Strength =	3	ksi	

Loading:

Vertical Loads:

Applied Dead Load =	0.223	klf
Wall Weight =	47	psf
Wall Height =	24.67	ft
Total Wall Weight =	1.15949	klf
Footing Weight =	0.920833333	klf
Applied Live Load =	0.429	klf
ASD Total Load, W =	2.732323333	klf
LRFD Total Load, Wu =	3.450388	klf

LRFD Factors: (ASCE 7 Combo)

Dead =	1.2
Live =	1.6

ASD Soil Pressures:

Required Footing Width =	1.366161667	ft	(Footing Width = W / Soil Bearing Pressure)
Actual Soil Bearing Pressure =	1261.072308	psf	(Actual Soil Bearing Pressure = W / Footing Width)
Chosen Footing Width =	2.166666667	ft	
Assumed Footing Span =	2	ft	
Is Footing Width Adequate?	**YES**		

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFDl Bearing Pressure =	1592.486769	psf	
h=	32	in	(h = H - 2 in.)
Cantilever =	5	in	(Cantilever = B/2 - Wall Width/2)
Vu1 =	0.66	k/ft	(LRFD Bearing Pressure* Cantilever)
ΦVn =	21.03	k/ft	(ACI 318-11 Equation 22-9, ΦVn = 0.75*(4/3)*sqrt(f'c)
Adequate in One-Way Shear?	**YES**		

Plain Concrete Flexure Check: Cantilevered Side of Footing

h=	32	in	(h = H - 2 in.)
Cantilever =	5	in	(Cantilever = B/2 - Wall Width/2)
Mu =	0.14	k-ft/ft	(Actual Soil Bearing Pressure* Cantilever)
S=	3072.00	in^3	(S = 12*h^2 / 4)
ΦMn =	63.10	k-ft/ft	(ACI 318-11 Equation 22-2, ΦMn = 0.9*5*sqrt(f'c)*S /
Adequate in Flexure?	**YES**		

One-Way Shear Check: For Spanning "X" Distance Listed

Vu1 =	3.45	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	64.22	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otl

Use #3 Stirrups at 18 in. O.C. (Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	318.24	klf	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wal
Adequate in Bearing?	**YES**		

Bottom Steel Design for Flexure : For Spanning "X" Distance Listed

Mu =	1.73	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.001	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy &
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ R
A's Required =	0.017	in^2	(As = Governing ρ*B*d)
Bar # =	5		
Number of Bars =	2	bars	
As Provided =	0.62	in^2	
Is Steel Adequate ?	**YES**		

Top Steel:

Bar # =	5	
Number of Bars =	2	bars
As Provided =	0.62	in^2

Temperature & Shrinkage Steel:

Minimum Steel =	1.2376	in^2/ft	(T&S Steel = 0.0018*B*H)
As Provided Top =	0.62	in^2/ft	
As Provided Bott =	0.62	in^2/ft	
As Provided Total =	1.24	in^2/ft	
T&S Steel Provided?	**YES**		

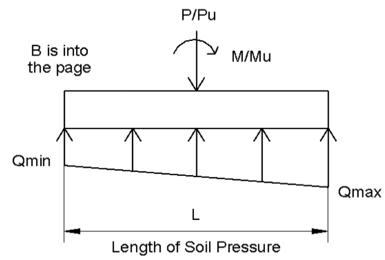
Final Footing Design:

Footing Width, B =	**26**	in
Footing Depth, H =	**34**	in

Top Steel = **(2) #5 bars**
Bottom Steel = **(2) #5 bars**
Stirrups = **#3 Stirrups at 18 in. O.C.**

Footing Designation: P1

Footing Location: Truss Shade Interior



General Information:

Footing Diameter, D =	30	in			
Footing Depth, H =	48	in			
Location =	Interior	in			
Steel Depth, d =	44.0625	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in			
Slab Depth Above Footing =	4	in			
Area of Footing =	5.161984	ft^2			(B*L)
Soil Bearing Pressure =	2	ksf			
Allowable or Effective SBC?	Allowable				
Concrete Strength =	3	ksi			
	B Direction			L Direction	
Column Size =	3.00 in		X	3.00 in	
Base Plate Size =	7.00 in		X	7.00 in	
Critical Section =	5.00 in		X	5.00 in	

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	1	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	3.3552896	k	Live =	1.6	
Applied Live Load =	0	k	Uplift=	1.6	
ASD Total Load, P =	4.3552896	k			
LRFD Total Load, Pu =	5.22634752	k			
ASD Uplift Load =	5.89	k			
LRFD Uplift Load =	9.424	k			

***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accc**

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.6	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.379	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	2.272	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	0.844	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.844	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(ASD M / Pu)
Kern =	0.379	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	2.272	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.012	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.012	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.012	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	-0.908	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	-2.09	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	98.70	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	49.06	in	(Critical Section B + d)
b2 =	49.06	in	(Column Height L + d)
b0 =	196.25	in	(2*b1 + 2*b2)
Vu2 =	-11.70	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	2131.34	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	3900.66	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	1420.89	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

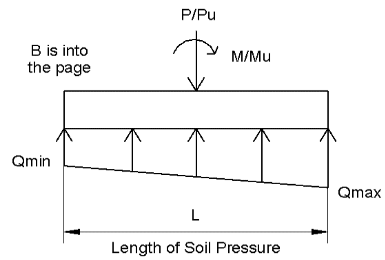
ΦPn =	162.435	k	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	5.89	k	(From Above)
Required Dead Load =	9.82	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	4.3552896	k	
Additional Slab Used =	5	ft	(Length of Additional Slab Past Edge of Footing in Each Direction
Additional Slab Area =	145.44	ft^2	From Each Edge of Footing)
Additional Slab Weight =	7.272	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	11.63	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Footing Designation: P2

Footing Location: Truss Shade Exterior



General Information:

Footing Diameter, D =	36	in		
Footing Depth, H =	72	in		
Location =	Corner	in		
Steel Depth, d =	68.0625	in		(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in		
Slab Depth Above Footing =	4	in		
Area of Footing =	3.139984	ft^2		(B*L)
Soil Bearing Pressure =	2	ksf		
Allowable or Effective SBC?	Allowable			
Concrete Strength =	3	ksi		
	B Direction			L Direction
Column Size =	3.00 in		X	3.00 in
Base Plate Size =	7.00 in		X	7.00 in
Critical Section =	5.00 in		X	5.00 in

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	0.25	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	2.8652354	k	Live =	1.6	
Applied Live Load =	0	k	Uplift=	1.6	
ASD Total Load, P =	3.1152354	k			
LRFD Total Load, Pu =	3.73828248	k			
ASD Uplift Load =	2.44	k			
LRFD Uplift Load =	3.904	k			

***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accc**

Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.6	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.295	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	1.772	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	0.992	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.992	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(ASD M / Pu)
Kern =	0.295	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	1.772	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.191	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.191	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.191	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	-2.158	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	-4.55	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	118.91	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	73.06	in	(Critical Section B + d)
b2 =	73.06	in	(Column Height L + d)
b0 =	292.25	in	(2*b1 + 2*b2)
Vu2 =	-40.40	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	4902.70	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	5440.22	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	3268.47	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	162.435	k	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

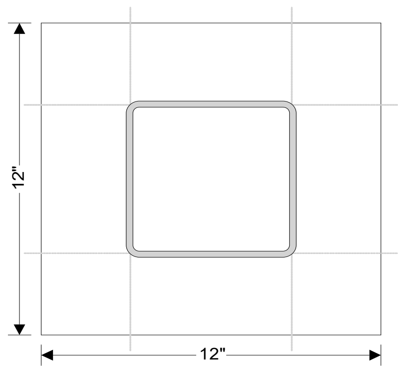
Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	2.44	k	(From Above)
Required Dead Load =	4.07	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	3.1152354	k	
Additional Slab Used =	4	ft	(Length of Additional Slab Past Edge of Footing in Each Direction
Additional Slab Area =	23.088	ft^2	From Each Edge of Footing)
Additional Slab Weight =	1.1544	k	
Wall Weight Over Footing =	0	klf	
Applied Wall Load on Footing =	0	k	
Length Parallel to Slab Edge =	0	ft	(B or L Depending on the Case)
Length Perpendicular to Slab Edge =	0	ft	(B or L Depending on the Case)
Area of Cont. Footing =	0	ft^2	
Length of Cont. Footing/Wall Used =	0	ft	This is TOTAL length of wall and continuous footing.
Wall + Cont. Footing Load =	0	k	
Total Dead Load =	4.27	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

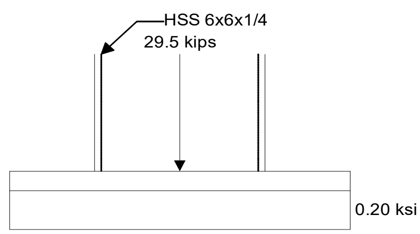
COLUMN BASE PLATE DESIGN

In accordance with AISC Steel Design Guide 1 and AISC 360-05

Tedds calculation version 2.1.02



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force	P_u = **29.5** kips (Compression)
Bending moment	M_u = **0.0** kip_in
Shear force	F_v = **0.0** kips

Column details

Column section	HSS 6x6x1/4
Depth	d = **6.000** in
Breadth	b_f = **6.000** in
Thickness	t = **0.233** in

Baseplate details

Depth	N = **12.000** in
Breadth	B = **12.000** in
Thickness	t_p = **0.750** in
Design strength	F_y = **36.0** ksi

Foundation geometry

Member thickness	h_a = **12.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **30.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **30.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **30.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **30.000** in
Minimum tensile strength, base plate	F_y = **36** ksi
Minimum tensile strength, column	F_{yCol} = **46** ksi
Compressive strength of concrete	f'_c = **3** ksi

Strength reduction factors

Compression	ϕ_c = **0.60**
Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**

Plate cantilever dimensions

Area of base plate	$A_1 = B \times N =$ **144.000** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min}) \times (B + 2 \times l_{min}) =$ **3600.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2) =$ **734.4** kips

Bending line cantilever distance m	$m = (N - 0.95 \times d) / 2 =$ **3.150** in
Bending line cantilever distance n	$n = (B - 0.95 \times b_f) / 2 =$ **3.150** in
Maximum bending line cantilever	$l = \max(m, n) =$ **3.150** in

Plate thickness

Required plate thickness	$t_{p,req} = l \times \sqrt{((2 \times P_u) / (\phi_b \times F_y \times B \times N))} =$ **0.354** in
Specified plate thickness	$t_p =$ **0.750** in

PASS - Thickness of plate exceeds required thickness

Design bearing strength (AISC 360-05-J8)

Design bearing strength	$P_p =$ **734.40** kips
Factored bearing strength	$\phi_c P_p =$ **440.64** kips

PASS - Allowable bearing stress exceeds applied bearing stress

Flange weld

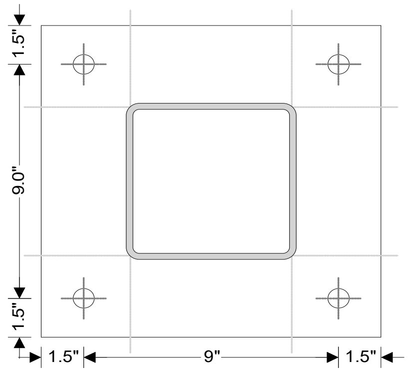
Flange weld leg length	$t_{wf} =$ **0.2500** in
Tension capacity of flange	$P_{tf} = b_f \times t \times F_{yCol} =$ **64.3** kips
Force in tension flange	$F_{tf} = M_u / (d - t) - P_u \times (b_f \times t) / A_{col} =$ **-7.9** kips
Critical force in flange	$F_f = \min(P_{tf}, \max(F_{tf}, 0\text{kips})) =$ **0.0** kips
Flange weld force per in	$R_{wf} = F_f / b_f =$ **0.0** kips/in
Electrode classification number	$F_{EXX} =$ **70.0** ksi
Design weld stress	$\phi F_w = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90\text{deg}))^{1.5}) =$ **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_w \times t_{wf} / \sqrt{(2)} =$ **8.4** kips/in

PASS - Available strength of flange weld exceeds force in flange weld

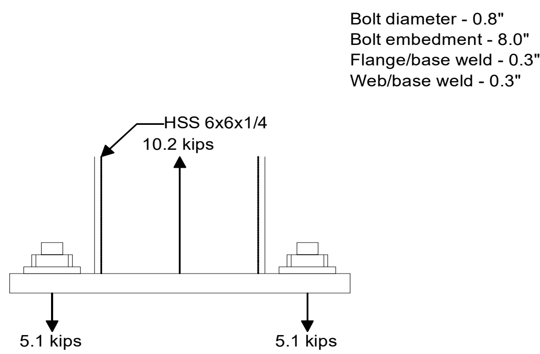
COLUMN BASE PLATE DESIGN

In accordance with AISC Steel Design Guide 1 and AISC 360-05

Tedds calculation version 2.1.02



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force	P_u = **-10.2** kips (Tension)
Bending moment	M_u = **0.0** kip_in
Shear force	F_v = **0.0** kips
Eccentricity	e = ABS(M_u / P_u) = **0.000** in
Anchor bolt to center of plate	f = 0in = **0.000** in

Column details

Column section	HSS 6x6x1/4
Depth	d = **6.000** in
Breadth	b_f = **6.000** in
Thickness	t = **0.233** in

Baseplate details

Depth	N = **12.000** in
Breadth	B = **12.000** in
Thickness	t_p = **0.750** in
Design strength	F_y = **36.0** ksi

Foundation geometry

Member thickness	h_a = **12.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **30.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **30.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **30.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **30.000** in

Holding down bolt and anchor plate details

Total number of bolts	N_{bolt} = **4**
Bolt diameter	d_o = **0.750** in
Bolt spacing	s_{bolt} = **9.000** in
Edge distance	e_1 = **1.500** in
Minimum tensile strength, base plate	F_y = **36** ksi
Minimum tensile strength, column	F_{yCol} = **46** ksi
Compressive strength of concrete	f'_c = **3** ksi

Strength reduction factors

Compression	ϕ_c = **0.60**

Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**

Bolt tension force

Tension force in one half of bolts	$T_u = ABS(P_u) / 2$ = **5.1** kips
Max tension is single bolt	$T_{rod} = T_u / N_{bolty}$ = **2.6** kips
Compression force in concrete	$f_{p,max}$ = 0 ksi

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines	$x = abs((N - 0.95 \times d) / 2 - e_1)$ = **1.650** in
Plate thickness required	$t_{p,req} = 2.11 \times \sqrt{((T_u \times x)/(B \times F_y))}$ = **0.294** in

PASS - Thickness of plate exceeds required thickness

Tension weld

Tension flange weld leg length	t_{wf} = **0.2500** in
Effective flange weld width	$l_{Tweld,eff} = l_{Tweld,eff\_ud}$ = **2** in
Tensile load per inch	$R_{wf} = T_{rod} / l_{Tweld,eff}$ = **1.3** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	$\phi F_w = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5})$ = **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_w \times t_{wf} / \sqrt{(2)}$ = **8.4** kips/in

PASS - Available strength of weld exceeds force in tension weld

Local stress on flange	$f_{T,local} = (T_{rod}) / (l_{Tweld,eff} \times t_f)$ = **5.472** ksi
Column flange allowable stress	$F_{yCol} / 1.67$ = **27.545** ksi

PASS - Local column capacity exceeds local column stress

ANCHOR BOLT DESIGN

In accordance with ACI318-08

Tedds calculation version 2.1.02

Anchor bolt geometry

Anchor bolt	Cast-in headed stud anchor
Diameter of anchor bolt	d_a = **0.75** in
Number of bolts in x direction	N_{boltx} = **2**
Number of bolts in y direction	N_{bolty} = **2**
Total number of bolts	$n_{total} = (N_{boltx} \times 2) + (N_{bolty} - 2) \times 2$ = **4**
Total number of bolts in tension	$n_{tens} = (N_{boltN} \times 2) + (N_{bolty} - 2) \times 2$ = **4**
Spacing of bolts in x direction	s_{boltx} = **9** in
Spacing of bolts in y direction	s_{bolty} = **9** in
Effective cross-sectional area of anchor	$A_{se} = \pi \times d_a^2 / 4$ = **0.442** in^2
Embedded depth of each anchor bolt	h_{ef} = **8** in

Material details

Minimum yield strength of steel	f_{ya} = **36** ksi
Nominal tensile strength of steel	f_{uta} = **43.5** ksi
Compressive strength of concrete	f'_c = **3** ksi
Concrete modification factor	λ = **1.00**

Strength reduction factors

Tension of steel element	$\phi_{t,s}$ = **0.75**
Shear of steel element	$\phi_{v,s}$ = **0.65**
Concrete tension	$\phi_{t,c}$ = **0.65**
Concrete shear	$\phi_{v,c}$ = **0.70**
Concrete tension for pullout	$\phi_{t,cB}$ = **0.70**

Concrete shear for pryout $\phi_{v,cB}$ = **0.70**

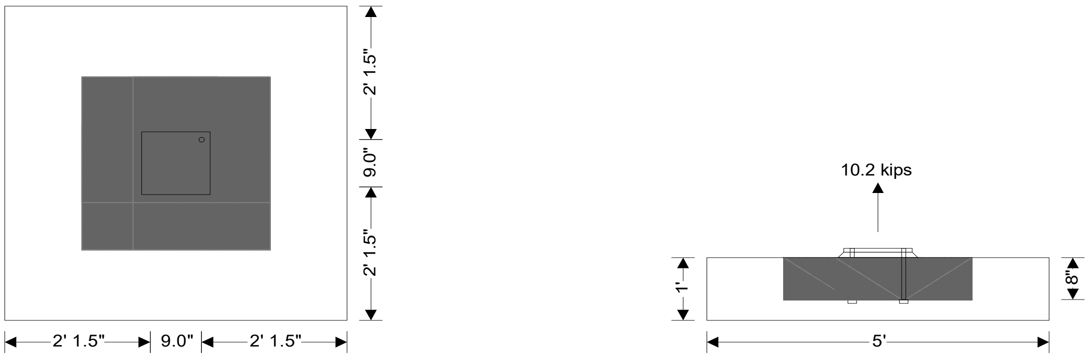
Steel strength of anchor in tension (D.5.1)

Nominal strength of anchor in tension $N_{sa} = A_{se} \times f_{uta}$ = **19.22** kips

Steel strength of anchor in tension $\phi N_{sa} = \phi_{t,s} \times N_{sa}$ = **14.41** kips

PASS - Steel strength of anchor exceeds max tension in single bolt

Check concrete breakout strength of anchor bolt in tension (D.5.2)



Concrete breakout - tension

Coeff for basic breakout strength in tension k_c = **24**

Breakout strength for single anchor in tension $N_b = k_c \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times h_{ef}^{1.5} \times 1 \text{ in}^{0.5}$ = **29.74** kips

Projected area for groups of anchors A_{Nc} = **1089** in^2

Projected area of a single anchor $A_{Nco} = 9 \times h_{ef}^2$ = **576** in^2

Min dist center of anchor to edge of concrete $c_{a,min}$ = **25.5** in

Mod factor for groups loaded eccentrically $\psi_{ec,N} = \min(1 / (1 + ((2 \times e'_N) / (3 \times h_{ef}))), 1)$ = **1.000**

Modification factor for edge effects $\psi_{ed,N}$ = 1.0 = **1.000**

Modification factor for no cracking at service loads $\psi_{c,N}$ = **1.000**

Modification factor for cracked concrete $\psi_{cp,N}$ = **1.000**

Nominal concrete breakout strength $N_{cbg} = A_{Nc} / A_{Nco} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$ = **56.24** kips

Concrete breakout strength $\phi N_{cbg} = \phi_{t,c} \times N_{cbg}$ = **36.55** kips

PASS - Breakout strength exceeds tension in bolts

Pullout strength (D.5.3)

Net bearing area of the head of anchor A_{brg} = **0.9** in^2

Mod factor for no cracking at service loads $\psi_{c,P}$ = **1.000**

Pullout strength for single anchor $N_p = 8 \times A_{brg} \times f'_c$ = **21.60** kips

Nominal pullout strength of single anchor $N_{pn} = \psi_{c,P} \times N_p$ = **21.60** kips

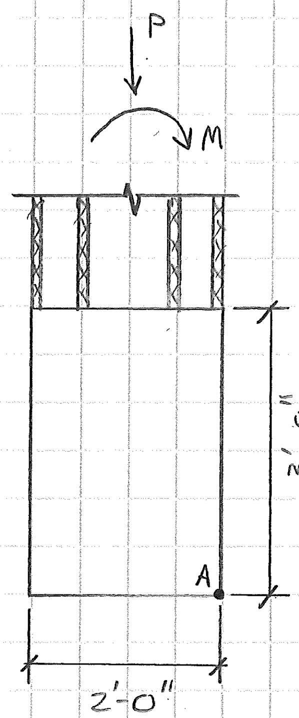
Pullout strength of single anchor $\phi N_{pn} = \phi_{t,cB} \times N_{pn}$ = **15.12** kips

PASS - Pullout strength of single anchor exceeds maximum axial force in single bolt

Side face blowout strength (D.5.4)

As $h_{ef} \le 2.5 \times \min(c_{a1}, c_{a2})$ the edge distance is considered to be far from an edge and blowout strength need not be considered

MONUMENT SIGN FOUNDATION



SIGN = 5 psf
CMU = 47 psf
CAST STONE = 150 pcf

DEAD LOAD

SIGN = 5 psf $(4^{ft})(8^{ft})$ = 160^{lbs}
CMU = 47 psf $(8^{ft} + 8^{ft} + 2^{ft} + 2^{ft})(16''/12)$ = $1,253^{lbs}$
STONE = 150 pcf $(8''/12)(2.17')(8.17')$ = $1,769^{lbs}$

P_{DL} = 160 + 1253 + 1769 = $3,183^{lbs}$

WIND LOAD

F = 1.0^{k} @ 3.3^{ft} ABOVE GRADE

M = $1.0^{k}(3.3^{ft})$ = 3.3 ft-k

OVERTURNING

$M_{DL} = 3.183^{lbs}(1^{ft}) = 3.183^{ft\text{-}k}$
$M_{FTG} = 150_{pcf}(2^{ft})(3^{ft})(8^{ft})(1^{ft}) = 7.2^{ft\text{-}k}$

$$F.S. = \frac{0.6(3.183+7.2)}{0.6(3.3)} = 3.15 > 1.5 \quad \text{O.K.}$$

BEARING

ALLOWABLE BEARING PRESSURE = 1.2^{ksf}

$$q_u = \frac{3.183^{k}}{2^{ft}(8^{ft})} = 0.199^{ksf}$$

$$F.S. = \frac{1.2}{0.199} = 6 > 2.0 \quad \text{O.K.}$$

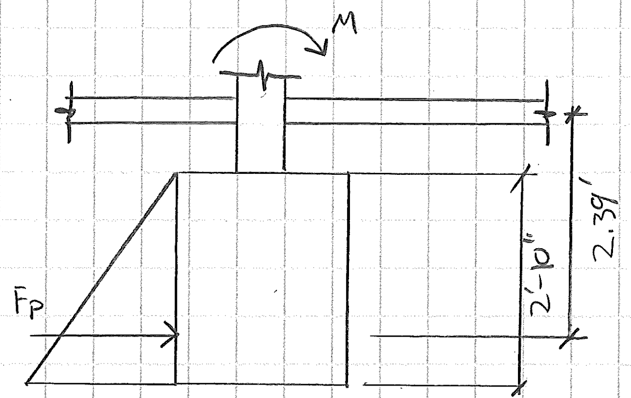
BSE STRUCTURAL ENGINEERS

Project WESTLAKE - LEE'S SUMMIT Project No. 22-365

Calc. By KMB Checked By JH Date 10/14/2022

TRASH ENCLOSURE FOUNDATION

BASE MOMENT = 2.6k*4.4ft / 12.083' = 0.947 $^{ft\text{-}k/ft}$ (FACTORED)



*ASSUME SLAB PINS WALL + PASSIVE PRESSURE RESISTS O.T.

$\gamma = 125pcf \qquad K_p = 2.2$

$F_p = \frac{1}{2}(125pcf)(2.83^{ft})^2(2.2) = 1{,}101^{\,lb/ft} = 1.1^{\,k/ft}$

WIND LOAD = 0.947 $^{ft\text{-}k/ft}$ / 2.39' = 0.396 $^{k/ft}$ T OR C

$0.6(1.1^{k/ft}) = 0.661^{\,k/ft} > 0.396^{\,k/ft}$ O.K.

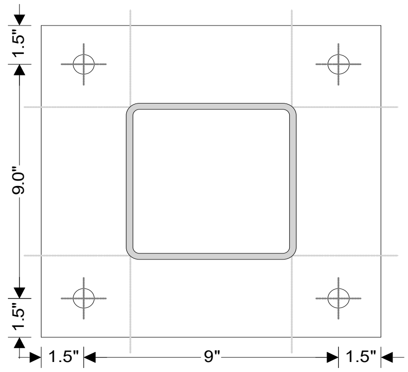
COLUMN BASE PLATE DESIGN

REAR HSS CANOPY

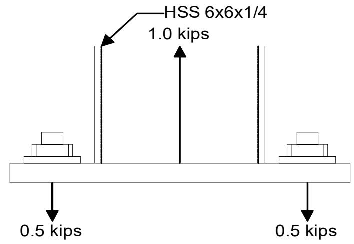
In accordance with AISC Steel Design Guide 1 and AISC 360-05

Tedds calculation version 2.1.02

Bolt diameter - 0.8"
Bolt embedment - 8.0"
Flange/base weld - 0.3"
Web/base weld - 0.3"



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force	P_u = **-1.0** kips (Tension)
Bending moment	M_u = **0.0** kip_in
Shear force	F_v = **0.0** kips
Eccentricity	e = ABS(M_u / P_u) = **0.000** in
Anchor bolt to center of plate	f = 0in = **0.000** in

Column details

Column section	HSS 6x6x1/4
Depth	d = **6.000** in
Breadth	b_f = **6.000** in
Thickness	t = **0.233** in

Baseplate details

Depth	N = **12.000** in
Breadth	B = **12.000** in
Thickness	t_p = **0.750** in
Design strength	F_y = **36.0** ksi

Foundation geometry

Member thickness	h_a = **24.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **8.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **8.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **8.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **8.000** in

Holding down bolt and anchor plate details

Total number of bolts	N_{bolt} = **4**
Bolt diameter	d_o = **0.750** in
Bolt spacing	s_{bolt} = **9.000** in
Edge distance	e_1 = **1.500** in
Minimum tensile strength, base plate	F_y = **36** ksi
Minimum tensile strength, column	F_{yCol} = **46** ksi
Compressive strength of concrete	f'_c = **3** ksi

Strength reduction factors

Compression	ϕ_c = **0.60**

Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**

Bolt tension force

Tension force in one half of bolts	T_u = ABS(P_u) / 2 = **0.5** kips
Max tension is single bolt	T_{rod} = T_u / N_{bolty} = **0.3** kips
Compression force in concrete	$f_{p,max}$ = 0 ksi

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines	x = abs((N - 0.95 × d) / 2 - e_1) = **1.650** in
Plate thickness required	$t_{p,req}$ = 2.11 × √((T_u × x)/(B × F_y)) = **0.092** in

PASS - Thickness of plate exceeds required thickness

Tension weld

Tension flange weld leg length	t_{wf} = **0.2500** in
Effective flange weld width	$l_{Tweld,eff}$ = l_{Tweld,eff_ud} = **2** in
Tensile load per inch	R_{wf} = T_{rod} / $l_{Tweld,eff}$ = **0.1** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	ϕF_w = ϕ_v × 0.60 × F_{EXX} × (1.0 + 0.5 × (sin(90deg))$^{1.5}$) = **47.250**ksi
Design strength of weld per in	ϕR_{nf} = ϕF_w × t_{wf} / √(2) = **8.4** kips/in

PASS - Available strength of weld exceeds force in tension weld

Local stress on flange	$f_{T,local}$ = (T_{rod}) / ($l_{Tweld,eff}$ × t_f) = **0.536** ksi
Column flange allowable stress	F_{yCol} / 1.67 = **27.545** ksi

PASS - Local column capacity exceeds local column stress

<u>ANCHOR BOLT DESIGN</u>

In accordance with ACI318-08

Tedds calculation version 2.1.02

Anchor bolt geometry

Anchor bolt	Cast-in headed stud anchor
Diameter of anchor bolt	d_a = **0.75** in
Number of bolts in x direction	N_{boltx} = **2**
Number of bolts in y direction	N_{bolty} = **2**
Total number of bolts	n_{total} = (N_{boltx} × 2) + (N_{bolty} - 2) × 2 = **4**
Total number of bolts in tension	n_{tens} = (N_{boltN} × 2) + (N_{bolty} - 2) × 2 = **4**
Spacing of bolts in x direction	s_{boltx} = **9** in
Spacing of bolts in y direction	s_{bolty} = **9** in
Effective cross-sectional area of anchor	A_{se} = π × d_a^2 / 4 = **0.442** in^2
Embedded depth of each anchor bolt	h_{ef} = **8** in

Material details

Minimum yield strength of steel	f_{ya} = **36** ksi
Nominal tensile strength of steel	f_{uta} = **43.5** ksi
Compressive strength of concrete	f'_c = **3** ksi
Concrete modification factor	λ = **1.00**

Strength reduction factors

Tension of steel element	$\phi_{t,s}$ = **0.75**
Shear of steel element	$\phi_{v,s}$ = **0.65**
Concrete tension	$\phi_{t,c}$ = **0.65**
Concrete shear	$\phi_{v,c}$ = **0.70**
Concrete tension for pullout	$\phi_{t,cB}$ = **0.70**

Concrete shear for pryout $\phi_{v,cB}$ = **0.70**

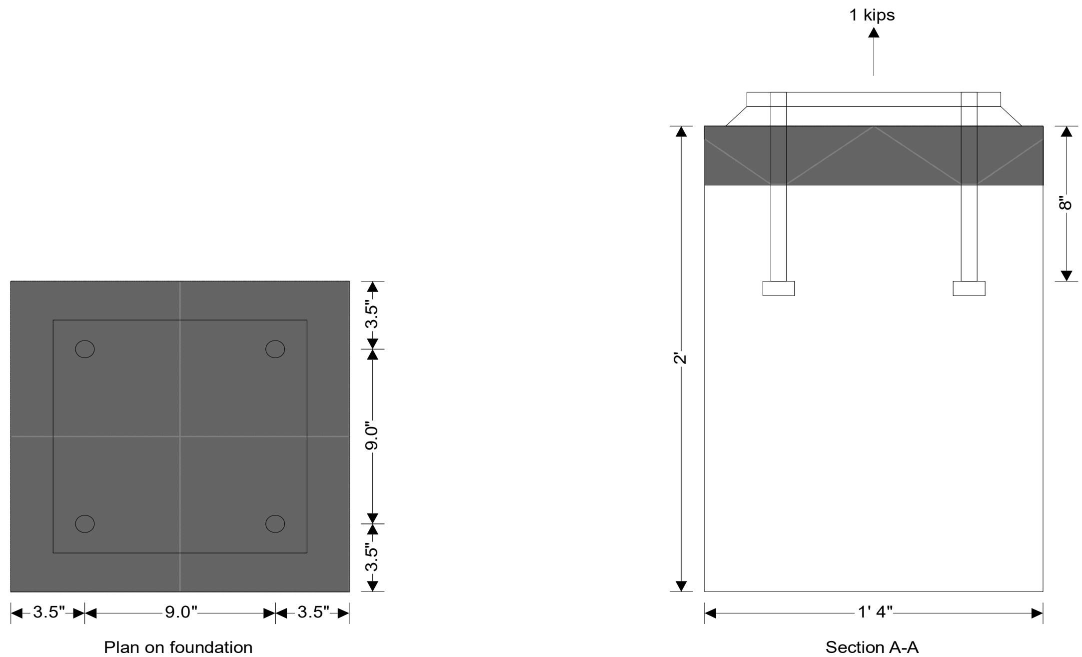
Steel strength of anchor in tension (D.5.1)

Nominal strength of anchor in tension $N_{sa} = A_{se} \times f_{uta}$ = **19.22** kips

Steel strength of anchor in tension $\phi N_{sa} = \phi_{t,s} \times N_{sa}$ = **14.41** kips

PASS - Steel strength of anchor exceeds max tension in single bolt

Check concrete breakout strength of anchor bolt in tension (D.5.2)



Concrete breakout - tension

The anchors are located at less than 1.5h_{ef} from 4 edges. Therefore the effective embedded depth has to be limited to 3.00" in accordance with D.5.2.3

Limiting embedded depth $h_{ef,lim}$ = **3.00** in

Coeff for basic breakout strength in tension k_c = **24**

Breakout strength for single anchor in tension $N_b = k_c \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times h_{ef,lim}^{1.5} \times 1 \text{ in}^{0.5}$ = **6.83** kips

Projected area for groups of anchors A_{Nc} = **256** in^2

Projected area of a single anchor $A_{Nco} = 9 \times h_{ef,lim}^2$ = **81** in^2

Min dist center of anchor to edge of concrete $c_{a,min}$ = **3.5** in

Mod factor for groups loaded eccentrically $\psi_{ec,N} = \min(1 / (1 + ((2 \times e'_N) / (3 \times h_{ef,lim}))), 1)$ = **1.000**

Modification factor for edge effects $\psi_{ed,N} = 0.7 + 0.3 \times (c_{a,min} / (1.5 \times h_{ef,lim}))$ = **0.933**

Modification factor for no cracking at service loads $\psi_{c,N}$ = **1.000**

Modification factor for cracked concrete $\psi_{cp,N}$ = **1.000**

Nominal concrete breakout strength $N_{cbg} = A_{Nc} / A_{Nco} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$ = **20.15** kips

Concrete breakout strength $\phi N_{cbg} = \phi_{t,c} \times N_{cbg}$ = **13.10** kips

PASS - Breakout strength exceeds tension in bolts

Pullout strength (D.5.3)

Net bearing area of the head of anchor A_{brg} = **0.9** in^2

Mod factor for no cracking at service loads $\psi_{c,P}$ = **1.000**

Pullout strength for single anchor $N_p = 8 \times A_{brg} \times f'_c$ = **21.60** kips

Nominal pullout strength of single anchor $N_{pn} = \psi_{c,P} \times N_p$ = **21.60** kips

Pullout strength of single anchor $\phi N_{pn} = \phi_{t,cB} \times N_{pn} =$ **15.12** kips

PASS - Pullout strength of single anchor exceeds maximum axial force in single bolt

Side face blowout strength (D.5.4)

As h_{ef} <= 2.5 $\times$ min(c_{a1}, c_{a2}) the edge distance is considered to be far from an edge and blowout strength need not be considered

Summary

The lateral stability system of the project referenced above consists of metal deck diaphragms, and exterior masonry shear walls. The roof diaphragm is designed to transfer lateral loads from the exterior walls into the shear walls. Exterior shear walls allow for the direct transfer of lateral forces into the foundation.

The following section of calculations covers the complete design of the lateral stability system for the project referenced above including the distribution of lateral forces into the individual elements. Refer to the "Loads" section of these calculations for the determination of all wind and seismic loads.

Loads
DL = 15psf
LLr = 20psf
Flat SL = 14psf
Drift SL = 59.8psf peak over 11ft

West/East Wall
Trib Width = 29.67 ft / 2 = 14.84 ft

Loading
DL = 15psf*14.84ft = 0.223 klf
LLr = 20psf*14.84ft = 0.297 klf
SL = 14psf*14.84ft + (59.8psf-14psf)(11ft/2)(29.67ft - 11ft/3)/29.67ft = 0.429 klf

Loads
DL = 15psf
LLr = 20psf
Flat SL = 14psf
Drift SL = 59.8psf peak over 11ft

North/South Wall
Trib Width = 6 ft

Distributed Loading
DL = 15psf*6ft = 0.09 klf
LLr = 20psf*6ft = 0.12 klf
SL = 14psf*6ft = 0.084 klf

Concentrated Loads

Reactions from South Beam, Short
DL = 2.25k / 2 = 1.125k
LLr = 3k / 2 = 1.5k
SL = 5k / 2 = 2.5k
Reactions from 15ft of Girder
DL = 0.433 klf*15ft = 6.50k
LLr = 0.346 klf*15ft = 5.19k
SL = 0.404 klf*15ft = 6.06k

BSE STRUCTURAL ENGINEERS

Project Westlake - Edmond, OK Project No. 19-083

Calc. By RJD Checked By ________ Date 4/3/2019

Wind Load Distribution - Single Diaphragm Design, Envelope Method

General Building Info :

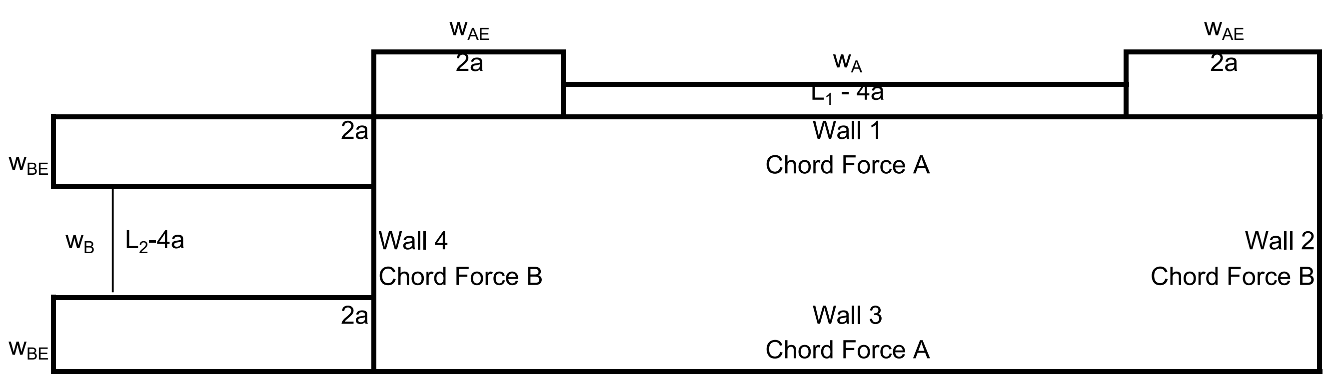
	Length	Wall Height to Roof	Parapet Height	Total Wall Height
Wall 1 =	112.00 ft	17.34 ft	7.33 ft	24.67 ft
Wall 2 =	136.00 ft	18.67 ft	6.00 ft	24.67 ft
Wall 3 =	112.00 ft	17.34 ft	7.33 ft	24.67 ft
Wall 4 =	136.00 ft	16.00 ft	8.67 ft	24.67 ft

General Wind Loading Info :

ASD Factor =	0.6	(1 for ACSE 7-05, 0.6 for ASCE 7-10)
2a =	13.88 ft	(2 x Length of End Zone per ACSE)
Zone 1 Pressure =	5.00 psf	(Refer to Tedd's Calculations for Pressures)
Zone 1E Pressure =	9.70 psf	
Zone 4 Pressure =	10.60 psf	
Zone 4E Pressure =	13.80 psf	
WW Parapet Pressure =	36.35 psf	
LW Parapet Pressure =	24.23 psf	

Wind Loading @ Roof:

L_1 - 4a =	84.24 ft	
Distributed Force, w_A =	579.30 plf	(Pressures are Based on Entered Wind Pressures)
Distributed Force, w_{AE} =	647.80 plf	
L_2 - 4a =	108.24 ft	
Distributed Force, w_B =	599.49 plf	(Pressures are Based on Entered Wind Pressures)
Distributed Force, w_{BE} =	666.96 plf	



Shear Wall Loads:

Wall 1 =	25.0 k ASD	x Factor =	41.7 k LRFD	(Distributed Load x Effective Trib)
Wall 2 =	20.0 k ASD	x Factor =	33.4 k LRFD	(Distributed Load x Effective Trib)
Wall 3 =	25.0 k ASD	x Factor =	41.7 k LRFD	(Distributed Load x Effective Trib)
Wall 4 =	20.0 k ASD	x Factor =	33.4 k LRFD	(Distributed Load x Effective Trib)

Diaphragm Loads:

Wall 1 =	223.4 plf ASD	x Factor =	372.3 plf LRFD	(Wall 1 Load*1000/Wall 1 Length)
Wall 2 =	147.3 plf ASD	x Factor =	245.5 plf LRFD	(Wall 2 Load*1000/Wall 2 Length)
Wall 3 =	223.4 plf ASD	x Factor =	372.3 plf LRFD	(Wall 3 Load*1000/Wall 3 Length)
Wall 4 =	147.3 plf ASD	x Factor =	245.5 plf LRFD	(Wall 4 Load*1000/Wall 4 Length)

Deck Design:

Deck =	22 GA Type B Metal Roof deck	(Per Vulcraft Catalog)
Weld Pattern =	36/7	(Per Vulcraft Catalog)
Sidelap Fasteners =	(2) #10 TEK Screws	(Per Vulcraft Catalog)
Allowable Load =	359.0 plf ASD	(Per Vulcraft Catalog)
Max Diaphragm Load =	223.4 plf ASD	
	OK	

Chord Loads:

Chord Force A =	4.0 k ASD	x Factor =	6.7 k LRFD	(M_{wind} against A / Wall B Length)
Chord Force B =	7.5 k ASD	x Factor =	12.4 k LRFD	(M_{wind} against B / Wall A Length)

Chord Design:

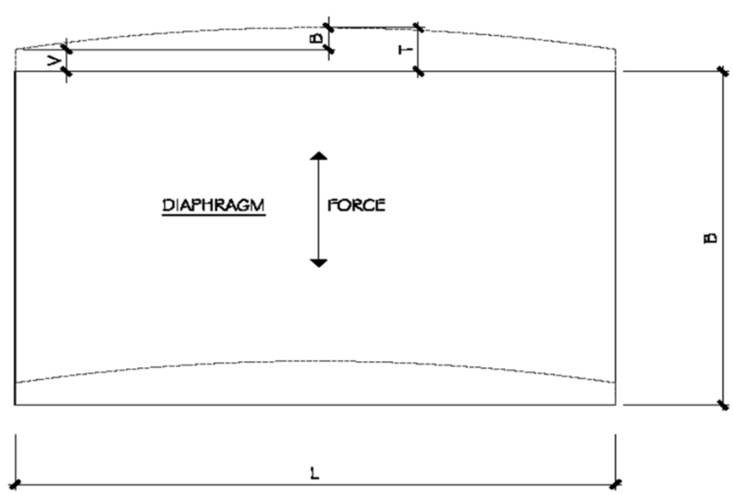
Chord A Design:		
Angle =	L3x3x1/4	
Angle Area =	1.4 in^2	(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	46.7 k LRFD	(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	21.0 k LRFD	(AISC Manual Table 4-11)
	OK	
Chord B Design:		
Angle =	L3x3x1/4	
Angle Area =	1.4 in^2	(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	46.7 k LRFD	(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	21.0 k LRFD	(AISC Manual Table 4-11)
	OK	

Diaphragm Deflection Calculations : Simply Supported

Span	Max Joist Spacing (ft)	L (ft)	B (ft)	E (ksi)	Chord Area A (in^2)
Wall 2 to 4	6	112.00	136.00	29000	1.44
Wall 1 to 3	6	136.00	112.00	29000	1.44

Values From Vulcraft Deck Catalog			Loading	Deflections		
K_2	DB	K_1	q(lb/ft)	ΔB (in)	ΔV (in)	ΔT (in)
870	129	0.308	347.58204	0.02	0.072658606	0.09
870	129	0.308	359.6913	0.07	0.134623949	0.21

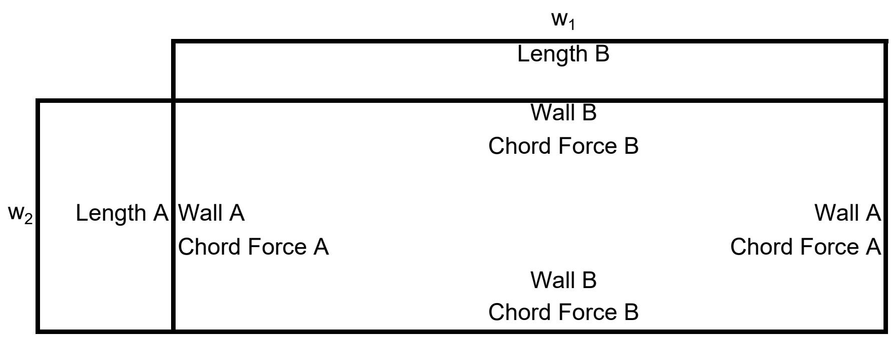
ΔB=	Deflection due to moment in inches	$5qL^4/384EI$
ΔV=	Deflection due to shear in inches	$qL^2/8BG'$
ΔT=	Total deflection in inches	ΔB+ΔV
q(lb/ft)=	distributed load on diaphragm	
K_1=	Factor from steel deck catalog	
DB=	Factor from steel deck catalog	
K_2 =	Factor from steel deck catalog	
A(in^2)=	Area of chord member	
E (ksi)=	Modulus of elasticity of steel	
B (ft)=	Diaphragm Depth	
L (ft)=	Diaphragm Span	
I (in^4)=	Moment of Inertia of diaphragm	$2A(B/2)^2$
G' (k/in) =	Effective shear modulus of decking	$K_2/(3.78+(0.38*DB/Span)+3*K_1*Span)$
Span =	Max Joist Spacing	



Seismic Load Distribution - Single Diaphragm Design

General Info :

Seismic Response Coeff. C_s =	0.0533	(from Tedds Seismic Load Calculations)
Roof Dead Load =	15.00 psf	
Wall Length A =	136.00 ft	
Wall Weight A =	47.00 psf	
Wall Height A =	16.00 ft	(Roof to FFE)
Parapet Height A =	8.67 ft	(Top of Parapet to Roof)
Distributed Force, w_1 =	188.90 plf	((Length A*Dead+(0.5*Height B+Parapet)*Weight B) * C_S)
Wall Length B =	112.00 ft	
Wall Weight B =	47.00 psf	
Wall Height B =	17.34 ft	(Roof to FFE)
Parapet Height B =	7.33 ft	(Top of Parapet to Roof)
Distributed Force, w_2 =	173.06 plf	((Length B*Dead+(0.5*Height A+Parapet)*Weight A) * C_S)



Shear Wall Loads from Diaphragm:

Wall A =	7.4 k ASD	/0.7 =	10.6 k LRFD	((0.7*w_1*Length B*0.5)/1000)
Wall B =	8.2 k ASD	/0.7 =	11.8 k LRFD	((0.7*w_2*Length A*0.5)/1000)

Total Shear Wall Loads (Includes Wall Weight):

Wall A =	13.3 k ASD	/0.7 =	19.0 k LRFD	(Dia. A+Length A*Weight A*Height A*C_S)
Wall B =	13.1 k ASD	/0.7 =	18.7 k LRFD	(Dia. B+Length B*Weight B*Height B*C_S)

Diaphragm Loads:

Wall A =	54.4 plf ASD	/0.7 =	77.8 plf LRFD	(Wall A Load*1000/Wall A Length)
Wall B =	73.6 plf ASD	/0.7 =	105.1 plf LRFD	(Wall B Load*1000/Wall B Length)

Deck Design:

Deck =	22 GA Type B Metal Roof deck	(Per Vulcraft Catalog)
Weld Pattern =	36/7	(Per Vulcraft Catalog)
Sidelap Fasteners =	(2) #10 TEK Screws	(Per Vulcraft Catalog)
Allowable Load =	359.0 plf ASD	(Per Vulcraft Catalog)
Max Diaphragm Load =	73.6 plf ASD	
	OK	

Chord Loads:

Chord Force A =	2.5 k ASD	/0.7 =	3.6 k LRFD	(M_{wind} against A / Wall B Length)
Chord Force B =	1.5 k ASD	/0.7 =	2.2 k LRFD	(M_{wind} against B / Wall A Length)

Chord Design:

Chord A Design:			
Angle =	L3x3x1/4		
Angle Area =	1.4 in^2		(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	46.7 k LRFD		(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	21.0 k LRFD	k	(AISC Manual Table 4-11)
	OK		
Chord B Design:			
Angle =	L3x3x1/4		
Angle Area =	1.4 in^2		(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	46.7 k LRFD		(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	21.0 k LRFD	k	(AISC Manual Table 4-11)
	OK		

Bentley

Current Date: 11/7/2022 2:05 PM
Units system: English
File name: G:\2022\22-365 Westlake Ace Hardware - Lee's Summit, MO\Calculations\D - Lateral\North Wall.msw

Design Results

Masonry wall

NORTH WALL

GENERAL INFORMATION:

**Global status : ** OK

Design code	:	TMS 402-13 SD

Geometry:

Total height	:	24.67 [ft]
Total length	:	115.34 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Materials:

Material	:	CMU 1.5-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Partial grouting
Mortar bed type	:	Full bed
Masonry compression strength (F`m)	:	1.5 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	24 [Kip/in2]
Joint reinforcement allowable tension strength (Fs)	:	30 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1350 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC B
Response modification factor	:	2.00
Shear wall type	:	Ordinary

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	17.34	7.63	0.09

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	10.00	0.00	3.34	8.00
Lower left	88.65	0.00	9.34	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LLR	Roof Live Load
SL	No	SNOW	Snow Load
Wx	No	WIND	Wind in X
Wz	No	WIND	Wind in Z
EQx	No	EQ	Seismic in X
EQz	No	EQ	Seismic in Z

D1	Yes	1.4DL
D2	Yes	1.2DL+0.5LL
D3	Yes	1.2DL+0.5SL
D4	Yes	1.2DL+1.6LL
D5	Yes	1.2DL+1.6SL
D6	Yes	1.2DL+0.5Wx
D7	Yes	1.2DL+0.5Wz
D8	Yes	1.2DL+1.6LL+0.5Wx
D9	Yes	1.2DL+1.6LL+0.5Wz
D10	Yes	1.2DL+1.6SL+0.5Wx
D11	Yes	1.2DL+1.6SL+0.5Wz
D12	Yes	1.2DL+Wx
D13	Yes	1.2DL+Wz
D14	Yes	1.2DL+Wx+0.5LL
D15	Yes	1.2DL+Wz+0.5LL
D16	Yes	1.2DL+Wx+0.5SL
D17	Yes	1.2DL+Wz+0.5SL
D18	Yes	1.2DL+0.2SL
D19	Yes	1.2DL+EQx
D20	Yes	1.2DL+EQz
D21	Yes	1.2DL+EQx+0.2SL
D22	Yes	1.2DL+EQz+0.2SL
D23	Yes	0.9DL+Wx
D24	Yes	0.9DL+Wz
D25	Yes	0.9DL+EQx
D26	Yes	0.9DL+EQz
S1	Yes	DL
S2	Yes	DL+LL
S3	Yes	DL+SL
S4	Yes	DL+0.75LL
S5	Yes	DL+0.75SL
S6	Yes	DL+0.6Wx
S7	Yes	DL+0.6Wz
S8	Yes	DL+0.7EQx
S9	Yes	DL+0.7EQz
S10	Yes	DL+0.525EQx
S11	Yes	DL+0.525EQz
S12	Yes	DL+0.75SL
S13	Yes	DL+0.525EQx+0.75SL
S14	Yes	DL+0.525EQz+0.75SL
S15	Yes	0.6DL+0.6Wx
S16	Yes	0.6DL+0.6Wz
S17	Yes	0.6DL+0.7EQx
S18	Yes	0.6DL+0.7EQz

Concentrated loads:

Story	Condition	Direction	Magnitude [Kip]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	6.50	0.00	28.00
1	DL	Vertical	6.50	0.00	56.00
1	DL	Vertical	6.50	0.00	84.00
1	LL	Vertical	5.19	0.00	28.00
1	LL	Vertical	5.19	0.00	56.00
1	LL	Vertical	5.19	0.00	84.00
1	DL	Vertical	1.13	0.00	34.17
1	LL	Vertical	1.50	0.00	34.17
1	DL	Vertical	1.13	0.00	56.00
1	LL	Vertical	1.50	0.00	56.00
1	DL	Vertical	1.13	0.00	77.83
1	LL	Vertical	1.50	0.00	77.83
1	SL	Vertical	6.06	0.00	28.00
1	SL	Vertical	6.06	0.00	56.00
1	SL	Vertical	6.06	0.00	84.00
1	SL	Vertical	2.50	0.00	34.17
1	SL	Vertical	2.50	0.00	56.00
1	SL	Vertical	2.50	0.00	77.83

Distributed loads:

Consider self weight : No

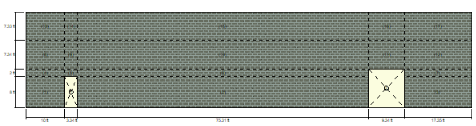
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	DL	Vertical	0.09	0.00
1	LL	Vertical	0.12	0.00
1	SL	Vertical	0.08	0.00
1	Wx	Horizontal	0.37	0.00
1	EQx	Horizontal	0.11	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wz	0.01
1	EQz	0.00
Parapet	Wz	0.04
Parapet	EQz	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	10.00	8.00
2	13.34	0.00	75.31	8.00
3	97.99	0.00	17.35	8.00
4	0.00	8.00	10.00	2.00
5	10.00	8.00	3.34	2.00
6	13.34	8.00	75.31	2.00
7	97.99	8.00	17.35	2.00
8	0.00	10.00	10.00	7.34
9	10.00	10.00	3.34	7.34
10	13.34	10.00	75.31	7.34
11	88.65	10.00	9.34	7.34
12	97.99	10.00	17.35	7.34
13	0.00	17.34	10.00	7.33
14	10.00	17.34	3.34	7.33
15	13.34	17.34	75.31	7.33
16	88.65	17.34	9.34	7.33
17	97.99	17.34	17.35	7.33

Vertical reinforcement:

Segment	Bars	Spacing [in]	Ld [in]
1	5-#4	24.00	25.17
2	38-#4	24.00	25.17
3	9-#4	24.00	25.17
4	5-#4	24.00	25.17
5	2-#4	24.00	25.17
6	38-#4	24.00	25.17

7	9-#4	24.00	25.17
8	5-#4	24.00	25.17
9	2-#4	24.00	25.17
10	38-#4	24.00	25.17
11	5-#4	24.00	25.17
12	9-#4	24.00	25.17
13	5-#4	24.00	25.17
14	2-#4	24.00	25.17
15	38-#4	24.00	25.17
16	5-#4	24.00	25.17
17	9-#4	24.00	25.17

Results: Combined axial flexure

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D26(Top)	0.98	0.73	0.73	16.50	0.04
2	D24(Top)	26.13	5.82	5.84	129.60	0.05
3	D24(Top)	2.51	1.63	1.63	28.86	0.06
4	D24(Top)	0.85	1.92	1.92	16.47	0.12
5	D24(Top)	0.31	0.60	0.60	5.51	0.11
6	D24(Top)	26.07	14.90	14.94	129.58	0.12
7	D24(Top)	2.07	3.56	3.57	28.74	0.12
8	D24(Top)	0.73	9.76	9.78	16.43	0.59
9	D24(Top)	0.22	3.32	3.33	5.48	0.61
10	D24(Top)	26.55	74.69	75.15	129.71	0.58
11	D24(Top)	0.57	9.24	9.25	15.31	0.60
12	D24(Top)	1.66	17.14	17.17	28.62	0.60
13	D17(Bottom)	-0.15	9.71	9.71	16.18	0.60
14	D24(Bottom)	0.25	3.31	3.31	5.49	0.60
15	D17(Bottom)	-0.85	76.12	76.12	121.96	0.62
16	D24(Bottom)	0.71	9.13	9.13	15.36	0.59
17	D24(Bottom)	0.20	16.99	16.99	28.21	0.60

Results: Flexural reinforcement area

Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D26(Max)	0.00	1.00	2.73	0.37
2	D26(Max)	0.00	7.53	20.56	0.37
3	D26(Max)	0.00	1.73	4.74	0.37
4	D26(Bottom)	0.00	1.00	2.73	0.37
5	D26(Bottom)	0.00	0.33	0.91	0.37
6	D26(Bottom)	0.00	7.53	20.56	0.37
7	D26(Bottom)	0.00	1.73	4.74	0.37
8	D26(Bottom)	0.00	1.00	2.73	0.37
9	D26(Bottom)	0.00	0.33	0.91	0.37
10	D26(Bottom)	0.00	7.53	20.56	0.37
11	D26(Bottom)	0.00	0.93	2.55	0.37
12	D26(Bottom)	0.00	1.73	4.74	0.37
13	D26(Bottom)	0.00	1.00	2.73	0.37
14	D26(Bottom)	0.00	0.33	0.91	0.37
15	D26(Bottom)	0.00	7.53	20.56	0.37
16	D26(Bottom)	0.00	0.93	2.55	0.37
17	D26(Bottom)	0.00	1.73	4.74	0.37

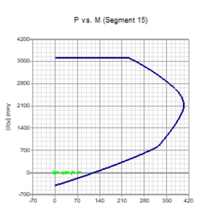
Intermediate results for axial-bending

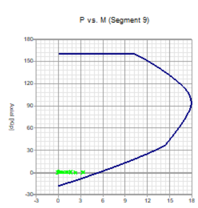
Segment	Condition	c [in]	d [in]	Mcr [Kip*ft]
1	D26(Top)	0.53	3.81	0.90
2	D24(Top)	0.55	3.81	0.93
3	D24(Top)	0.53	3.81	0.90
4	D24(Top)	0.53	3.81	0.89
5	D24(Top)	0.53	3.81	0.89
6	D24(Top)	0.55	3.81	0.93
7	D24(Top)	0.53	3.81	0.90
8	D24(Top)	0.53	3.81	0.89
9	D24(Top)	0.53	3.81	0.89
10	D24(Top)	0.55	3.81	0.93
11	D24(Top)	0.53	3.81	0.89
12	D24(Top)	0.53	3.81	0.90
13	D17(Bottom)	0.52	3.81	0.88
14	D24(Bottom)	0.53	3.81	0.89
15	D17(Bottom)	0.52	3.81	0.88
16	D24(Bottom)	0.53	3.81	0.89
17	D24(Bottom)	0.52	3.81	0.88

Inertias

Segment	Condition	Ig [in4]	Icr [in4]
1	D26(Top)	365.44	24.75
2	D24(Top)	365.44	25.52
3	D24(Top)	365.44	24.90
4	D24(Top)	365.44	24.71
5	D24(Top)	365.44	24.73
6	D24(Top)	365.44	25.52
7	D24(Top)	365.44	24.82
8	D24(Top)	365.44	24.67
9	D24(Top)	365.44	24.64
10	D24(Top)	365.44	25.54
11	D24(Top)	365.44	24.63
12	D24(Top)	365.44	24.74
13	D17(Bottom)	365.44	24.39
14	D24(Bottom)	365.44	24.67
15	D17(Bottom)	365.44	24.41
16	D24(Bottom)	365.44	24.68
17	D24(Bottom)	365.44	24.48

Interaction diagrams, P vs. M:





Results: Axial compression

Segment	Condition	Pu [Kip]	ϕPn [Kip]	Ratio
1	D9(Max)	4.02	318.50	0.01
2	D11(Max)	86.40	2398.61	0.04
3	D8(Bottom)	10.95	552.59	0.02
4	D4(Bottom)	3.56	318.50	0.01
5	D4(Bottom)	1.08	106.38	0.01
6	D10(Bottom)	85.14	2398.61	0.04
7	D8(Bottom)	10.50	552.59	0.02

8	D4(Bottom)	3.15	318.50	0.01	
9	D9(Bottom)	1.04	106.38	0.01	
10	D11(Max)	86.93	2398.61	0.04	
11	D8(Bottom)	2.50	297.48	0.01	
12	D8(Bottom)	8.50	552.59	0.02	
13	D23(Top)	0.09	451.10	0.00	
14	D5(Bottom)	0.75	150.67	0.00	
15	D23(Bottom)	0.73	3397.24	0.00	
16	D11(Bottom)	2.17	421.33	0.01	
17	D16(Bottom)	1.01	782.66	0.00	

Axial stress

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	Fn [Kip/in2]	Ratio
1	D9(Max)	4.02	0.01	0.30	0.02
2	D11(Max)	86.40	0.02	0.30	0.07
3	D8(Bottom)	10.95	0.01	0.30	0.04
4	D9(Bottom)	3.56	0.01	0.30	0.02
5	D4(Bottom)	1.08	0.01	0.30	0.02
6	D10(Bottom)	85.14	0.02	0.30	0.07
7	D8(Bottom)	10.50	0.01	0.30	0.04
8	D4(Bottom)	3.15	0.01	0.30	0.02
9	D4(Bottom)	1.04	0.01	0.30	0.02
10	D11(Max)	86.93	0.02	0.30	0.07
11	D8(Bottom)	2.50	0.00	0.30	0.02
12	D8(Bottom)	8.50	0.01	0.30	0.03
13	D11(Max)	-0.34	0.00	0.30	0.00
14	D5(Bottom)	0.75	0.00	0.30	0.01
15	D11(Max)	-1.77	0.00	0.30	0.00
16	D5(Bottom)	2.17	0.00	0.30	0.01
17	D16(Bottom)	1.01	0.00	0.30	0.00

Results: Shear

Segment	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D24(Max)	0.03	3.21	0.01
2	D24(Max)	0.04	3.26	0.01
3	D24(Max)	0.05	3.22	0.02
4	D24(Max)	0.07	3.21	0.02
5	D24(Top)	0.07	3.21	0.02
6	D24(Top)	0.09	3.26	0.03
7	D24(Top)	0.08	3.22	0.03
8	D24(Top)	0.13	3.21	0.04
9	D24(Top)	0.14	3.20	0.04
10	D24(Max)	0.13	3.26	0.04
11	D24(Max)	0.16	3.21	0.05
12	D24(Max)	0.13	3.21	0.04
13	D13(Max)	0.21	3.19	0.06
14	D24(Max)	0.22	3.21	0.07
15	D13(Max)	0.22	3.19	0.07
16	D24(Max)	0.21	3.21	0.07
17	D24(Max)	0.20	3.19	0.06

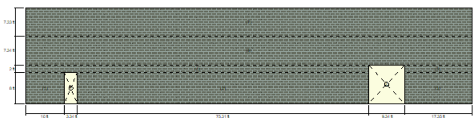
Deflection

Segment	Condition	δs [in]	δmax [in]	δs/δmax
1	S9(Top)	0.01	1.46	0.00
2	S7(Top)	0.01	1.46	0.00
3	S16(Top)	0.01	1.46	0.00
4	S16(Top)	0.01	1.46	0.01
5	S16(Top)	0.01	1.46	0.01

6	S16(Top)	0.01	1.46	0.01	
7	S16(Top)	0.01	1.46	0.01	
8	S7(Top)	0.06	1.46	0.04	
9	S16(Top)	0.07	1.46	0.04	
10	S7(Top)	0.07	1.46	0.04	
11	S7(Top)	0.07	1.46	0.04	
12	S16(Top)	0.07	1.46	0.04	
13	S16(Bottom)	0.01	0.62	0.02	
14	S16(Bottom)	0.01	0.62	0.02	
15	S16(Bottom)	0.01	0.62	0.02	
16	S7(Bottom)	0.01	0.62	0.02	
17	S16(Bottom)	0.01	0.62	0.02	

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	10.00	8.00
2	13.34	0.00	75.31	8.00
3	97.99	0.00	17.35	8.00
4	0.00	8.00	88.65	2.00
5	97.99	8.00	17.35	2.00
6	0.00	10.00	115.34	7.34
7	0.00	17.34	115.34	7.33

Reinforcement:

	Vertical reinforcement			Horizontal reinforcement		
Segment	**Bars**	**Spacing** [in]	**Ld** [in]	**Bars**	**Spacing** [in]	**Ld** [in]
1	5-#4	24.00	0.00	6-W1.7	16.00	7.63
2	38-#4	24.00	0.00	6-W1.7	16.00	7.63
3	9-#4	24.00	0.00	6-W1.7	16.00	7.63
4	5-#4	24.00	0.00	2-W1.7	16.00	7.63
	2-#4	24.00	0.00	2-W1.7	16.00	7.63
	38-#4	24.00	0.00	2-W1.7	16.00	7.63
5	9-#4	24.00	0.00	2-W1.7	16.00	7.63
6	5-#4	24.00	0.00	6-W1.7	16.00	7.63
	2-#4	24.00	0.00	6-W1.7	16.00	7.63
	38-#4	24.00	0.00	6-W1.7	16.00	7.63
	5-#4	24.00	0.00	6-W1.7	16.00	7.63
	9-#4	24.00	0.00	6-W1.7	16.00	7.63
7	5-#4	24.00	0.00	--	0.00	0.00
	2-#4	24.00	0.00	--	0.00	0.00
	38-#4	24.00	0.00	--	0.00	0.00
	5-#4	24.00	0.00	--	0.00	0.00
	9-#4	24.00	0.00	--	0.00	0.00

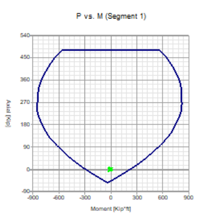
Results: Combined axial flexure

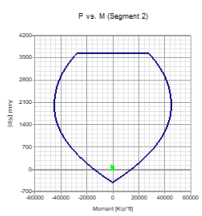
Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D23(Max)	-3.29	-13.49	275.71	0.05
2	D10(Bottom)	86.20	-323.44	16928.53	0.02
3	D23(Bottom)	6.56	-40.84	862.06	0.05
4	D10(Max)	88.43	-786.51	23169.63	0.03
5	D11(Max)	7.27	19.68	806.64	0.02
6	D23(Bottom)	29.86	-254.30	35106.66	0.01
7	D16(Bottom)	2.36	42.50	33609.02	0.00

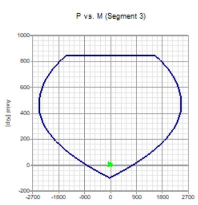
Results: Flexural reinforcement area

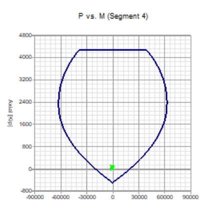
Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D1(Top)	0.00	1.00	14.43	0.07
2	D1(Top)	0.00	7.60	251.49	0.03
3	D25(Bottom)	0.00	1.80	44.07	0.04
4	D26(Bottom)	0.00	9.00	285.81	0.03
5	D23(Bottom)	0.00	1.80	44.07	0.04
6	D26(Bottom)	0.00	11.80	404.84	0.03
7	D26(Bottom)	0.00	11.80	404.84	0.03

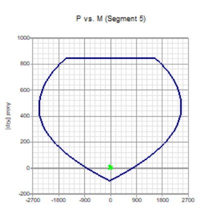
Interaction diagrams, P vs. M:

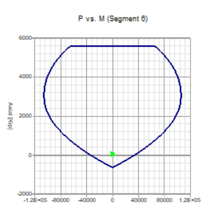


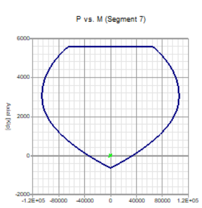












Results: Axial compression

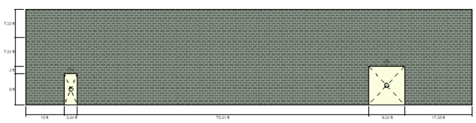
Segment	Condition	Pu [Kip]	ϕPn [Kip]	Ratio
1	D9(Bottom)	3.93	318.50	0.01
2	D10(Max)	86.29	2402.91	0.04
3	D8(Max)	11.02	557.31	0.02
4	D5(Max)	89.57	2832.07	0.03
5	D8(Bottom)	10.50	557.31	0.02
6	D11(Bottom)	97.00	3691.65	0.03
7	D10(Bottom)	2.49	5240.19	0.00

Results: Shear

Segment	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D23(Max)	2.87	43.31	0.07
2	D23(Max)	33.22	411.63	0.08
3	D16(Max)	8.31	87.62	0.09
4	D23(Max)	36.03	478.55	0.08
5	D16(Top)	8.13	95.49	0.09
6	D23(Max)	44.75	644.06	0.07
7	D12(Max)	3.10	581.94	0.01

LINTEL DESIGN:

Status : OK



Geometry:

Lintel	X Coordinate [ft]	Y Coordinate [ft]	Length [ft]	Depth [in]
1	10.00	0.00	3.34	24.00
2	88.65	0.00	9.34	24.00

Reinforcement:

Lintel	Top long. reinforcement Bars	Top long. reinforcement Extent [in]	Bottom long. reinforcement Bars	Bottom long. reinforcement Extent [in]	Transverse reinforcement Bars	Transverse reinforcement Spacing [in]	Ld [in]
1	1-#4	0.50	1-#4	1.50	--	0.00	0.00
2	1-#4	2.50	1-#4	0.00	--	0.00	0.00

Results: Bending

Lintel	Condition	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D5(Top)	2.62	19.12	0.14
2	D16(Bottom)	-1.79	19.12	0.09

Results: Flexural reinforcement area

Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D2(Bottom)	0.00	0.40	2.34	0.17
2		0.00	0.00	0.00	0.00

Results: Cracking moment

Lintel	Condition	1.3 Mcr [Kip*ft]	Mn [Kip*ft]	Ratio
1	D26(Bottom)	6.67	21.25	0.31
2	D26(Bottom)	6.67	21.25	0.31

Results: Shear

Lintel	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D23(Top)	0.84	11.57	0.07
2	D16(Top)	3.41	11.57	0.29

Results: Deflection

Lintel	Condition	δs [in]	δmax [in]	Ratio
1	S3(Top)	0.00	0.07	0.02
2	S6(Bottom)	-0.01	0.19	0.04

Notes:

* Pu = Factored axial load
* Pn = Nominal compression strength
* δ = Moment magnification factor
* Mu = Factored total flexural moment
* Mua = Factored flexural moment from analysis
* Mn = Nominal moment strength
* Mcr = Nominal cracking moment
* ft = Stress due to flexural tension
* fc = Stress due to flexural compression
* Fn = Nominal stress
* Vu = Factored shear force
* Vn = Nominal shear strength
* δs = Calculated deflection
* δmax = Maximum allowable deflection
* ld = Embedment length
* Ag = Gross cross sectional area of a member
* As = Effective cross sectional area of reinforcement
* c = Distance from the fiber of maximum compressive strain to the neutral axis
* d = Distance from the extreme compression fiber to centroid of tension reinforcement

Current Date: 10/24/2022 4:42 PM
Units system: English
File name: G:\2022\22-365 Westlake Ace Hardware - Lee's Summit, MO\Calculations\D - Lateral\South Wall.msw

Design Results

Masonry wall

SOUTH WALL

GENERAL INFORMATION:

Global status : OK

Design code	:	TMS 402-13 SD

Geometry:

Total height	:	24.67 [ft]
Total length	:	112.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Materials:

Material	:	CMU 1.5-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Partial grouting
Mortar bed type	:	Full bed
Masonry compression strength (F`m)	:	1.5 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	24 [Kip/in2]
Joint reinforcement allowable tension strength (Fs)	:	30 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1350 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC B
Response modification factor	:	2.00
Shear wall type	:	Ordinary

Number of stories: **1**

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	17.34	7.63	0.09

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	45.67	0.00	22.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LLR	Roof Live Load
SL	No	SNOW	Snow Load
Wx	No	WIND	Wind in X
Wz	No	WIND	Wind in Z
EQx	No	EQ	Seismic in X
EQz	No	EQ	Seismic in Z
D1	Yes		1.4DL

D2	Yes	1.2DL+0.5LL
D3	Yes	1.2DL+0.5SL
D4	Yes	1.2DL+1.6LL
D5	Yes	1.2DL+1.6SL
D6	Yes	1.2DL+0.5Wx
D7	Yes	1.2DL+0.5Wz
D8	Yes	1.2DL+1.6LL+0.5Wx
D9	Yes	1.2DL+1.6LL+0.5Wz
D10	Yes	1.2DL+1.6SL+0.5Wx
D11	Yes	1.2DL+1.6SL+0.5Wz
D12	Yes	1.2DL+Wx
D13	Yes	1.2DL+Wz
D14	Yes	1.2DL+Wx+0.5LL
D15	Yes	1.2DL+Wz+0.5LL
D16	Yes	1.2DL+Wx+0.5SL
D17	Yes	1.2DL+Wz+0.5SL
D18	Yes	1.2DL+0.2SL
D19	Yes	1.2DL+EQx
D20	Yes	1.2DL+EQz
D21	Yes	1.2DL+EQx+0.2SL
D22	Yes	1.2DL+EQz+0.2SL
D23	Yes	0.9DL+Wx
D24	Yes	0.9DL+Wz
D25	Yes	0.9DL+EQx
D26	Yes	0.9DL+EQz
S1	Yes	DL
S2	Yes	DL+LL
S3	Yes	DL+SL
S4	Yes	DL+0.75LL
S5	Yes	DL+0.75SL
S6	Yes	DL+0.6Wx
S7	Yes	DL+0.6Wz
S8	Yes	DL+0.7EQx
S9	Yes	DL+0.7EQz
S10	Yes	DL+0.525EQx
S11	Yes	DL+0.525EQz
S12	Yes	DL+0.75SL
S13	Yes	DL+0.525EQx+0.75SL
S14	Yes	DL+0.525EQz+0.75SL
S15	Yes	0.6DL+0.6Wx
S16	Yes	0.6DL+0.6Wz
S17	Yes	0.6DL+0.7EQx
S18	Yes	0.6DL+0.7EQz

Concentrated loads:

Story	Condition	Direction	Magnitude [Kip]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	6.50	0.00	28.00
1	DL	Vertical	6.50	0.00	56.00
1	DL	Vertical	6.50	0.00	84.00
1	LL	Vertical	5.19	0.00	28.00
1	LL	Vertical	5.19	0.00	56.00
1	LL	Vertical	5.19	0.00	84.00
1	DL	Vertical	1.13	0.00	34.17
1	LL	Vertical	1.50	0.00	34.17
1	DL	Vertical	1.13	0.00	56.00
1	LL	Vertical	1.50	0.00	56.00
1	DL	Vertical	1.13	0.00	77.83
1	LL	Vertical	1.50	0.00	77.83
1	SL	Vertical	6.06	0.00	28.00
1	SL	Vertical	6.06	0.00	56.00
1	SL	Vertical	6.06	0.00	84.00
1	SL	Vertical	2.50	0.00	34.17
1	SL	Vertical	2.50	0.00	56.00
1	SL	Vertical	2.50	0.00	77.83

Distributed loads:

Consider self weight : No

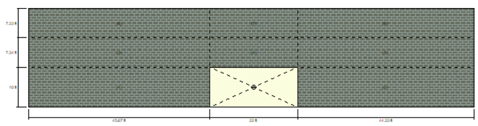
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	DL	Vertical	0.09	0.00
1	LL	Vertical	0.12	0.00
1	SL	Vertical	0.08	0.00
1	Wx	Horizontal	0.37	0.00
1	EQx	Horizontal	0.11	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wz	0.01
1	EQz	0.00
Parapet	Wz	0.04
Parapet	EQz	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	45.67	10.00
2	67.67	0.00	44.33	10.00
3	0.00	10.00	45.67	7.34
4	45.67	10.00	22.00	7.34
5	67.67	10.00	44.33	7.34
6	0.00	17.34	45.67	7.33
7	45.67	17.34	22.00	7.33
8	67.67	17.34	44.33	7.33

Vertical reinforcement:

Segment	Bars	Spacing [in]	Ld [in]
1	23-#4	24.00	25.17
2	22-#4	24.00	25.17
3	23-#4	24.00	25.17
4	11-#4	24.00	25.17
5	22-#4	24.00	25.17
6	23-#4	24.00	25.17
7	11-#4	24.00	25.17
8	22-#4	24.00	25.17

Results: Combined axial flexure

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D24(Top)	13.46	9.66	9.68	77.91	0.12
2	D24(Top)	13.03	9.45	9.47	75.62	0.13
3	D24(Top)	12.09	46.21	46.47	77.52	0.60
4	D24(Top)	5.27	19.71	19.76	37.19	0.53
5	D24(Top)	12.00	44.37	44.61	75.33	0.59
6	D24(Bottom)	1.59	46.77	46.78	74.55	0.63
7	D17(Bottom)	-3.20	19.70	19.69	34.79	0.57
8	D24(Bottom)	1.52	45.42	45.43	72.36	0.63

Results: Flexural reinforcement area

Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D26(Max)	0.00	4.57	12.47	0.37
2	D26(Max)	0.00	4.43	12.10	0.37
3	D26(Bottom)	0.00	4.57	12.47	0.37
4	D26(Bottom)	0.00	2.20	6.01	0.37
5	D26(Bottom)	0.00	4.43	12.10	0.37
6	D26(Bottom)	0.00	4.57	12.47	0.37
7	D26(Bottom)	0.00	2.20	6.01	0.37
8	D26(Bottom)	0.00	4.43	12.10	0.37

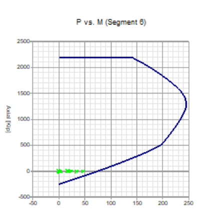
Intermediate results for axial-bending

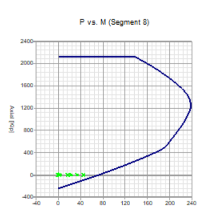
Segment	Condition	c [in]	d [in]	Mcr [Kip*ft]
1	D24(Top)	0.55	3.81	0.93
2	D24(Top)	0.55	3.81	0.93
3	D24(Top)	0.55	3.81	0.92
4	D24(Top)	0.54	3.81	0.90
5	D24(Top)	0.55	3.81	0.92
6	D24(Bottom)	0.52	3.81	0.89
7	D17(Bottom)	0.51	3.81	0.88
8	D24(Bottom)	0.52	3.81	0.89

Inertias

Segment	Condition	Ig [in4]	Icr [in4]
1	D24(Top)	365.44	25.36
2	D24(Top)	365.44	25.36
3	D24(Top)	365.44	25.27
4	D24(Top)	365.44	25.19
5	D24(Top)	365.44	25.29
6	D24(Bottom)	365.44	24.55
7	D17(Bottom)	365.44	23.98
8	D24(Bottom)	365.44	24.55

Interaction diagrams, P vs. M:





Results: Axial compression

Segment	Condition	Pu [Kip]	ϕPn [Kip]	Ratio
1	D5(Bottom)	50.52	1454.48	0.03
2	D10(Bottom)	50.00	1412.00	0.04
3	D5(Bottom)	43.28	1454.48	0.03
4	D10(Top)	17.29	700.69	0.02
5	D10(Bottom)	42.25	1412.00	0.03
6	D10(Bottom)	5.62	2060.04	0.00
7	D5(Top)	0.48	992.42	0.00
8	D11(Bottom)	4.90	1999.87	0.00

Axial stress

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	Fn [Kip/in2]	Ratio
1	D5(Bottom)	50.52	0.02	0.30	0.07
2	D10(Bottom)	50.00	0.02	0.30	0.07
3	D11(Bottom)	43.28	0.02	0.30	0.06
4	D10(Top)	17.29	0.01	0.30	0.05
5	D10(Bottom)	42.25	0.02	0.30	0.06
6	D10(Bottom)	5.62	0.00	0.30	0.01
7	D5(Bottom)	-5.38	0.00	0.30	0.01
8	D5(Bottom)	4.90	0.00	0.30	0.01

Results: Shear

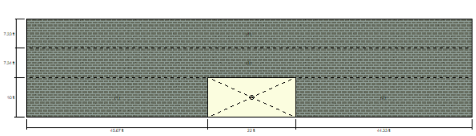
Segment	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D24(Top)	0.09	3.25	0.03
2	D24(Top)	0.08	3.25	0.03
3	D24(Top)	0.15	3.24	0.04
4	D24(Max)	0.12	3.23	0.04
5	D24(Top)	0.13	3.25	0.04
6	D24(Bottom)	0.23	3.20	0.07
7	D13(Max)	0.19	3.19	0.06
8	D24(Bottom)	0.23	3.20	0.07

Deflection

Segment	Condition	δs [in]	δmax [in]	δs/δmax
1	S16(Top)	0.01	1.46	0.01
2	S16(Top)	0.01	1.46	0.01
3	S7(Top)	0.07	1.46	0.05
4	S16(Top)	0.06	1.46	0.04
5	S7(Top)	0.07	1.46	0.05
6	S7(Bottom)	0.01	0.62	0.02
7	S7(Bottom)	0.01	0.62	0.02
8	S16(Bottom)	0.01	0.62	0.02

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	45.67	10.00
2	67.67	0.00	44.33	10.00
3	0.00	10.00	112.00	7.34
4	0.00	17.34	112.00	7.33

Reinforcement:

Segment	Vertical reinforcement			Horizontal reinforcement		
	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	23-#4	24.00	0.00	8-W1.7	16.00	7.63
2	22-#4	24.00	0.00	8-W1.7	16.00	7.63
3	23-#4	24.00	0.00	6-W1.7	16.00	7.63
	11-#4	24.00	0.00	6-W1.7	16.00	7.63
	22-#4	24.00	0.00	6-W1.7	16.00	7.63
4	23-#4	24.00	0.00	--	0.00	0.00
	11-#4	24.00	0.00	--	0.00	0.00
	22-#4	24.00	0.00	--	0.00	0.00

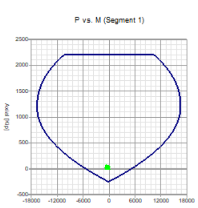
Results: Combined axial flexure

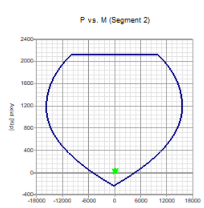
Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D10(Max)	49.17	-569.85	6252.27	0.09
2	D11(Max)	48.78	436.93	5561.71	0.08
3	D23(Bottom)	29.86	-269.27	32865.79	0.01
4	D23(Bottom)	1.59	45.22	30600.78	0.00

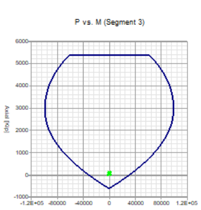
Results: Flexural reinforcement area

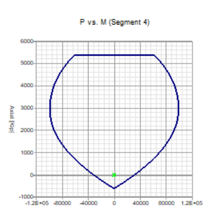
Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D1(Top)	0.00	4.60	134.08	0.03
2	D23(Bottom)	0.00	4.40	114.41	0.04
3	D1(Top)	0.00	11.20	351.73	0.03
4	D1(Top)	0.00	11.20	351.73	0.03

Interaction diagrams, P vs. M:









Results: Axial compression

Segment	Condition	Pu [Kip]	ϕPn [Kip]	Ratio
1	D5(Max)	50.76	1456.73	0.03
2	D10(Bottom)	50.00	1409.74	0.04
3	D10(Max)	96.72	3567.17	0.03
4	D10(Bottom)	4.70	5052.33	0.00

Results: Shear

Segment	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D23(Max)	19.31	215.87	0.09
2	D16(Max)	25.88	242.54	0.11
3	D23(Bottom)	43.27	618.99	0.07
4	D23(Max)	2.92	545.20	0.01

Notes:

* Pu = Factored axial load
* Pn = Nominal compression strength
* δ = Moment magnification factor
* Mu = Factored total flexural moment
* Mua = Factored flexural moment from analysis
* Mn = Nominal moment strength
* Mcr = Nominal cracking moment
* ft = Stress due to flexural tension
* fc = Stress due to flexural compression
* Fn = Nominal stress
* Vu = Factored shear force
* Vn = Nominal shear strength
* δs = Calculated deflection
* δmax = Maximum allowable deflection
* ld = Embedment length
* Ag = Gross cross sectional area of a member
* As = Effective cross sectional area of reinforcement
* c = Distance from the fiber of maximum compressive strain to the neutral axis
* d = Distance from the extreme compression fiber to centroid of tension reinforcement

Bentley

Current Date: 11/7/2022 1:59 PM
Units system: English
File name: G:\2022\22-365 Westlake Ace Hardware - Lee's Summit, MO\Calculations\D - Lateral\West Wall.msw

Design Results

Masonry wall

WEST WALL

GENERAL INFORMATION:

**Global status : ** OK

Design code	:	TMS 402-13 SD

Geometry:

Total height	:	24.67 [ft]
Total length	:	136.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Materials:

Material	:	CMU 1.5-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Partial grouting
Mortar bed type	:	Full bed
Masonry compression strength (F`m)	:	1.5 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	24 [Kip/in2]
Joint reinforcement allowable tension strength (Fs)	:	30 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1350 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Number of stories: 1

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	16.00	7.63	0.09

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	2.17	0.00	3.33	7.33
Lower left	28.83	0.00	3.33	7.33

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
LL	No	LLR	Roof Live Load
SL	No	SNOW	Snow Load
Wx	No	WIND	Wind in X
Wz	No	WIND	Wind in Z
EQx	No	EQ	Seismic in X
EQz	No	EQ	Seismic in Z
D1	Yes		1.4DL
D2	Yes		1.2DL+0.5LL
D3	Yes		1.2DL+0.5SL
D4	Yes		1.2DL+1.6LL
D5	Yes		1.2DL+1.6SL

D6	Yes	1.2DL+0.5Wx
D7	Yes	1.2DL+0.5Wz
D8	Yes	1.2DL+1.6LL+0.5Wx
D9	Yes	1.2DL+1.6LL+0.5Wz
D10	Yes	1.2DL+1.6SL+0.5Wx
D11	Yes	1.2DL+1.6SL+0.5Wz
D12	Yes	1.2DL+Wx
D13	Yes	1.2DL+Wz
D14	Yes	1.2DL+0.5LL+Wx
D15	Yes	1.2DL+0.5LL+Wz
D16	Yes	1.2DL+0.5SL+Wx
D17	Yes	1.2DL+0.5SL+Wz
D18	Yes	1.2DL+0.2SL
D19	Yes	1.2DL+EQx
D20	Yes	1.2DL+EQz
D21	Yes	1.2DL+0.2SL+EQx
D22	Yes	1.2DL+0.2SL+EQz
D23	Yes	0.9DL+Wx
D24	Yes	0.9DL+Wz
D25	Yes	0.9DL+EQx
D26	Yes	0.9DL+EQz
S1	Yes	DL
S2	Yes	DL+LL
S3	Yes	DL+SL
S4	Yes	DL+0.75LL
S5	Yes	DL+0.75SL
S6	Yes	DL+0.6Wx
S7	Yes	DL+0.6Wz
S8	Yes	DL+0.7EQx
S9	Yes	DL+0.7EQz
S10	Yes	DL+0.525EQx
S11	Yes	DL+0.525EQz
S12	Yes	DL+0.75SL
S13	Yes	DL+0.75SL+0.525EQx
S14	Yes	DL+0.75SL+0.525EQz
S15	Yes	0.6DL+0.6Wx
S16	Yes	0.6DL+0.6Wz
S17	Yes	0.6DL+0.7EQx
S18	Yes	0.6DL+0.7EQz

Distributed loads:

Consider self weight : No

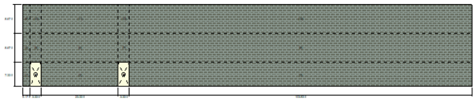
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	DL	Vertical	0.22	0.00
1	SL	Vertical	0.43	0.00
1	LL	Vertical	0.30	0.00
1	Wx	Horizontal	0.25	0.00
1	EQx	Horizontal	0.08	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wz	0.01
1	EQz	0.00
Parapet	Wz	0.04
Parapet	EQz	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.17	7.33
2	5.50	0.00	23.33	7.33
3	32.17	0.00	103.83	7.33
4	0.00	7.33	2.17	8.67
5	2.17	7.33	3.33	8.67
6	5.50	7.33	23.33	8.67
7	28.83	7.33	3.33	8.67
8	32.17	7.33	103.83	8.67
9	0.00	16.00	2.17	8.67
10	2.17	16.00	3.33	8.67
11	5.50	16.00	23.33	8.67
12	28.83	16.00	3.33	8.67
13	32.17	16.00	103.83	8.67

Vertical reinforcement:

Segment	Bars	Spacing [in]	Ld [in]
1	1-#4	24.00	25.17
2	12-#4	24.00	25.17
3	52-#4	24.00	25.17
4	1-#4	24.00	25.17
5	2-#4	24.00	25.17
6	12-#4	24.00	25.17
7	2-#4	24.00	25.17
8	52-#4	24.00	25.17
9	1-#4	24.00	25.17
10	2-#4	24.00	25.17
11	12-#4	24.00	25.17
12	2-#4	24.00	25.17
13	52-#4	24.00	25.17

Results: Combined axial flexure

Segment	Condition	Pu [Kip]	Mua [Kip*ft]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D24(Top)	0.58	0.86	0.86	3.68	0.23
2	D24(Top)	5.29	7.76	7.78	39.36	0.20
3	D24(Top)	20.99	32.99	33.04	174.42	0.19
4	D24(Top)	0.38	2.65	2.67	3.62	0.74
5	D24(Top)	0.62	4.55	4.59	5.58	0.82
6	D24(Top)	4.87	32.02	32.31	39.24	0.82
7	D24(Top)	0.62	4.73	4.77	5.58	0.85
8	D24(Top)	20.83	142.40	143.67	174.38	0.82
9	D17(Bottom)	-0.06	2.80	2.80	3.50	0.80
10	D17(Bottom)	-0.05	4.43	4.43	5.39	0.82
11	D24(Bottom)	0.07	32.20	32.21	37.88	0.85
12	D17(Bottom)	-0.02	4.52	4.52	5.40	0.84
13	D17(Bottom)	-0.04	141.70	141.70	168.47	0.84

Results: Flexural reinforcement area

Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D26(Bottom)	0.00	0.22	0.54	0.40
2	D26(Max)	0.00	2.33	6.37	0.37
3	D26(Max)	0.00	10.38	28.35	0.37
4	D26(Bottom)	0.00	0.22	0.54	0.40
5	D26(Bottom)	0.00	0.33	0.91	0.37
6	D26(Bottom)	0.00	2.33	6.37	0.37
7	D26(Bottom)	0.00	0.33	0.91	0.37
8	D26(Bottom)	0.00	10.38	28.35	0.37
9	D26(Bottom)	0.00	0.22	0.54	0.40
10	D26(Bottom)	0.00	0.33	0.91	0.37
11	D26(Bottom)	0.00	2.33	6.37	0.37
12	D26(Bottom)	0.00	0.33	0.91	0.37
13	D26(Bottom)	0.00	10.38	28.35	0.37

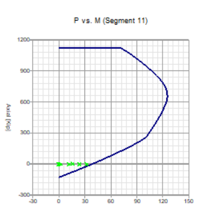
Intermediate results for axial-bending

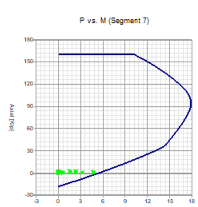
Segment	Condition	c [in]	d [in]	Mcr [Kip*ft]
1	D24(Top)	0.55	3.81	0.92
2	D24(Top)	0.54	3.81	0.92
3	D24(Top)	0.54	3.81	0.91
4	D24(Top)	0.54	3.81	0.92
5	D24(Top)	0.54	3.81	0.90
6	D24(Top)	0.54	3.81	0.91
7	D24(Top)	0.54	3.81	0.90
8	D24(Top)	0.54	3.81	0.91
9	D17(Bottom)	0.52	3.81	0.88
10	D17(Bottom)	0.52	3.81	0.88
11	D24(Bottom)	0.52	3.81	0.88
12	D17(Bottom)	0.52	3.81	0.88
13	D17(Bottom)	0.52	3.81	0.88

Inertias

Segment	Condition	Ig [in4]	Icr [in4]
1	D24(Top)	365.44	25.27
2	D24(Top)	365.44	25.15
3	D24(Top)	365.44	25.07
4	D24(Top)	365.44	24.99
5	D24(Top)	365.44	25.02
6	D24(Top)	365.44	25.09
7	D24(Top)	365.44	25.02
8	D24(Top)	365.44	25.07
9	D17(Bottom)	365.44	24.36
10	D17(Bottom)	365.44	24.39
11	D24(Bottom)	365.44	24.45
12	D17(Bottom)	365.44	24.42
13	D17(Bottom)	365.44	24.44

Interaction diagrams, P vs. M:





Results: Axial compression

Segment	Condition	Pu [Kip]	ϕPn [Kip]	Ratio
1	D11(Max)	3.26	50.22	0.06
2	D11(Bottom)	26.91	799.15	0.03
3	D10(Bottom)	101.91	3556.17	0.03
4	D11(Bottom)	2.75	50.22	0.05
5	D10(Top)	2.95	114.15	0.03
6	D11(Bottom)	25.14	799.15	0.03
7	D10(Top)	2.97	114.15	0.03
8	D10(Bottom)	101.20	3556.17	0.03
9	D23(Bottom)	0.10	64.46	0.00
10	D23(Top)	0.06	146.51	0.00
11	D11(Bottom)	0.33	1025.72	0.00
12	D23(Top)	0.00	146.51	0.00
13	D23(Bottom)	0.22	4564.41	0.00

Axial stress

Segment	Condition	Pu [Kip]	Pu/Ag [Kip/in2]	Fn [Kip/in2]	Ratio
1	D11(Max)	3.26	0.03	0.30	0.09
2	D5(Bottom)	26.91	0.02	0.30	0.07
3	D10(Bottom)	101.91	0.02	0.30	0.06
4	D5(Bottom)	2.75	0.02	0.30	0.08
5	D10(Top)	2.95	0.02	0.30	0.05
6	D11(Bottom)	25.14	0.02	0.30	0.06
7	D10(Top)	2.97	0.02	0.30	0.05
8	D10(Bottom)	101.20	0.02	0.30	0.06
9	D11(Bottom)	-0.11	0.00	0.30	0.00
10	D10(Bottom)	-0.12	0.00	0.30	0.00
11	D23(Bottom)	-0.34	0.00	0.30	0.00
12	D10(Bottom)	-0.04	0.00	0.30	0.00
13	D23(Bottom)	0.22	0.00	0.30	0.00

Results: Shear

Segment	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D24(Bottom)	0.06	3.25	0.02
2	D24(Top)	0.07	3.24	0.02
3	D24(Top)	0.05	3.23	0.02
4	D24(Top)	0.13	3.23	0.04
5	D24(Max)	0.16	3.23	0.05
6	D24(Top)	0.15	3.23	0.05
7	D24(Max)	0.17	3.23	0.05
8	D24(Top)	0.13	3.23	0.04
9	D24(Bottom)	0.21	3.19	0.07
10	D13(Max)	0.22	3.19	0.07
11	D24(Max)	0.24	3.19	0.08
12	D13(Max)	0.23	3.19	0.07
13	D13(Max)	0.24	3.19	0.07

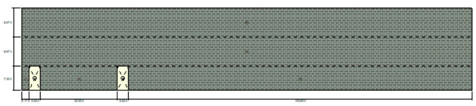
Deflection

Segment	Condition	δs [in]	δmax [in]	δs/δmax
1	S16(Top)	0.02	1.34	0.02
2	S16(Top)	0.02	1.34	0.01
3	S16(Top)	0.02	1.34	0.01
4	S7(Top)	0.07	1.34	0.05
5	S7(Top)	0.08	1.34	0.06

6	S7(Top)	0.08	1.34	0.06
7	S7(Top)	0.08	1.34	0.06
8	S7(Top)	0.08	1.34	0.06
9	S16(Bottom)	0.02	0.73	0.03
10	S7(Bottom)	0.02	0.73	0.03
11	S16(Bottom)	0.02	0.73	0.03
12	S16(Bottom)	0.02	0.73	0.03
13	S16(Bottom)	0.02	0.73	0.03

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.17	7.33
2	5.50	0.00	23.33	7.33
3	32.17	0.00	103.83	7.33
4	0.00	7.33	136.00	8.67
5	0.00	16.00	136.00	8.67

Reinforcement:

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	1-#4	24.00	0.00	--	0.00	0.00
2	12-#4	24.00	0.00	--	0.00	0.00
3	52-#4	24.00	0.00	--	0.00	0.00
4	1-#4	24.00	0.00	--	0.00	0.00
	2-#4	24.00	0.00	--	0.00	0.00
	12-#4	24.00	0.00	--	0.00	0.00
	2-#4	24.00	0.00	--	0.00	0.00
	52-#4	24.00	0.00	--	0.00	0.00
5	1-#4	24.00	0.00	--	0.00	0.00
	2-#4	24.00	0.00	--	0.00	0.00
	12-#4	24.00	0.00	--	0.00	0.00
	2-#4	24.00	0.00	--	0.00	0.00
	52-#4	24.00	0.00	--	0.00	0.00

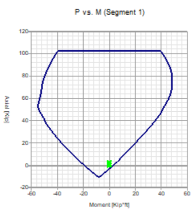
Results: Combined axial flexure

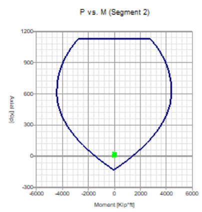
Segment	Condition	Pu [Kip]	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D11(Bottom)	3.11	0.09	5.99	0.01
2	D23(Bottom)	4.15	-29.36	1476.41	0.02
3	D23(Bottom)	24.00	-154.88	28041.11	0.01
4	D23(Bottom)	28.11	-319.82	48820.02	0.01
5	D23(Bottom)	-0.10	-12.93	47245.74	0.00

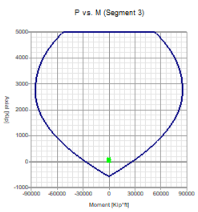
Results: Flexural reinforcement area

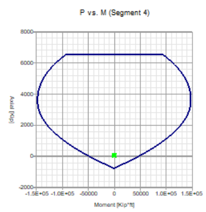
Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D6(Bottom)	0.00	0.20	1.08	0.19
2	D1(Top)	0.00	2.40	76.88	0.00
3	D1(Top)	0.00	10.40	373.38	0.00
4	D26(Max)	0.00	13.80	425.39	0.03
5	D26(Top)	0.00	13.80	425.39	0.03

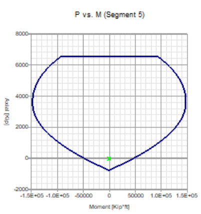
Interaction diagrams, P vs. M:











Results: Axial compression

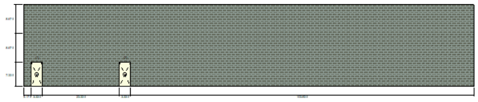
Segment	Condition	Pu [Kip]	ϕPn [Kip]	Ratio
1	D5(Max)	3.13	73.44	0.04
2	D11(Bottom)	26.91	804.51	0.03
3	D10(Bottom)	101.91	3556.93	0.03
4	D10(Bottom)	132.81	4673.92	0.03
5	D23(Top)	0.04	6004.92	0.00

Results: Shear

Segment	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D23(Max)	0.39	8.27	0.05
2	D23(Max)	6.13	110.14	0.06
3	D16(Max)	27.81	532.51	0.05
4	D23(Max)	34.47	696.44	0.05
5	D23(Max)	2.74	698.98	0.00

LINTEL DESIGN:

Status : OK



Geometry:

Lintel	X Coordinate [ft]	Y Coordinate [ft]	Length [ft]	Depth [in]
1	2.17	0.00	3.33	24.00
2	28.83	0.00	3.33	24.00

Reinforcement:

	Top long. reinforcement		Bottom long. reinforcement		Transverse reinforcement		
Lintel	Bars	Extent [in]	Bars	Extent [in]	Bars	Spacing [in]	Ld [in]
1	1-#4	1.50	1-#4	0.50	--	0.00	0.00
2	1-#4	0.50	1-#4	0.50	--	0.00	0.00

Results: Bending

Lintel	Condition	Mu [Kip*ft]	ϕMn [Kip*ft]	Ratio
1	D12(Top)	0.91	19.12	0.05
2	D23(Bottom)	-0.28	19.12	0.01

Results: Flexural reinforcement area

Segment	Condition	Pu [Kip]	As [in2]	Asmax [in2]	Ratio
1	D12(Top)	0.00	0.40	2.34	0.17
2		0.00	0.00	0.00	0.00

Results: Cracking moment

Lintel	Condition	1.3 Mcr [Kip*ft]	Mn [Kip*ft]	Ratio
1	D26(Bottom)	6.67	21.25	0.31
2	D26(Bottom)	6.67	21.25	0.31

Results: Shear

Lintel	Condition	Vu [Kip]	ϕVn [Kip]	Ratio
1	D10(Top)	0.63	11.57	0.05
2	D10(Top)	0.70	11.57	0.06

Results: Deflection

Lintel	Condition	δs [in]	δmax [in]	Ratio
1	S15(Top)	0.00	0.07	0.00
2	S15(Bottom)	0.00	0.07	0.00

Notes:

* Pu = Factored axial load
* Pn = Nominal compression strength
* δ = Moment magnification factor
* Mu = Factored total flexural moment
* Mua = Factored flexural moment from analysis
* Mn = Nominal moment strength
* Mcr = Nominal cracking moment
* ft = Stress due to flexural tension
* fc = Stress due to flexural compression
* Fn = Nominal stress
* Vu = Factored shear force
* Vn = Nominal shear strength
* δs = Calculated deflection
* δmax = Maximum allowable deflection
* ld = Embedment length
* Ag = Gross cross sectional area of a member
* As = Effective cross sectional area of reinforcement
* c = Distance from the fiber of maximum compressive strain to the neutral axis
* d = Distance from the extreme compression fiber to centroid of tension reinforcement