STRUCTURAL ENGINEERS

Project Paragon Star - Lot 20 Project No. 21-036

Calc. By DN Checked By TJ Date 8/6/2021

STRUCTURAL CALCULATIONS FOR :

Paragon Star - Lot 20
HUB BUILDING
Lee's Summit, MISSOURI

STATE OF MISSOURI
STEVEN N. BUSEY
NUMBER
E-25462
PROFESSIONAL ENGINEER
08-06-21

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**	**γ**	**Weight**
	(in)	**(lb/ft^3)**	**(lb/ft^2)**
Roofing Material	0.000		2.0
Polystyrene	3.000	15	3.7
1.5" Type B Metal Roof Deck	1.500		2.5
Joists	0.000		5.2
MEP	0.000		3.0
Collateral/Misc.	0.000		3.5
Totals	**4.500**		**20.0**

LIVE LOADING

LIVE LOAD CONSTRUCTION

Roof Live Load	**20 psf**
Floor Live Public	**100 psf**
Floor Live Office	**80 psf**
IBC 2012, Table 1607.1	

ATC Hazards by Location

Search Information

Coordinates: 38.93815140446288, -94.44568710947266

Elevation: 814 ft

Timestamp: 2021-06-22T15:07:53.548Z

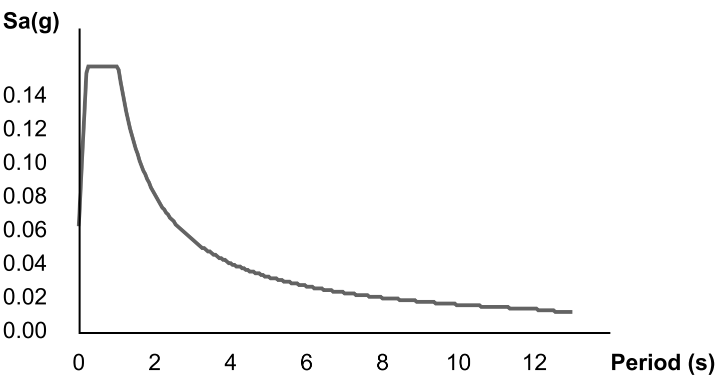
Hazard Type: Seismic

Reference Document: ASCE7-16

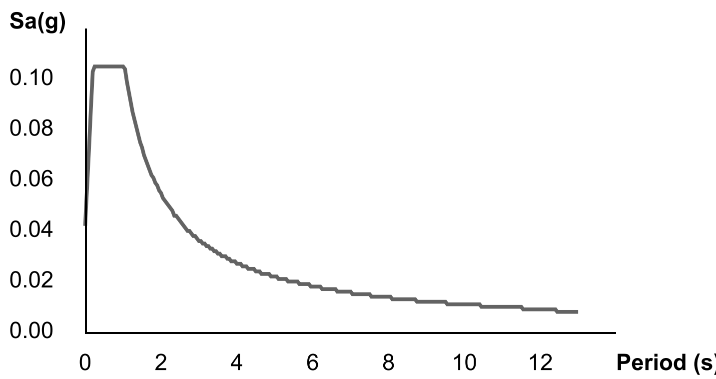
Risk Category: II

Site Class: D

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.099	MCE_R ground motion (period=0.2s)
S_1	0.068	MCE_R ground motion (period=1.0s)
S_{MS}	0.158	Site-modified spectral acceleration value
S_{M1}	0.164	Site-modified spectral acceleration value
S_{DS}	0.105	Numeric seismic design value at 0.2s SA
S_{D1}	0.109	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	B	Seismic design category
F_a	1.6	Site amplification factor at 0.2s
F_v	2.4	Site amplification factor at 1.0s
CR_S	0.928	Coefficient of risk (0.2s)

CR_1	0.877	Coefficient of risk (1.0s)
PGA	0.047	MCE_G peak ground acceleration
F_{PGA}	1.6	Site amplification factor at PGA
PGA_M	0.075	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	0.099	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.106	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.068	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.078	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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SEISMIC FORCES (ASCE 7-16)

Tedds calculation version 3.1.00

Site parameters

Site class D

Mapped acceleration parameters (Section 11.4.2)

at short period S_S = **0.099**

at 1 sec period S_1 = **0.068**

Site coefficientat short period (Table 11.4-1) F_a = **1.600**

at 1 sec period (Table 11.4-2) F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S$ = **0.158**

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

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS}$ = **0.106**

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

Seismic design category

Occupancy category (Table 1-1) II

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

A

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

B

Seismic design category B

Approximate fundamental period

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

From Table 12.8-2:

Structure type All other systems

Building period parameter C_t C_t = **0.02**

Building period parameter x x = **0.75**

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$ = **0.297** sec

Building fundamental period (Sect 12.8.2) $T = T_a$ = **0.297** sec

Long-period transition period T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1) A. Bearing_Wall_Systems

17. Light-framed walls with shear panels of all other materials

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) $C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0528**

Maximum (Eq 12.8-3) $C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e))$ = **0.1829**

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 = **0.0528**

ATC Hazards by Location

Search Information

Coordinates: 38.93815140446288, -94.44568710947266

Elevation: 814 ft

Timestamp: 2021-06-22T15:05:18.183Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	76 mph
MRI 25-Year	83 mph
MRI 50-Year	88 mph
MRI 100-Year	94 mph
Risk Category I	103 mph
Risk Category II	109 mph
Risk Category III	117 mph
Risk Category IV	122 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

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Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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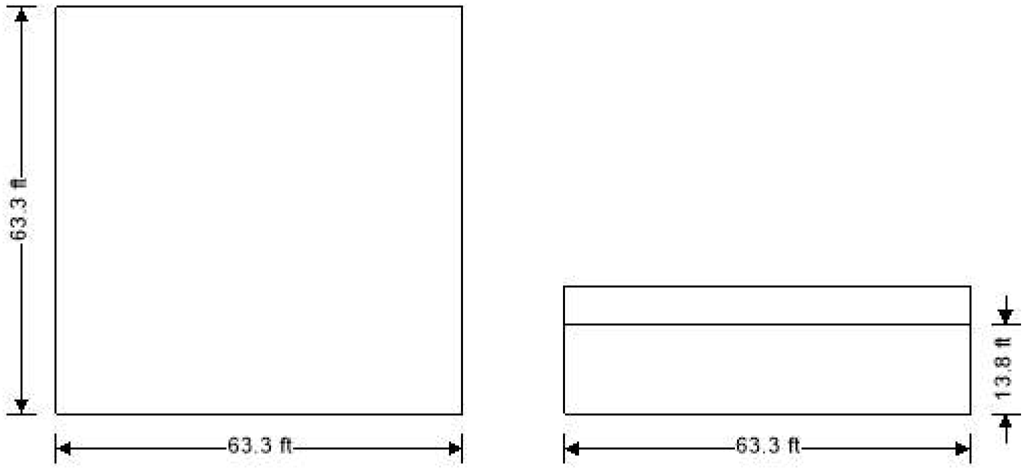
WIND LOADING

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.03



Plan   Elevation

Building data

Type of roof	Flat
Length of building	b = **63.33** ft
Width of building	d = **63.33** ft
Height to eaves	H = **13.83** ft
Height of parapet	h_p = **6.00** ft
Mean height	h = **13.83** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **5.53** ft
Plan length of Zone 2/2E when GC_{pf} negative	L_{Z2} = min(0.5 × d, 2.5 × H) = **31.67** ft
Plan length of Zone 3/3E encroachment on zone 2	L_{Z3} = max(0 ft,0.5 × d - L_{Z2}) = **0.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **814** ft
Ground elevation factor	K_e = exp(-0.0000362 × z_{gl}/1ft) = **0.97**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **21.3** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.90**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1psf/mph^2$ = **22.5** psf

Parapet pressures and forces

Velocity pressure at top of parapet q_p = **22.55** psf

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

Combined net parapet pressure, leeward $p_{pl} = q_p \times GC_{pnl}$ = **-22.55** psf

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

Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw}$ = **33.82** psf

Wind direction 0 deg (|| to width):

Leeward parapet force $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-8.6** kips

Windward parapet force $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **12.9** kips

Wind direction 90 deg (|| to length):

Leeward parapet force $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-8.6** kips

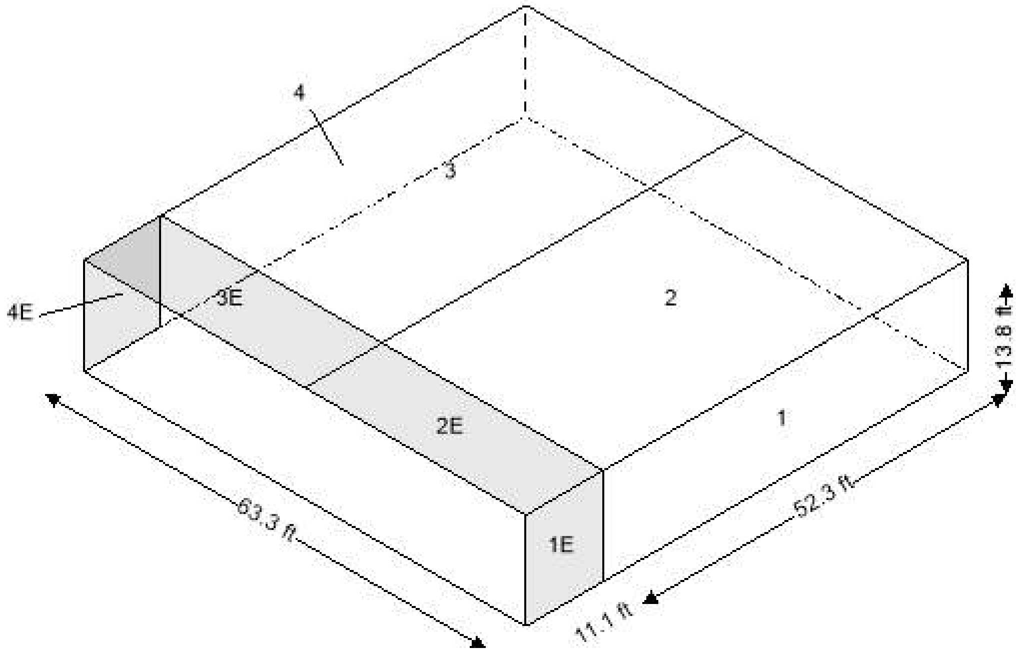
Windward parapet force $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **12.9** kips

Design wind pressures

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

Design wind pressures – Loadcase A

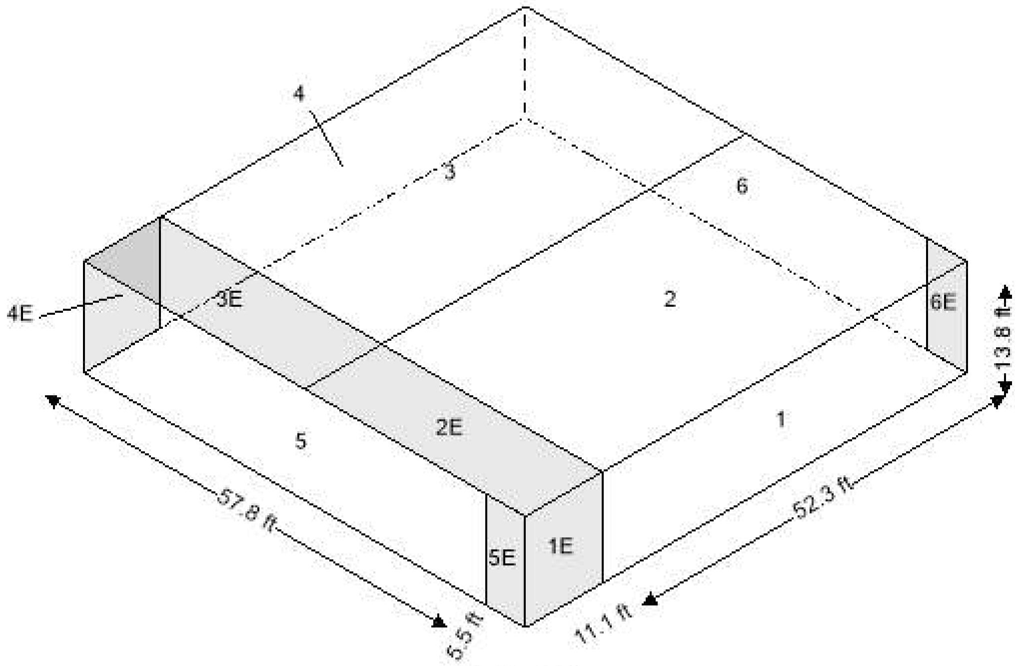
Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.40	4.7	12.4	723	3.4	8.9
2	-0.69	-18.6	-10.9	1655	-30.7	-18.0
3	-0.37	-11.7	-4.1	1655	-19.4	-6.7
4	-0.29	-10.0	-2.3	723	-7.2	-1.7
1E	0.61	9.2	16.9	153	1.4	2.6
2E	-1.07	-26.7	-19.0	350	-9.3	-6.7
3E	-0.53	-15.1	-7.5	350	-5.3	-2.6
4E	-0.43	-13.0	-5.3	153	-2.0	-0.8



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	-13.4	-5.8	723	-9.7	-4.2
2	-0.69	-18.6	-10.9	1655	-30.7	-18.0
3	-0.37	-11.7	-4.1	1655	-19.4	-6.7
4	-0.45	-13.4	-5.8	723	-9.7	-4.2
5	0.40	4.7	12.4	799	3.8	9.9
6	-0.29	-10.0	-2.3	799	-8.0	-1.9
1E	-0.48	-14.1	-6.4	153	-2.2	-1.0
2E	-1.07	-26.7	-19.0	350	-9.3	-6.7
3E	-0.53	-15.1	-7.5	350	-5.3	-2.6
4E	-0.48	-14.1	-6.4	153	-2.2	-1.0
5E	0.61	9.2	16.9	77	0.7	1.3
6E	-0.43	-13.0	-5.3	77	-1.0	-0.4

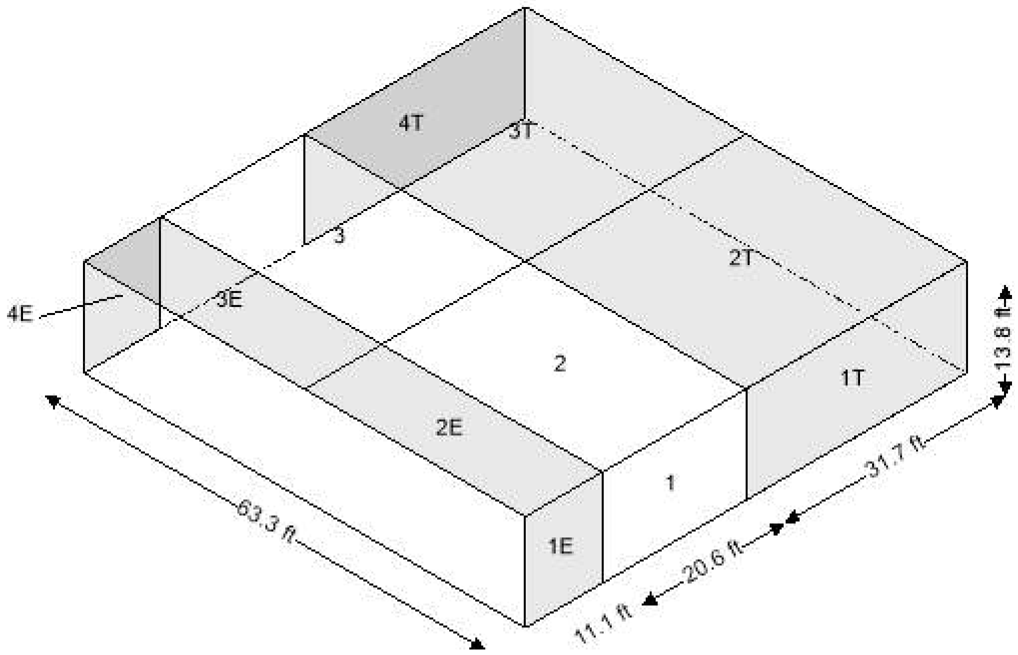


Loadcase B

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	4.7	12.4	285	1.3	3.5
2	-0.69	-18.6	-10.9	652	-12.1	-7.1
3	-0.37	-11.7	-4.1	652	-7.7	-2.6
4	-0.29	-10.0	-2.3	285	-2.9	-0.7
1E	0.61	9.2	16.9	153	1.4	2.6
2E	-1.07	-26.7	-19.0	350	-9.3	-6.7

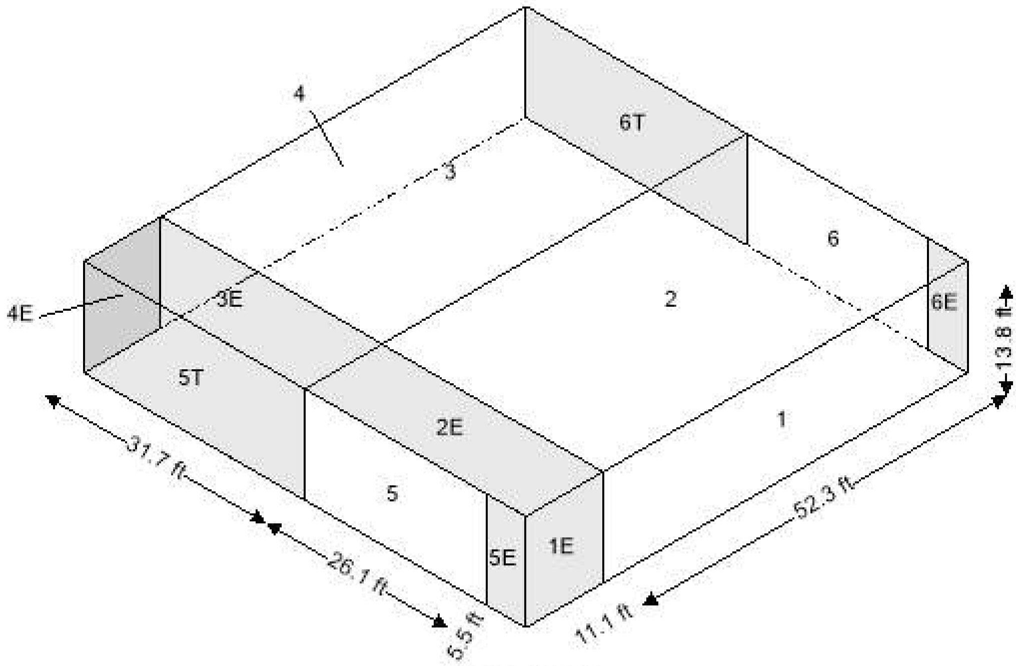
3E	-0.53	-15.1	-7.5	350	-5.3	-2.6
4E	-0.43	-13.0	-5.3	153	-2.0	-0.8
1T	-	1.2	3.1	438	0.5	1.4
2T	-	-4.6	-2.7	1003	-4.7	-2.7
3T	-	-2.9	-1.0	1003	-2.9	-1.0
4T	-	-2.5	-0.6	438	-1.1	-0.3



Loadcase AT

Design wind pressures – Loadcase BT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft²)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-13.4	-5.8	723	-9.7	-4.2
2	-0.69	-18.6	-10.9	1655	-30.7	-18.0
3	-0.37	-11.7	-4.1	1655	-19.4	-6.7
4	-0.45	-13.4	-5.8	723	-9.7	-4.2
5	0.40	4.7	12.4	400	1.9	4.9
6	-0.29	-10.0	-2.3	400	-4.0	-0.9
1E	-0.48	-14.1	-6.4	153	-2.2	-1.0
2E	-1.07	-26.7	-19.0	350	-9.3	-6.7
3E	-0.53	-15.1	-7.5	350	-5.3	-2.6
4E	-0.48	-14.1	-6.4	153	-2.2	-1.0
5E	0.61	9.2	16.9	38	0.4	0.6
6E	-0.43	-13.0	-5.3	77	-1.0	-0.4
5T	-	1.2	3.1	438	0.5	1.4
6T	-	-2.5	-0.6	438	-1.1	-0.3

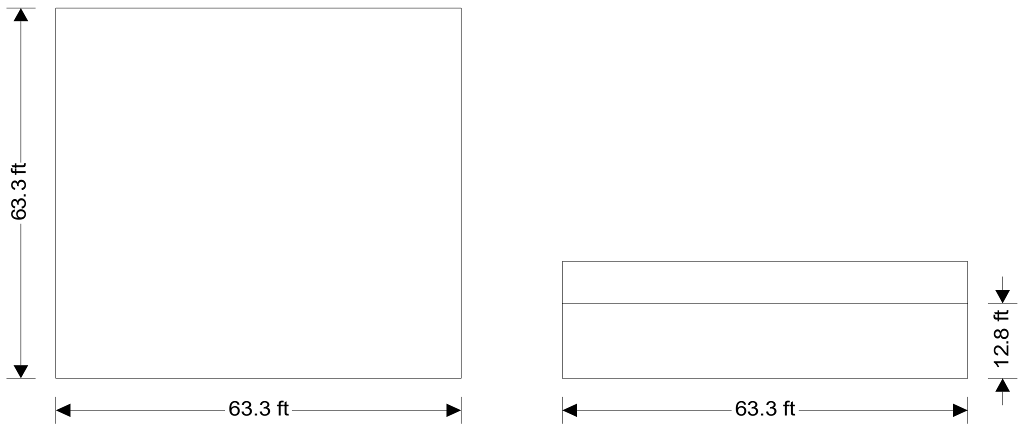

4
3
6T
6
2
4E
3E
6E
5T
13.8 ft
1
2E
31.7 ft
5
52.3 ft
1E
5E
26.1 ft
11.1 ft
5.5 ft
Loadcase BT

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.03



Plan

Elevation

Building data

Type of roof	Flat
Length of building	b = **63.33** ft
Width of building	d = **63.33** ft
Height to eaves	H = **12.83** ft
Height of parapet	h_p = **7.17** ft
Mean height	h = **12.83** 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} = **814** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **0.97**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Parapet internal pressure coef +ve (Table 26.11-1)	GC_{pi_pp} = **0.18**
Parapet internal pressure coef –ve (Table 26.11-1)	GC_{pi_np} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **21.3** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.90**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **22.6** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **21.34** psf

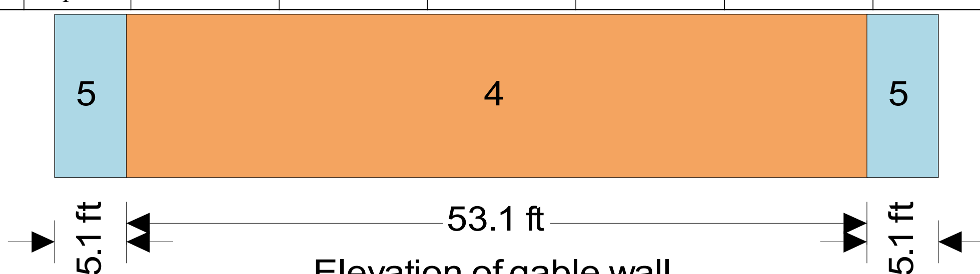
Equations used in tables

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

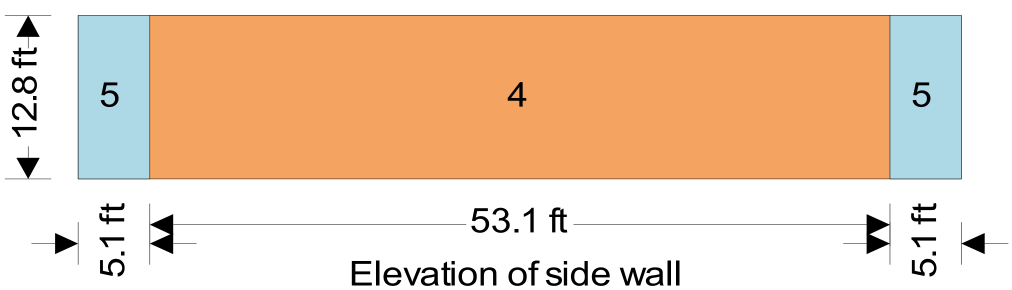
Parapet net pressure $p = q_p \times [GC_p - GC_{pi\_p}]$

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)
<10 sf	4	-	-	10.0	0.90	-0.99	23.0	-25.0
20 sf	4	-	-	20.0	0.85	-0.94	22.0	-23.9
50 sf	4	-	-	50.0	0.79	-0.88	20.7	-22.6
>100 sf	4	-	-	100.0	0.74	-0.83	19.7	-21.6
<10 sf	5	-	-	10.0	0.90	-1.26	23.0	-30.7
20 sf	5	-	-	20.0	0.85	-1.16	22.0	-28.7
50 sf	5	-	-	50.0	0.79	-1.04	20.7	-26.0
>100 sf	5	-	-	100.0	0.74	-0.94	19.7	-23.9
>10 sf (W)	5p	-	-	10.0	0.90	-2.30	24.4	-56.0
20 sf (W)	5p	-	-	20.0	0.85	-2.14	23.3	-52.4



Elevation of gable wall

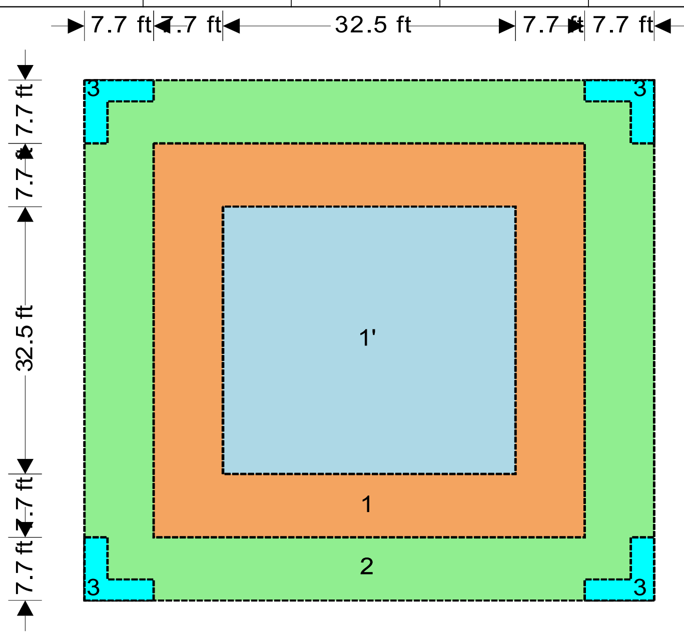


Elevation of side wall

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)
<=10 sf	1	-	-	10.0	0.30	-1.70	10.2 #	-40.1
20 sf	1	-	-	20.0	0.27	-1.58	9.6 #	-37.5
50 sf	1	-	-	50.0	0.23	-1.41	8.8 #	-34.0
>100 sf	1	-	-	100.0	0.20	-1.29	8.1 #	-31.3

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1'	-	-	10.0	0.30	-0.90	10.2 #	-23.0
20 sf	1'	-	-	20.0	0.27	-0.90	9.6 #	-23.0
50 sf	1'	-	-	50.0	0.23	-0.90	8.8 #	-23.0
>100 sf	1'	-	-	100.0	0.20	-0.90	8.1 #	-23.0
<=10 sf	2	-	-	10.0	0.90	-2.30	23.0	-52.9
20 sf	2	-	-	20.0	0.85	-2.14	22.0	-49.5
50 sf	2	-	-	50.0	0.79	-1.93	20.7	-45.0
>100 sf	2	-	-	100.0	0.74	-1.77	19.7	-41.6
<=10 sf	3	-	-	10.0	0.90	-2.30	23.0	-52.9
20 sf	3	-	-	20.0	0.85	-2.14	22.0	-49.5
50 sf	3	-	-	50.0	0.79	-1.93	20.7	-45.0
>100 sf	3	-	-	100.0	0.74	-1.77	19.7	-41.6



Plan on roof

Hazards by Location

Search Information

Coordinates: 38.93815140446288, -94.44568710947266

Elevation: 814 ft

Timestamp: 2021-06-22T15:06:31.052Z

Hazard Type: Snow

ASCE 7-16		ASCE 7-10		ASCE 7-05	
Ground Snow Load	20 lb/sqft	Ground Snow Load	20 lb/sqft	Ground Snow Load	20 lb/sqft

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer.

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SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details

Roof type	Flat
Width of roof	b = **61.00** 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

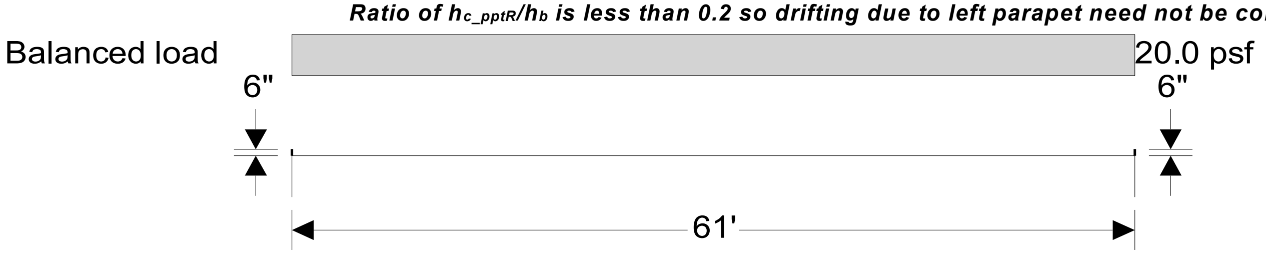
Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height of left parapet	h_{pptL} = **0.50** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **-0.34** ft

Ratio of h_{c_pptL}/h_b is less than 0.2 so drifting due to left parapet need not be considered

Right parapet

Height of right parapet	h_{pptR} = **0.50** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **-0.34** ft

Ratio of h_{c_pptR}/h_b is less than 0.2 so drifting due to left parapet need not be considered



Roof elevation

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details

Roof type	Flat
Width of roof	b = **40.00** ft

Ground snow load

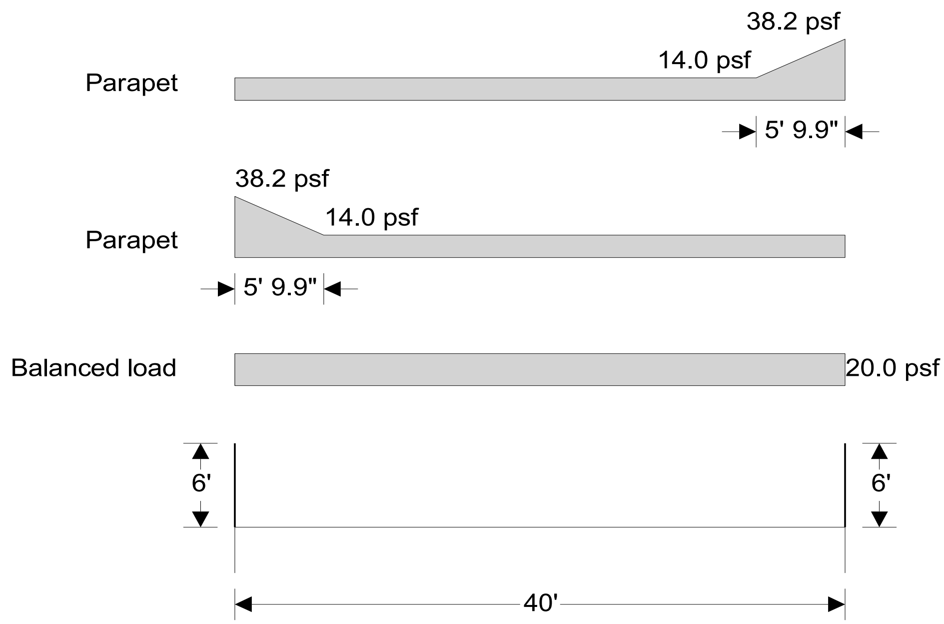
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal 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} = **6.00** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **5.16** ft
Length of roof - left parapet	$l_{u\_pptL} = b$ = **40.00** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.46** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.46** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **5.82** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **24.17** lb/ft^2

Right parapet

Height of right parapet	h_{pptR} = **6.00** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **5.16** ft
Length of roof - right parapet	$l_{u\_pptR} = b$ = **40.00** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.46** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.46** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **5.82** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **24.17** 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 = **63.33** ft

Ground snow load

Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3)$ = **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

Roof projection drifts

Max height of obstruction	$h_{obs} = \max(h_{1\_obs}, h_{2\_obs})$ = **6.00** ft
Width of obstruction	b_{obs} = **0.75** ft
Distance from LHS eaves	b_{1_obs} = **38.38** ft
Distance from RHS eaves	b_{2_obs} = **24.21** ft
Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height from balance load to top of projection	$h_{c\_obs} = h_{obs} - h_b$ = **5.16** ft
Length of lower roof	$l_{l\_obs} = \max(b_{1\_obs}, b_{2\_obs})$ = **38.38** ft
Drift height windward drift	$h_{d\_l\_obs} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{l\_obs}) \times 1ft^2)^{1/3} \times (p_g / 1lb/ft^2 + 10)^{1/4} - 1.5ft)$ = **1.42** ft
Drift height	$h_{d\_obs} = \min(h_{d\_l\_obs}, h_{obs} - h_b)$ = **1.42** ft
Drift width	$W_{d\_obs} = \min(4 \times h_{d\_l\_obs}, 8 \times (h_{obs} - h_b))$ = **5.68** ft
Drift surcharge load	$p_{d\_obs} = h_{d\_obs} \times \gamma$ = **23.59** lb/ft^2

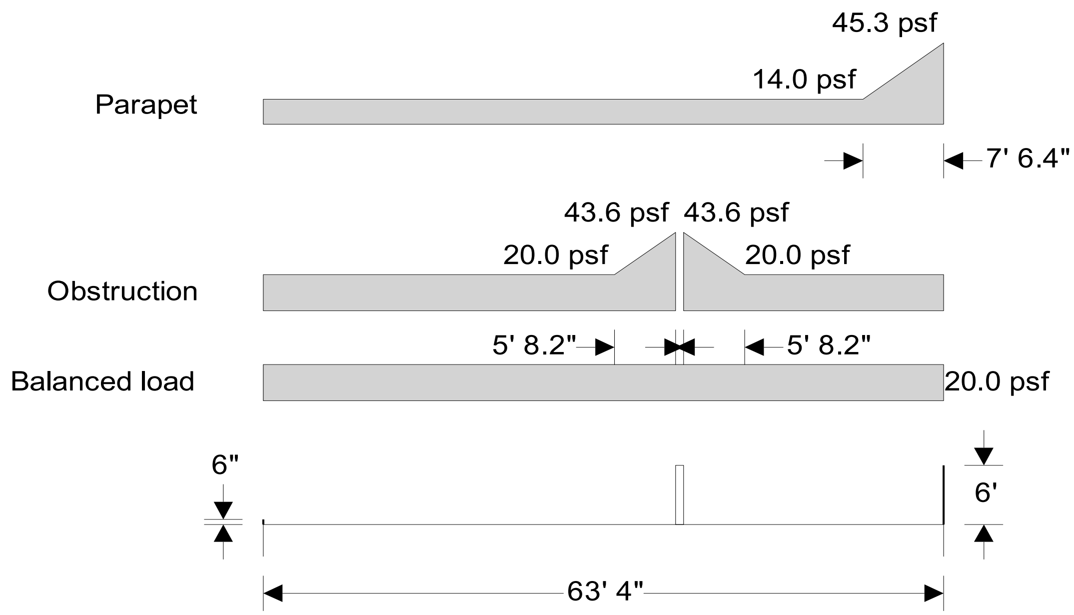
Left parapet

Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Height of left parapet	h_{pptL} = **0.50** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **-0.34** ft

Ratio of h_{c_pptL}/h_b is less than 0.2 so drifting due to left parapet need not be considered

Right parapet

Height of right parapet	h_{pptR} = **6.00** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **5.16** ft
Length of roof - right parapet	$l_{u\_pptR} = b$ = **63.33** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1ft^2)^{1/3} \times (p_g / 1lb/ft^2 + 10)^{1/4} - 1.5ft)$ = **1.88** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.88** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **7.53** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **31.27** lb/ft^2



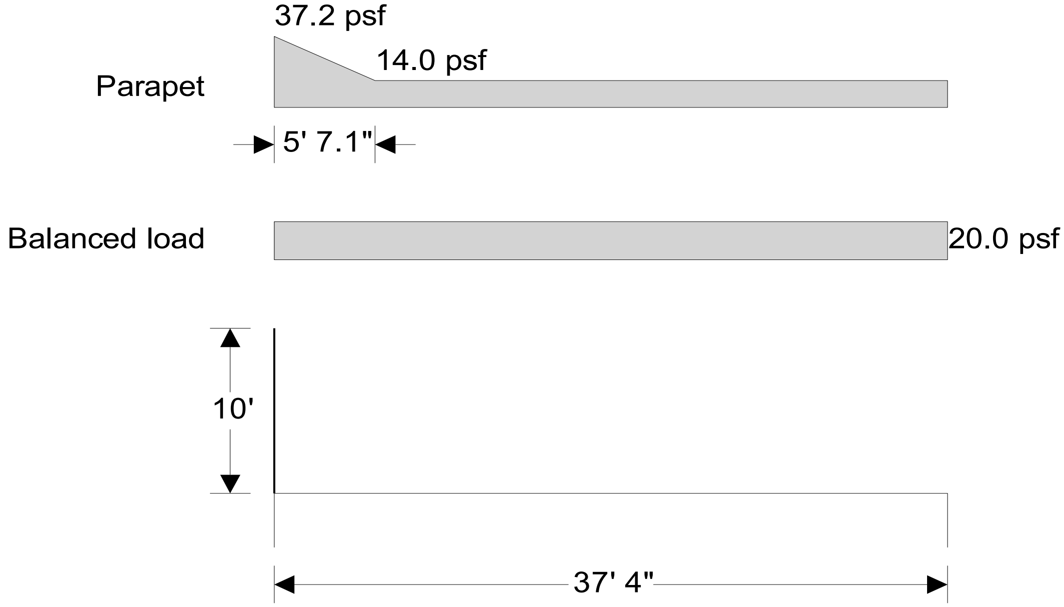
Roof elevation

SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details	
Roof type	Flat
Width of roof	b = **37.33** ft
Ground snow load	
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3)$ = **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} = **10.00** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **9.16** ft
Length of roof - left parapet	l_{u_pptL} = b = **37.33** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = \sqrt{(I_s)} \times 0.75 \times (0.43 \times (\max(20\ ft, l_{u\_pptL}) \times 1ft^2)^{1/3} \times (p_g / 1lb/ft^2 + 10)^{1/4} - 1.5ft)$ = **1.40** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.40** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **5.59** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **23.20** lb/ft^2



Roof elevation

SNOW LOADING

In accordance with ASCE7-16

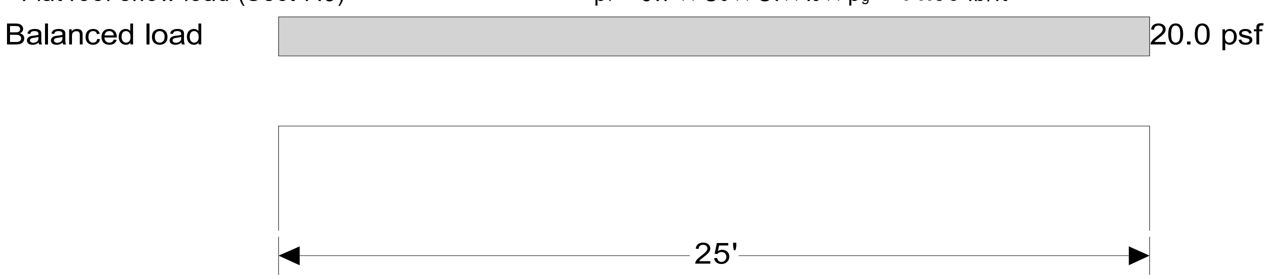
Tedds calculation version 1.0.09

Building details

Roof type	Flat
Width of roof	b = **25.00** ft

Ground snow load

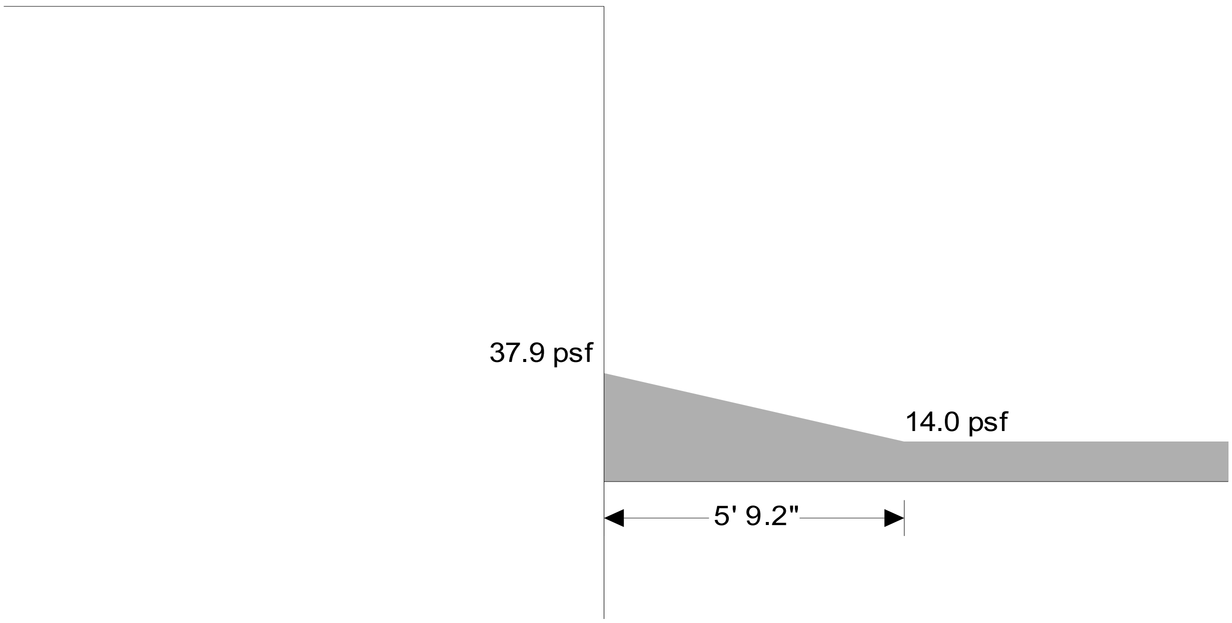
Ground snow load (Figure 7.2-1)	p_g = **20.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **16.60** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7.3-1)	Partially exposed
Exposure factor (Table 7.3-1)	C_e = **1.00**
Thermal 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



Roof elevation

Drift calculations

Balanced snow load height	$h_b = p_f / \gamma$ = **0.84** ft
Length of upper roof	l_u = **25.00** ft
Length of lower roof	l_l = **12.00** ft
Height diff between uppper and lower roofs	h_{diff} = **10.00** ft
Height from balance load to top of upper roof	$h_c = h_{diff} - h_b$ = **9.16** ft
Drift height leeward drift	$h_{d\_l} = \min(\sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}), 0.6 \times l_l)$ = **1.44** ft
Drift height windward drift	$h_{d\_w} = \min(0.75 \times \sqrt{(I_s)} \times (0.43 \times (\max(20\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}), \sqrt{(I_s \times p_g \times l_l / (4 \times \gamma))})$ = **0.92** ft
Maximum lw/ww drift height	$h_{d\_max} = \max(h_{d\_w}, h_{d\_l})$ = **1.44** ft
Drift height	$h_d = \min(h_{d\_max}, h_c)$ = **1.44** ft
Drift width	$W_d = \min(4 \times h_{d\_max}, 8 \times h_c)$ = **5.77** ft
Drift surcharge load	$p_d = h_d \times \gamma$ = **23.95** lb/ft^2



Elevation on snow drift

Summary

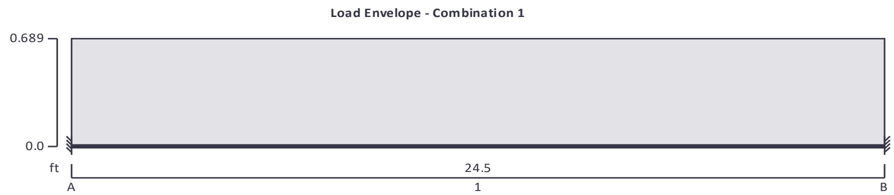
The gravity structure system of the project referenced above consists primarily of steel beams. The beams are supported by steel columns which are supported by the foundations. The locations of all steel framing are indicated on the structural framing plans.

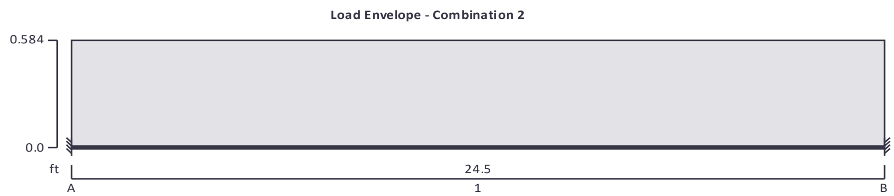
The following section of calculations covers the complete design of the steel gravity framing for the project referenced above. Refer to the "Loads" section of these calculations for the determination of all loads. Refer to the "Foundation" section of these calculations for the design of the continuous and spread footings supporting the steel framing.

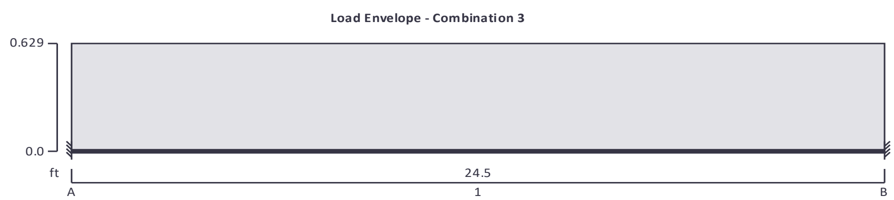
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

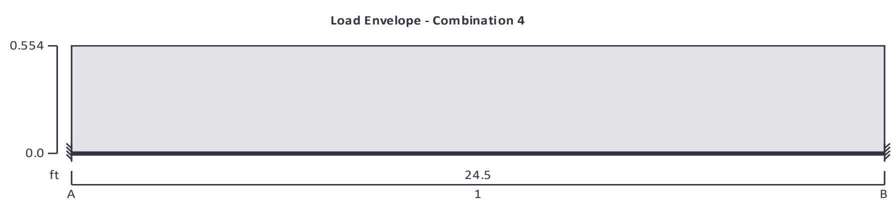
In accordance with AISC360-16 using the LRFD method

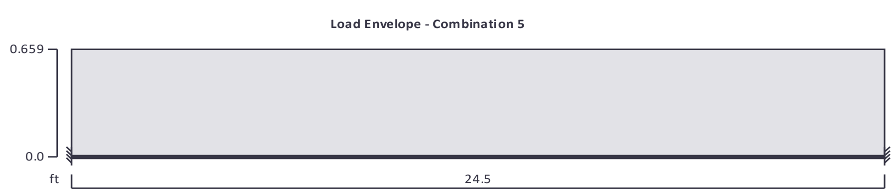
Tedds calculation version 3.0.14

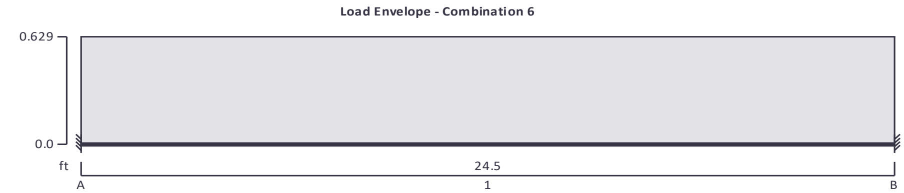


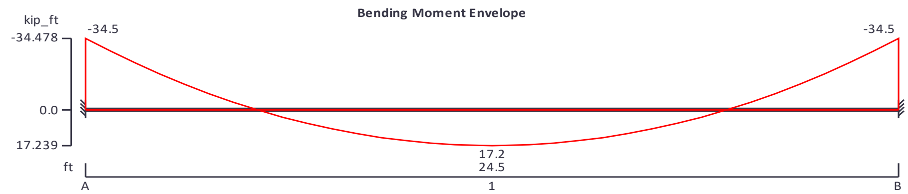


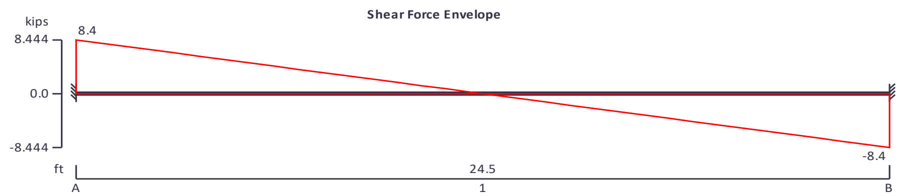












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.239 kips/ft
	Live full UDL 0.05 kips/ft
	Snow full UDL 0.05 kips/ft
	Wind full UDL 0.125 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60

		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00

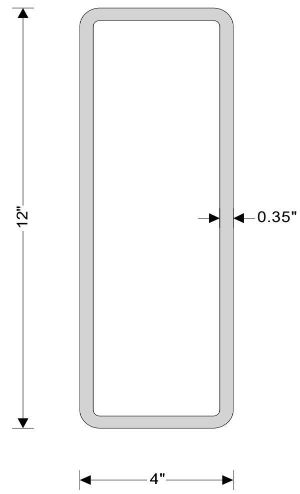
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **17.2** kips_ft	M_{min} = **-34.5** kips_ft
Maximum shear	V_{max} = **8.4** kips	V_{min} = **-8.4** kips
Deflection	δ_{max} = **0.1** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **8.4** kips	R_{A_min} = **6.8** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **3.4** kips	
Unfactored live load reaction at support A	R_{A_Live} = **0.6** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **1.5** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **0.6** kips	
Maximum reaction at support B	R_{B_max} = **8.4** kips	R_{B_min} = **6.8** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **3.4** kips	
Unfactored live load reaction at support B	R_{B_Live} = **0.6** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **1.5** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **0.6** kips	

Section details

Section type	**HSS 12x4x3/8 (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



Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

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 17)

Width to thickness ratio $(b_f - 3 \times t) / t$ = **8.46**

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 $(d - 3 \times t) / t$ = **31.38**

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 $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **8.444** kips

Web area $A_w = 2 \times (d - 3 \times t) \times t$ = **7.645** in^2

Web plate buckling coefficient k_v = **5**

Web shear coefficient - eq G2-9 C_{v2} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v2}$ = **211.007** kips

Resistance factor for shear ϕ_v = **0.90**

Design shear strength $V_c = \phi_v \times V_n$ = **189.907** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **34.478** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1 $M_{nyld} = M_p = F_y \times Z_x =$ **140.683** kips_ft

Lateral-torsional buckling - Section F7.4

Unbraced length $L_b = L_{s1} =$ **24.5** ft

Limiting unbraced length for yielding - eq F7-12 $L_p = 0.13 \times E \times r_y \times \sqrt{(J \times A)} / (F_y \times Z_x) =$ **9.19** ft

Limiting unbraced length for inelastic LTB - eq F7-13

$L_r = 2 \times E \times r_y \times \sqrt{(J \times A)} / (0.7 \times F_y \times S_x) =$ **264.77** ft

Lateral torsional buckling modification factor $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] =$ **2.38**

Nominal flexural strength for lateral torsional buckling - eq F7-10

$M_{nltb} = \min(C_b \times (M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_p) =$ **140.7** kip_ft

Nominal flexural strength $M_n = \min(M_{nyld}, M_{nltb}) =$ **140.683** kips_ft

Design flexural strength $M_c = \phi_b \times M_n =$ **126.615** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and live loads

Limiting deflection $\delta_{lim} = L_{s1} / 600 =$ **0.49** in

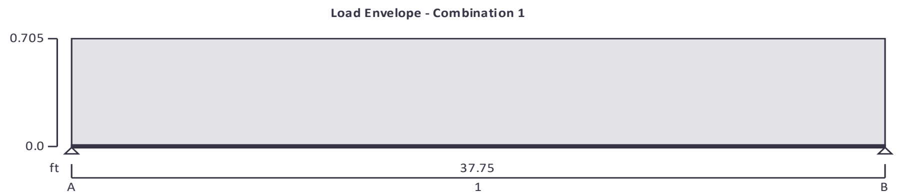
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.108** in

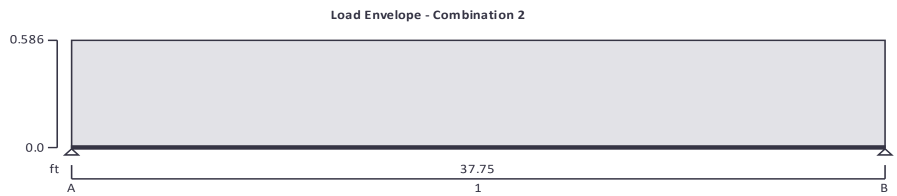
PASS - Maximum deflection does not exceed deflection limit

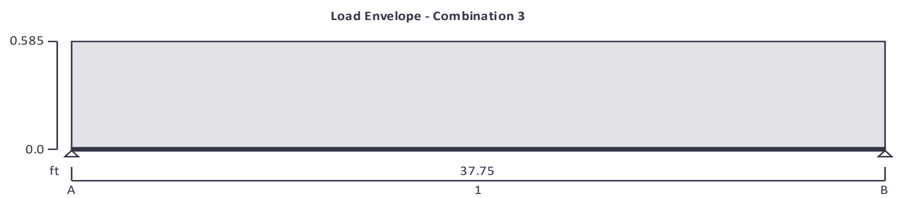
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

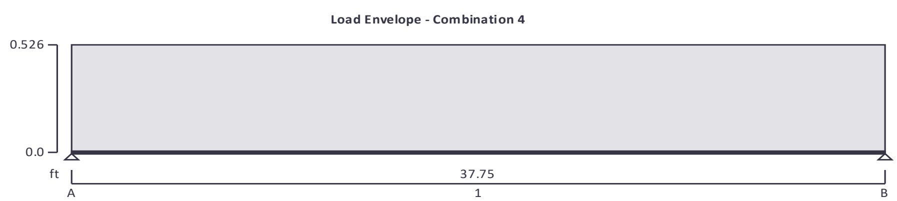
In accordance with AISC360-16 using the LRFD method

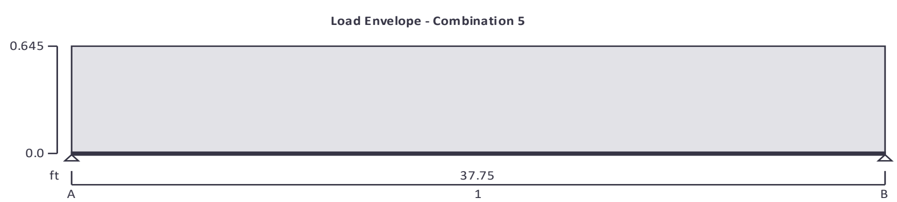
Tedds calculation version 3.0.14

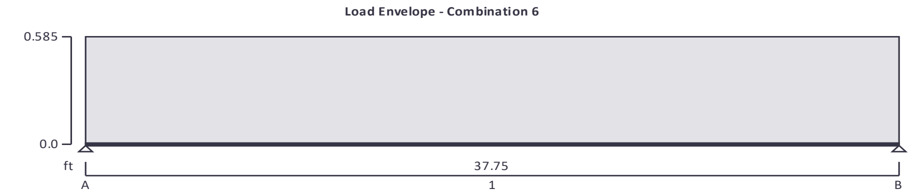


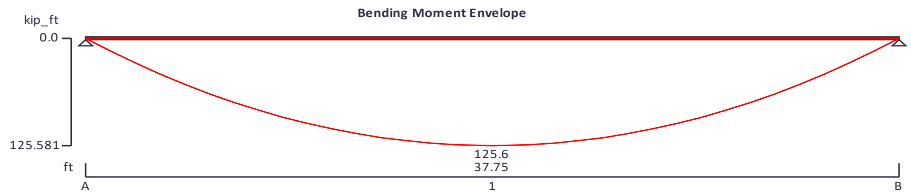


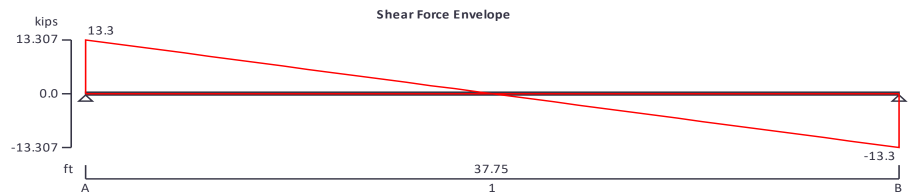












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.15 kips/ft
	Live full UDL 0.1 kips/ft
	Snow full UDL 0.1 kips/ft
	Wind full UDL 0.098 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60

		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00

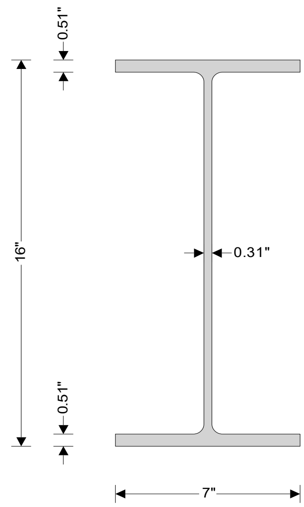
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **125.6** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **13.3** kips	V_{min} = **-13.3** kips
Deflection	δ_{max} = **1.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **13.3** kips	R_{A_min} = **9.9** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **3.6** kips	
Unfactored live load reaction at support A	R_{A_Live} = **1.9** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **1.8** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **1.9** kips	
Maximum reaction at support B	R_{B_max} = **13.3** kips	R_{B_min} = **9.9** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **3.6** kips	
Unfactored live load reaction at support B	R_{B_Live} = **1.9** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **1.8** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **1.9** kips	

Section details

Section type	**W 16x40 (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



Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio $b_f / (2 \times t_f)$ = **6.93**

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 $(d - 2 \times k) / t_w$ = **46.51**

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 $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **13.307** kips

Web area $A_w = d \times t_w$ = **4.88** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **146.400** kips

Resistance factor for shear ϕ_v = **1.00**

Design shear strength $V_c = \phi_v \times V_n$ = **146.400** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **125.581** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x =$ **304.167** kips_ft

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240 =$ **1.888** in

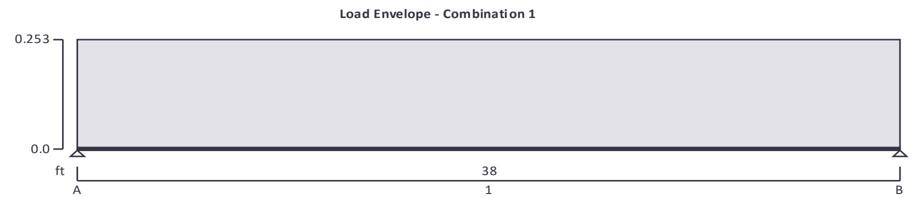
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **1.187** in

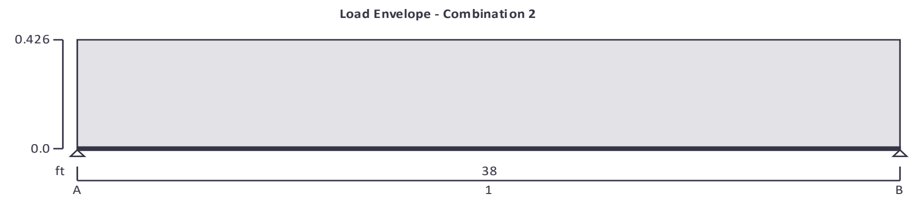
PASS - Maximum deflection does not exceed deflection limit

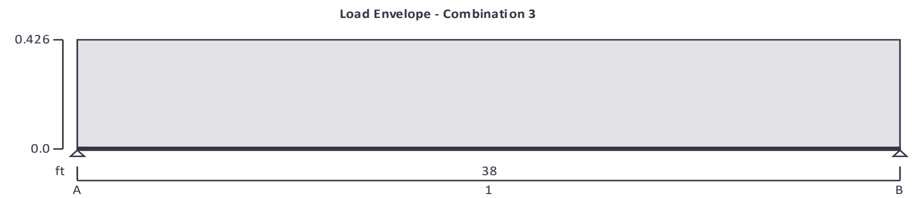
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

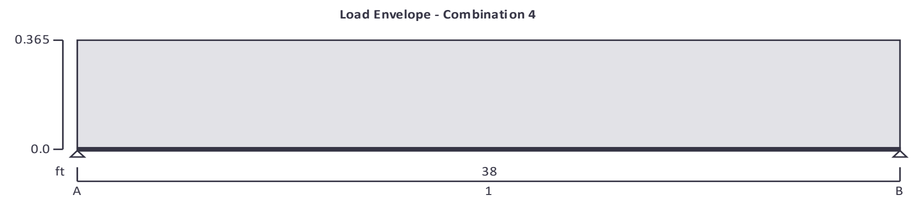
In accordance with AISC360-16 using the LRFD method

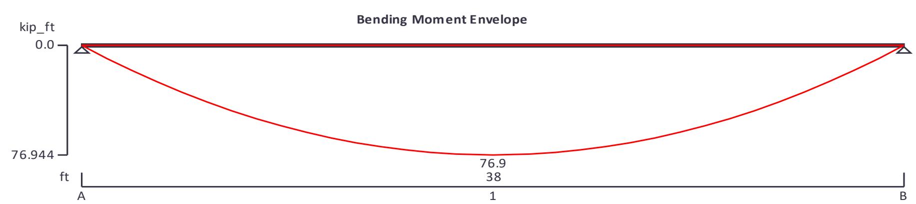
Tedds calculation version 3.0.15

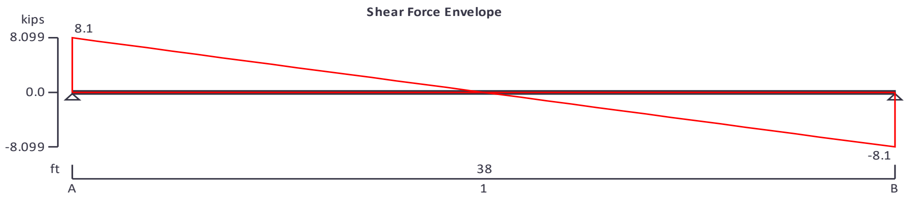












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.15 kips/ft
	Roof Live full UDL 0.1 kips/ft
	Snow full UDL 0.1 kips/ft
	Wind full UDL 0.098 kips/ft

Load combinations

Load combination 1 - 1.4DL	Support A	Dead × 1.40
		Dead × 1.40
	Support B	Dead × 1.40
Load combination 2 - 1.2DL + 1.6RLL + 0.5WL	Support A	Dead × 1.20
		Roof Live × 1.60
		Wind × 0.50
		Dead × 1.20
		Roof Live × 1.60
		Wind × 0.50
	Support B	Dead × 1.20
		Roof Live × 1.60
		Wind × 0.50
Load combination 3 - 1.2DL + 1.6SL + 0.5WL	Support A	Dead × 1.20
		Wind × 0.50
		Snow × 1.60
		Dead × 1.20
		Wind × 0.50
		Snow × 1.60
	Support B	Dead × 1.20
		Wind × 0.50
		Snow × 1.60
Load combination 4 - 1.2DL + 1.0WL + 0.5RLL	Support A	Dead × 1.20
		Roof Live × 0.50
		Wind × 1.00
		Dead × 1.20
		Roof Live × 0.50

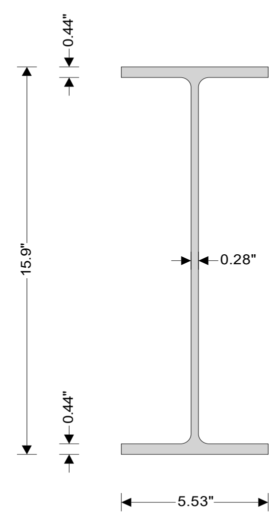
		Wind × 1.00
	Support B	Dead × 1.20
		Roof Live × 0.50
		Wind × 1.00

Analysis results

Maximum moment	M_{max} = **76.9** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **8.1** kips	V_{min} = **-8.1** kips
Deflection	δ_{max} = **1.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **8.1** kips	R_{A_min} = **4.8** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **3.4** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof Live}$ = **1.9** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **1.9** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **1.9** kips	
Maximum reaction at support B	R_{B_max} = **8.1** kips	R_{B_min} = **4.8** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **3.4** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof Live}$ = **1.9** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **1.9** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **1.9** kips	

Section details

Section type	**W 16x31 (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



Resistance factors

Resistance factor for tensile yielding	ϕ_{ty} = **0.90**
Resistance factor for tensile rupture	ϕ_{tr} = **0.75**
Resistance factor for compression	ϕ_c = **0.90**
Resistance factor for flexure	ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **6.28**	
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	$(d - 2 \times k) / t_w$ = **51.69**	
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	$V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **8.099** kips
Web area	$A_w = d \times t_w$ = **4.373** in^2
Web plate buckling coefficient	k_v = **5.34**
Web shear coefficient - eq G2-3	C_{v1} = **1**
Nominal shear strength – eq G6-1	$V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **131.175** kips
Resistance factor for shear	ϕ_v = **1.00**
Design shear strength	$V_c = \phi_v \times V_n$ = **131.175** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **76.944** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1	$M_{nyld} = M_p = F_y \times Z_x$ = **225** kips_ft
Nominal flexural strength	$M_n = M_{nyld}$ = **225.000** kips_ft
Design flexural strength	$M_c = \phi_b \times M_n$ = **202.500** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and snow loads

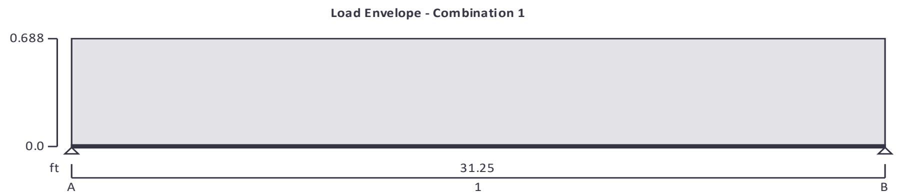
Limiting deflection	$\delta_{lim} = L_{s1} / 240$ = **1.9** in
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **1.213** in

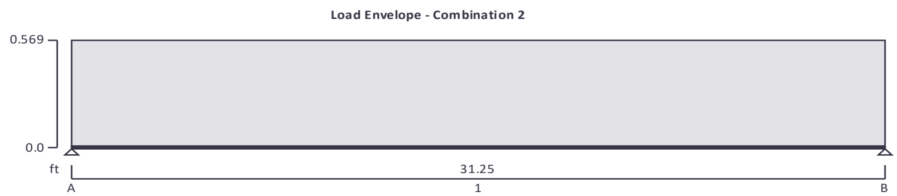
PASS - Maximum deflection does not exceed deflection limit

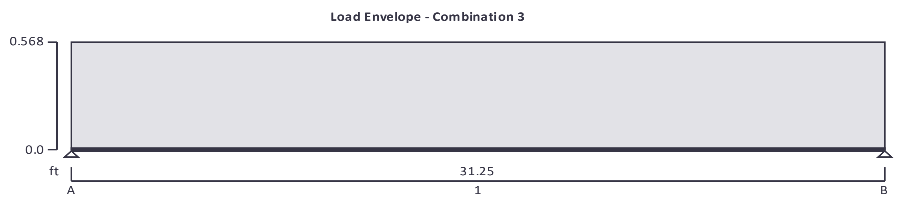
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

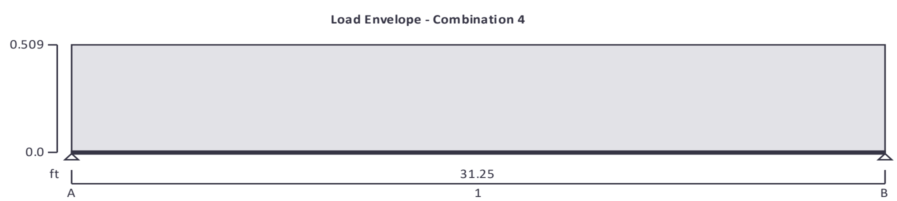
In accordance with AISC360-16 using the LRFD method

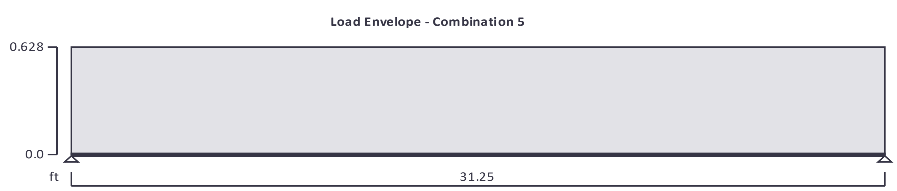
Tedds calculation version 3.0.14

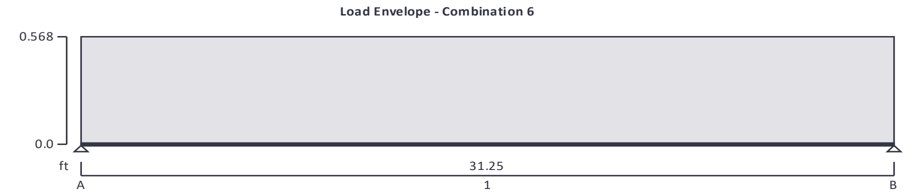


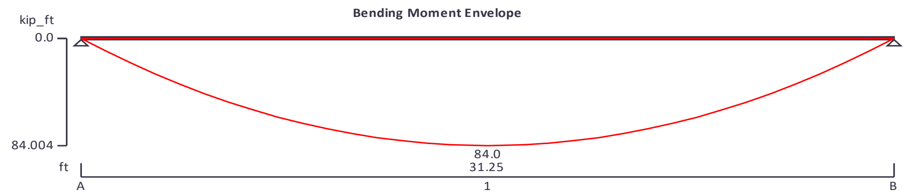


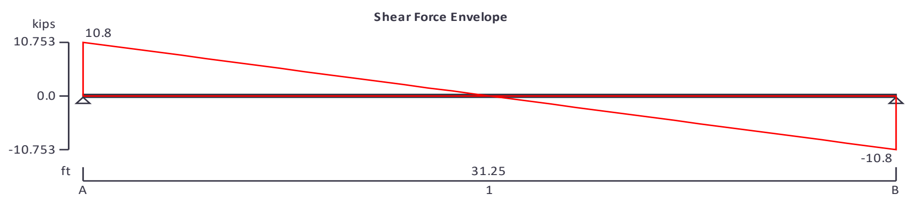












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.15 kips/ft
	Live full UDL 0.1 kips/ft
	Snow full UDL 0.1 kips/ft
	Wind full UDL 0.098 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60

		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00

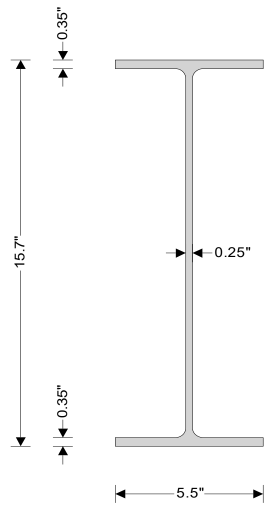
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **84** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **10.8** kips	V_{min} = **-10.8** kips
Deflection	δ_{max} = **0.9** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **10.8** kips	R_{A_min} = **8** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **2.8** kips	
Unfactored live load reaction at support A	R_{A_Live} = **1.6** kips	
Unfactored wind load reaction at support A	R_{A_Wind} = **1.5** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **1.6** kips	
Maximum reaction at support B	R_{B_max} = **10.8** kips	R_{B_min} = **8** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **2.8** kips	
Unfactored live load reaction at support B	R_{B_Live} = **1.6** kips	
Unfactored wind load reaction at support B	R_{B_Wind} = **1.5** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **1.6** kips	

Section details

Section type	**W 16x26 (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



Resistance factors

Resistance factor for tensile yielding	ϕ_{ty} = **0.90**
Resistance factor for tensile rupture	ϕ_{tr} = **0.75**
Resistance factor for compression	ϕ_c = **0.90**
Resistance factor for flexure	ϕ_b = **0.90**

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	$b_f / (2 \times t_f)$ = **7.97**	
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	$(d - 2 \times k) / t_w$ = **56.82**	
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	$V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **10.753** kips
Web area	$A_w = d \times t_w$ = **3.925** in^2
Web plate buckling coefficient	k_v = **5.34**
Web shear coefficient - eq G2-3	C_{v1} = **1**
Nominal shear strength – eq G6-1	$V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **117.750** kips
Resistance factor for shear	ϕ_v = **0.90**
Design shear strength	$V_c = \phi_v \times V_n$ = **105.975** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength	$M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **84.004** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x =$ **184.167** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **184.167** kips_ft

Design flexural strength $M_c = \phi_b \times M_n =$ **165.750** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240 =$ **1.563** in

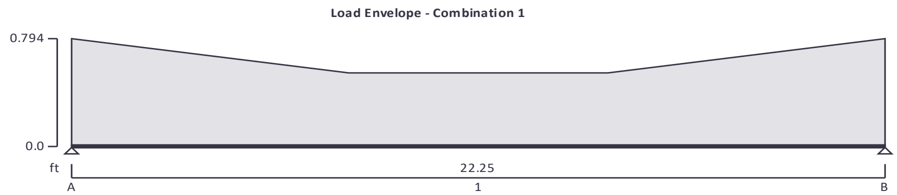
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **0.925** in

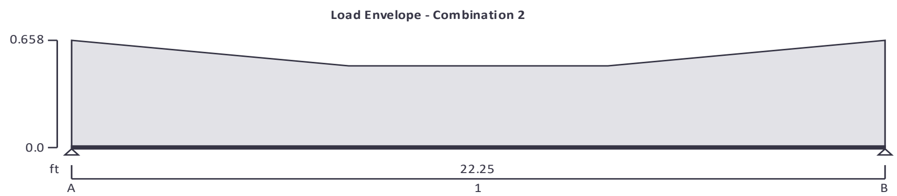
PASS - Maximum deflection does not exceed deflection limit

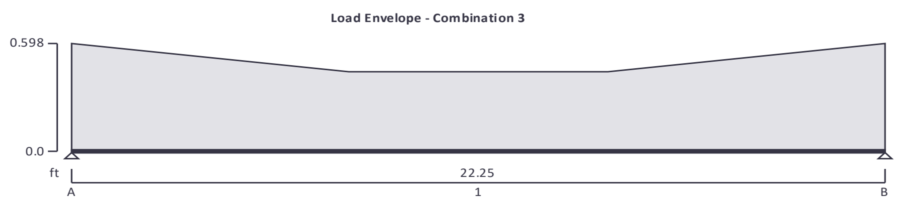
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

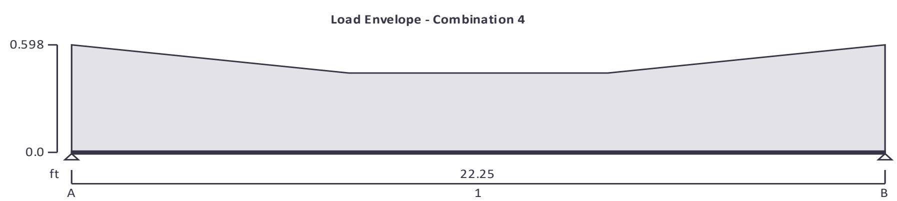
In accordance with AISC360-16 using the LRFD method

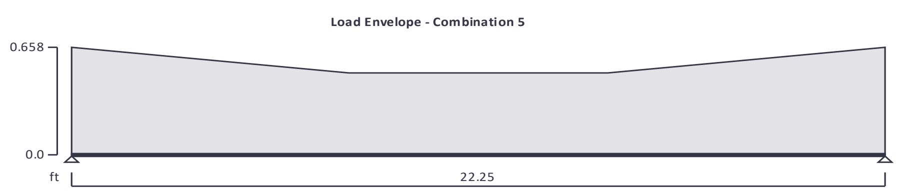
Tedds calculation version 3.0.14

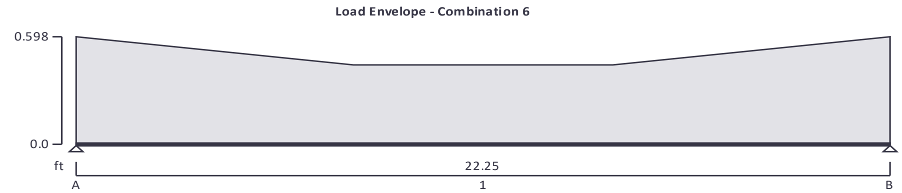


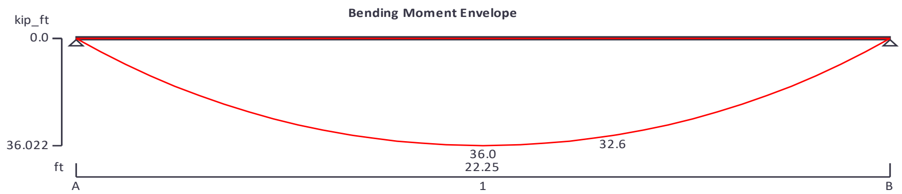


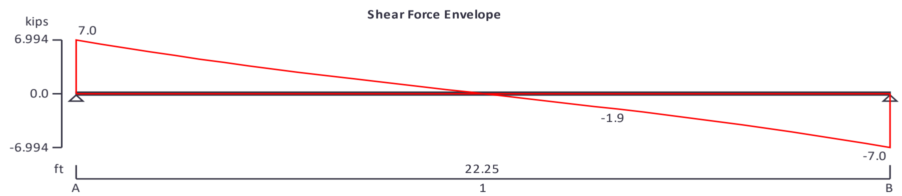












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.1 kips/ft
	Live full UDL 0.1 kips/ft
	Snow partial VDL 0.157 kips/ft at 0.00 in to 0 kips/ft at 90.96 in
	Snow partial VDL 0 kips/ft at 176.04 in to 0.157 kips/ft at 267.00 in
	Dead full UDL 0.1 kips/ft
	Snow full UDL 0.07 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20

		Live × 1.60
		Wind × 1.60
		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60

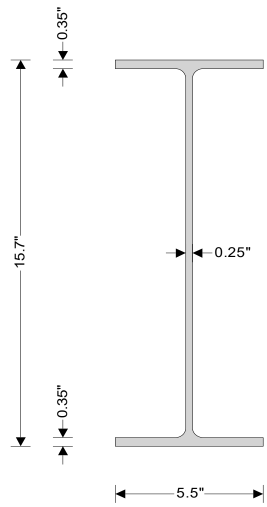
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **36** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **7** kips	V_{min} = **-7** kips
Deflection	δ_{max} = **0.3** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **7** kips	R_{A_min} = **5.5** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **2.5** kips	
Unfactored live load reaction at support A	R_{A_Live} = **1.1** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **1.4** kips	
Maximum reaction at support B	R_{B_max} = **7** kips	R_{B_min} = **5.5** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **2.5** kips	
Unfactored live load reaction at support B	R_{B_Live} = **1.1** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **1.4** kips	

Section details

Section type	**W 16x26 (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



Resistance factors

Resistance factor for tensile yielding — ϕ_{ty} = **0.90**

Resistance factor for tensile rupture — ϕ_{tr} = **0.75**

Resistance factor for compression — ϕ_c = **0.90**

Resistance factor for flexure — ϕ_b = **0.90**

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 — $b_f / (2 \times t_f)$ = **7.97**

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 — $(d - 2 \times k) / t_w$ = **56.82**

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 — $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **6.994** kips

Web area — $A_w = d \times t_w$ = **3.925** in^2

Web plate buckling coefficient — k_v = **5.34**

Web shear coefficient - eq G2-3 — C_{v1} = **1**

Nominal shear strength – eq G6-1 — $V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **117.750** kips

Resistance factor for shear — ϕ_v = **0.90**

Design shear strength — $V_c = \phi_v \times V_n$ = **105.975** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength — $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **36.022** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **184.167** kips_ft

Nominal flexural strength $M_n = M_{nyld}$ = **184.167** kips_ft

Design flexural strength $M_c = \phi_b \times M_n$ = **165.750** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240$ = **1.113** in

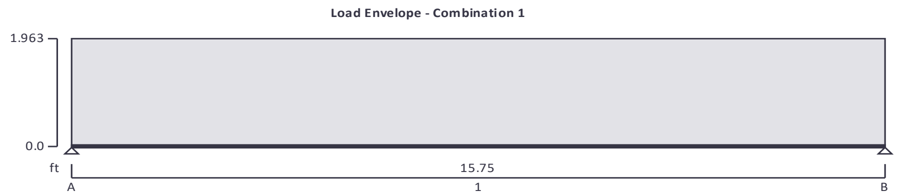
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.268** in

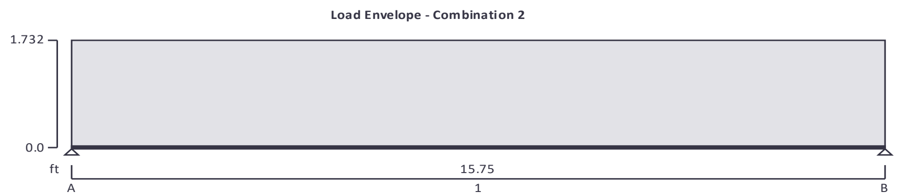
PASS - Maximum deflection does not exceed deflection limit

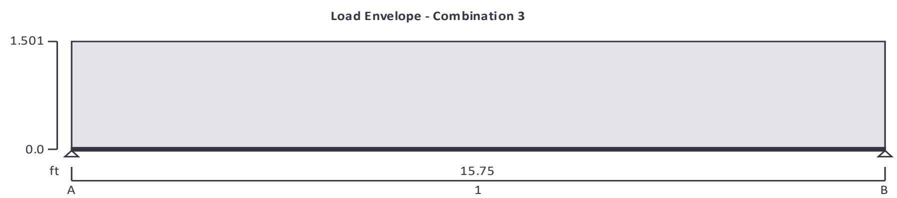
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

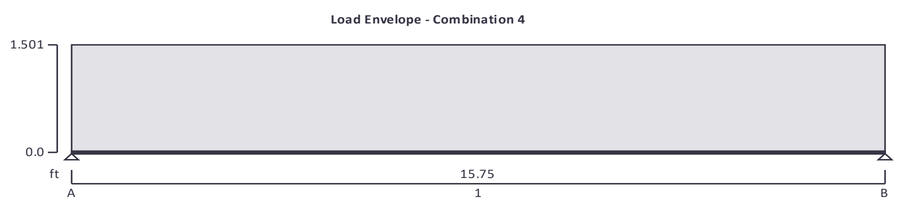
In accordance with AISC360-16 using the LRFD method

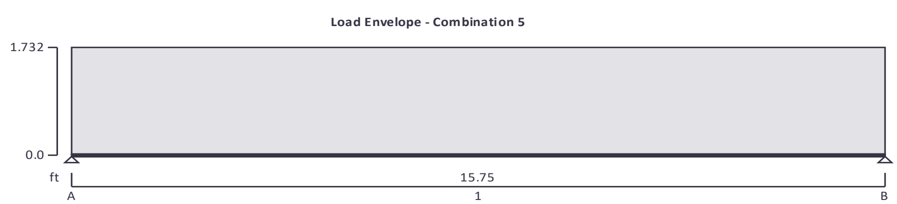
Tedds calculation version 3.0.14

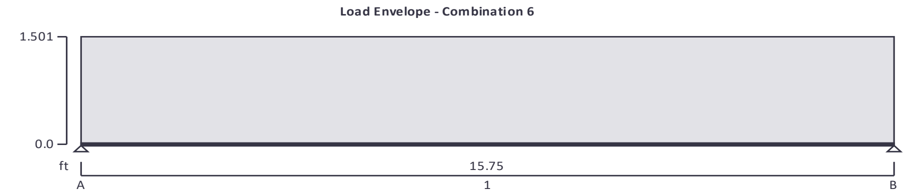


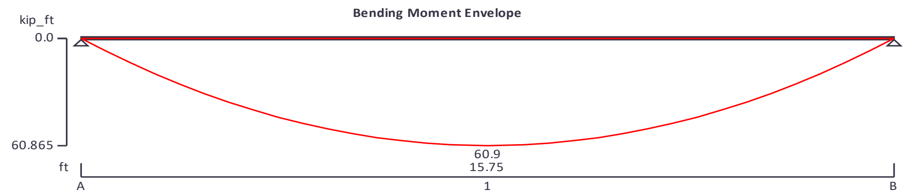


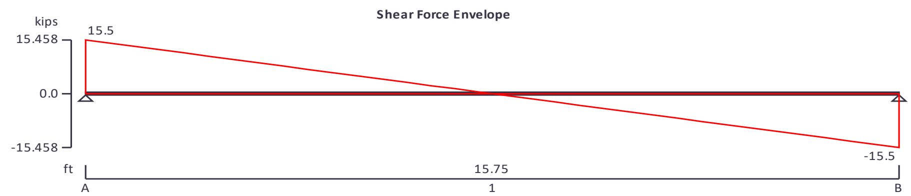












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.578 kips/ft
	Live full UDL 0.385 kips/ft
	Snow full UDL 0.385 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60

		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00

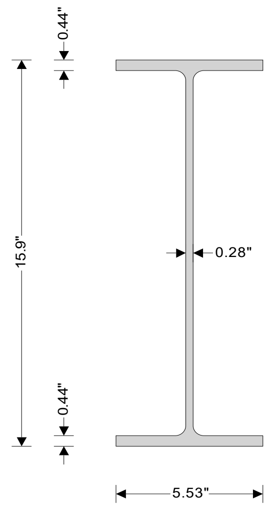
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **60.9** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **15.5** kips	V_{min} = **-15.5** kips
Deflection	δ_{max} = **0.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **15.5** kips	R_{A_min} = **11.8** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **4.8** kips	
Unfactored live load reaction at support A	R_{A_Live} = **3** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **3** kips	
Maximum reaction at support B	R_{B_max} = **15.5** kips	R_{B_min} = **11.8** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **4.8** kips	
Unfactored live load reaction at support B	R_{B_Live} = **3** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **3** kips	

Section details

Section type	**W 16x31 (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



Resistance factors

Resistance factor for tensile yielding	ϕ_{ty} = **0.90**
Resistance factor for tensile rupture	ϕ_{tr} = **0.75**
Resistance factor for compression	ϕ_c = **0.90**
Resistance factor for flexure	ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **6.28**	
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	$(d - 2 \times k) / t_w$ = **51.69**	
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	$V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **15.458** kips
Web area	$A_w = d \times t_w$ = **4.373** in^2
Web plate buckling coefficient	k_v = **5.34**
Web shear coefficient - eq G2-3	C_{v1} = **1**
Nominal shear strength – eq G6-1	$V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **131.175** kips
Resistance factor for shear	ϕ_v = **1.00**
Design shear strength	$V_c = \phi_v \times V_n$ = **131.175** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength	$M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **60.865** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **225** kips_ft

Nominal flexural strength $M_n = M_{nyld}$ = **225.000** kips_ft

Design flexural strength $M_c = \phi_b \times M_n$ = **202.500** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240$ = **0.788** in

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.176** in

PASS - Maximum deflection does not exceed deflection limit

STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

In accordance with AISC360-16 using the LRFD method

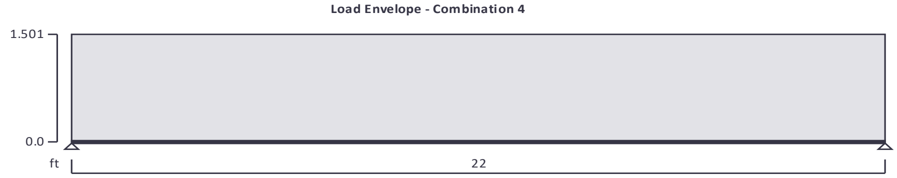
Tedds calculation version 3.0.14

Load Envelope - Combination 1

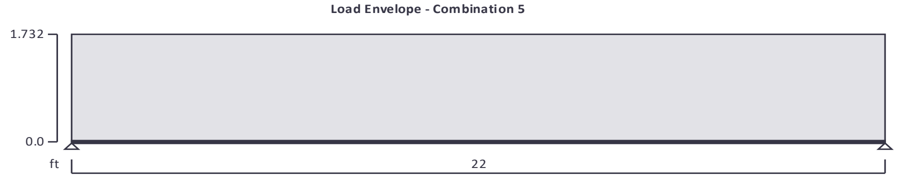
Load Envelope - Combination 2

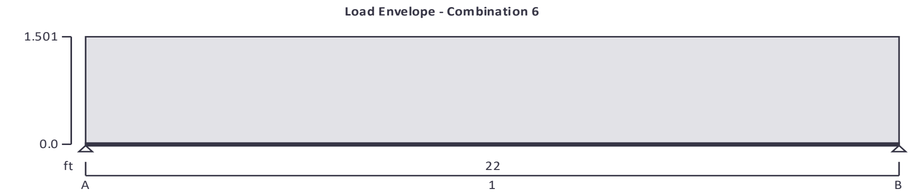
Load Envelope - Combination 3

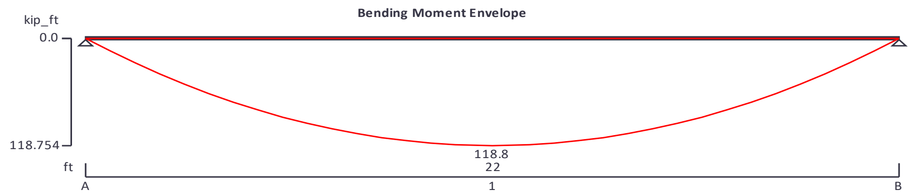
Load Envelope - Combination 4

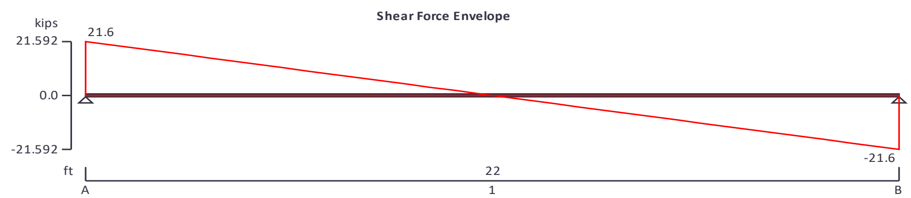


Load Envelope - Combination 5









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 0.578 kips/ft	
	Live full UDL 0.385 kips/ft	
	Snow full UDL 0.385 kips/ft	
Load combinations		
Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60

		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00

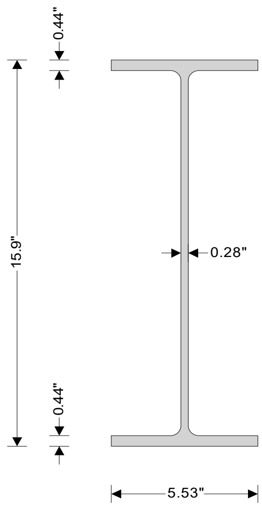
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **118.8** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **21.6** kips	V_{min} = **-21.6** kips
Deflection	δ_{max} = **0.7** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **21.6** kips	R_{A_min} = **16.5** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **6.7** kips	
Unfactored live load reaction at support A	R_{A_Live} = **4.2** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **4.2** kips	
Maximum reaction at support B	R_{B_max} = **21.6** kips	R_{B_min} = **16.5** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **6.7** kips	
Unfactored live load reaction at support B	R_{B_Live} = **4.2** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **4.2** kips	

Section details

Section type	**W 16x31 (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



Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio $b_f / (2 \times t_f)$ = **6.28**

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 $(d - 2 \times k) / t_w$ = **51.69**

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 $V_r = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **21.592** kips

Web area $A_w = d \times t_w$ = **4.373** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **131.175** kips

Resistance factor for shear ϕ_v = **1.00**

Design shear strength $V_c = \phi_v \times V_n$ = **131.175** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **118.754** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **225** kips_ft

Nominal flexural strength $M_n = M_{nyld}$ = **225.000** kips_ft

Design flexural strength $M_c = \phi_b \times M_n$ = **202.500** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240$ = **1.1** in

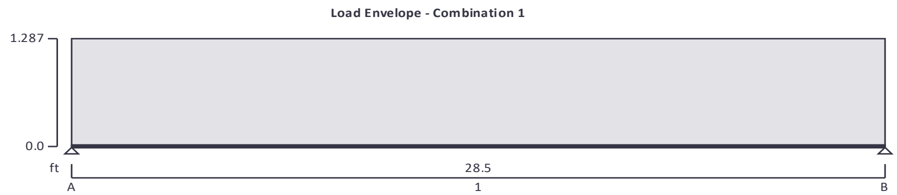
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **0.668** in

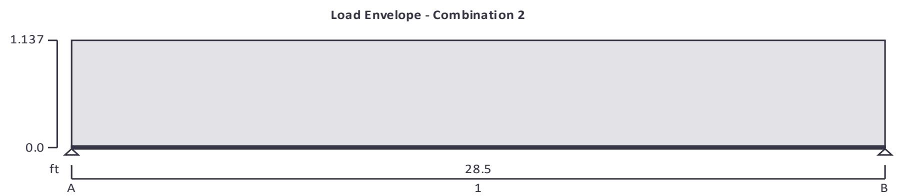
PASS - Maximum deflection does not exceed deflection limit

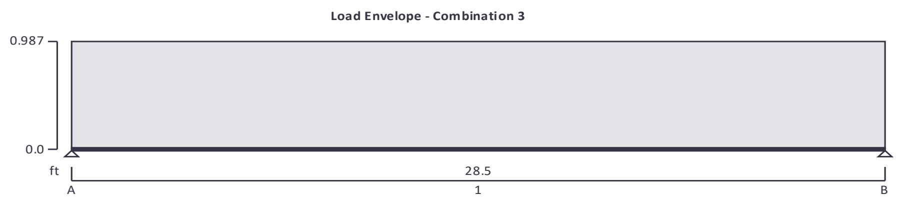
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

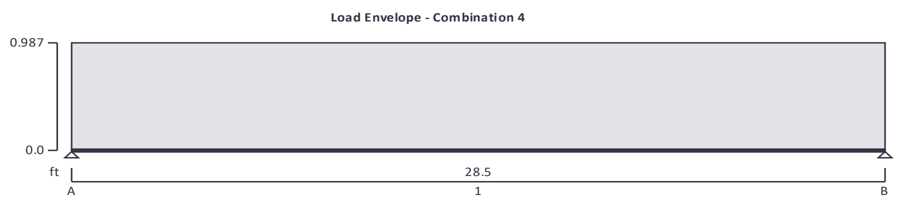
In accordance with AISC360-16 using the LRFD method

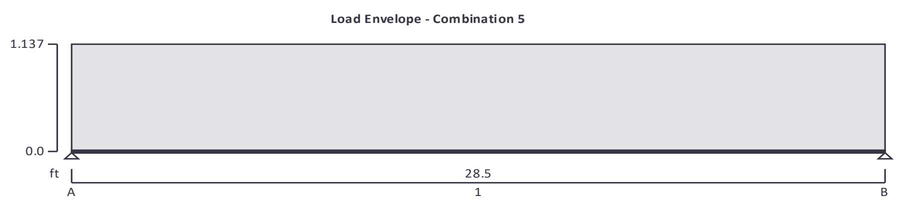
Tedds calculation version 3.0.14

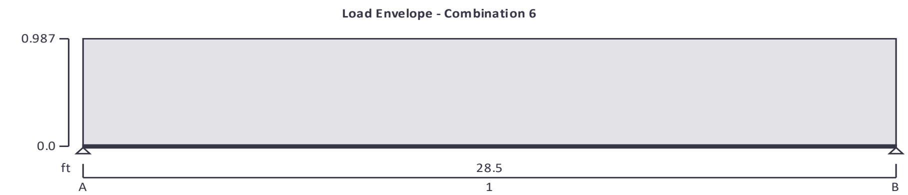


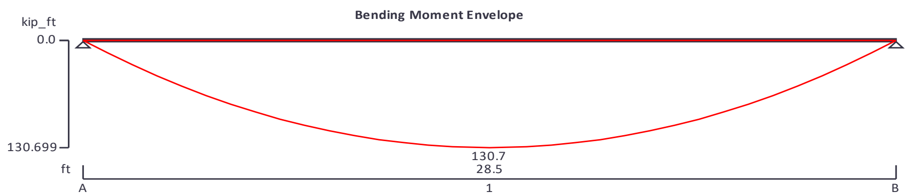


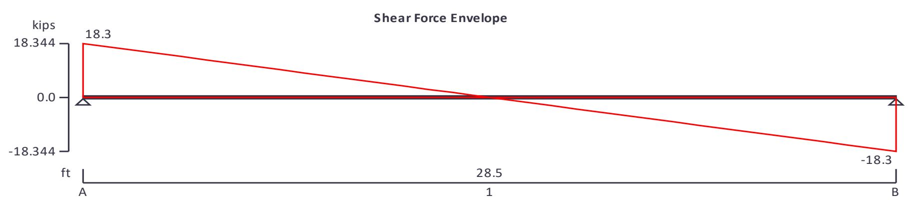












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.375 kips/ft
	Live full UDL 0.25 kips/ft
	Snow full UDL 0.25 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60

		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00

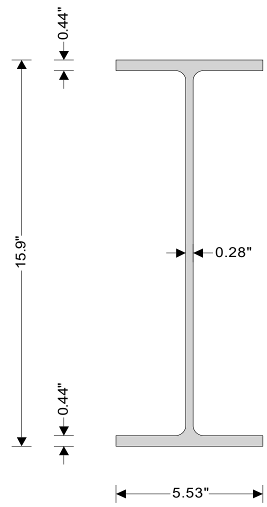
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **130.7** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **18.3** kips	V_{min} = **-18.3** kips
Deflection	δ_{max} = **1.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **18.3** kips	R_{A_min} = **14.1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **5.8** kips	
Unfactored live load reaction at support A	R_{A_Live} = **3.6** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **3.6** kips	
Maximum reaction at support B	R_{B_max} = **18.3** kips	R_{B_min} = **14.1** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **5.8** kips	
Unfactored live load reaction at support B	R_{B_Live} = **3.6** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **3.6** kips	

Section details

Section type	**W 16x31 (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



Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio $b_f / (2 \times t_f)$ = **6.28**

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 $(d - 2 \times k) / t_w$ = **51.69**

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 $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **18.344** kips

Web area $A_w = d \times t_w$ = **4.373** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **131.175** kips

Resistance factor for shear ϕ_v = **1.00**

Design shear strength $V_c = \phi_v \times V_n$ = **131.175** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **130.699** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x =$ **225** kips_ft

Nominal flexural strength $M_n = M_{nyld} =$ **225.000** kips_ft

Design flexural strength $M_c = \phi_b \times M_n =$ **202.500** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240 =$ **1.425** in

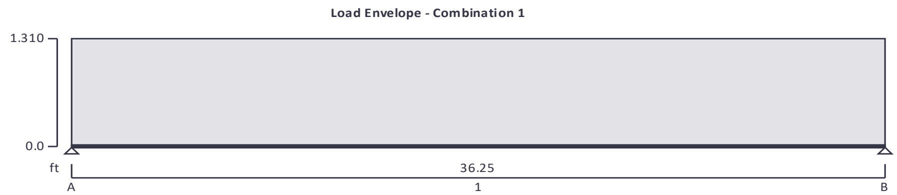
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **1.237** in

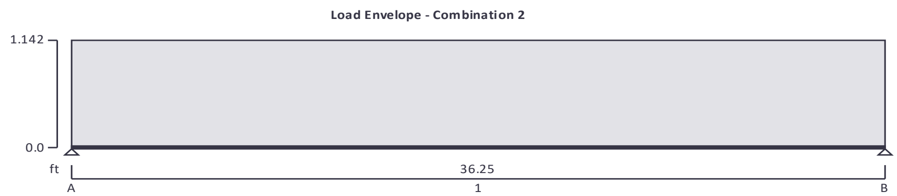
PASS - Maximum deflection does not exceed deflection limit

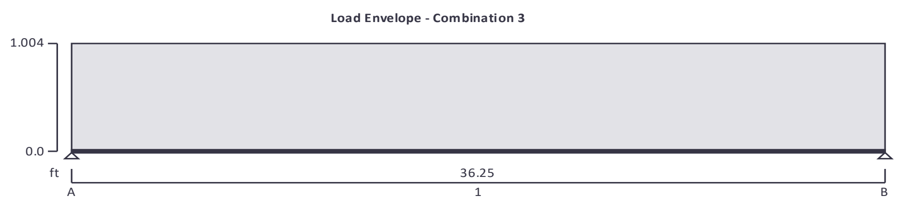
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

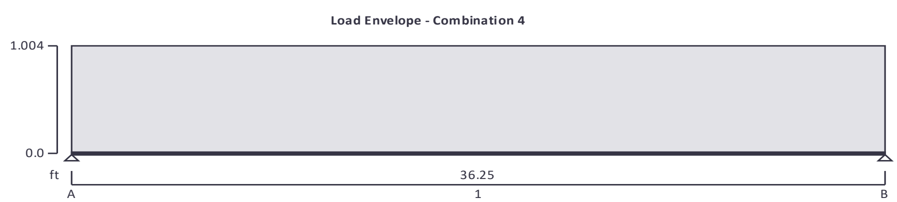
In accordance with AISC360-16 using the LRFD method

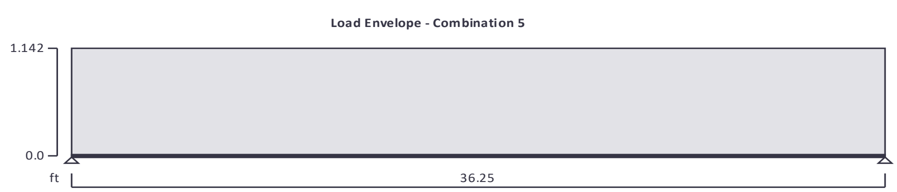
Tedds calculation version 3.0.14

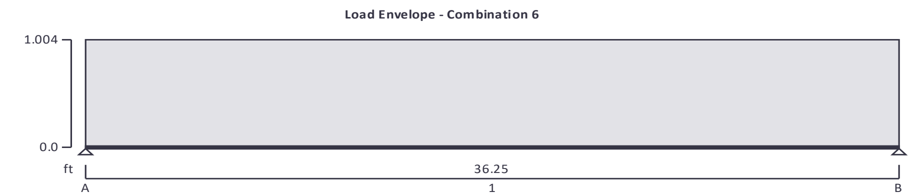


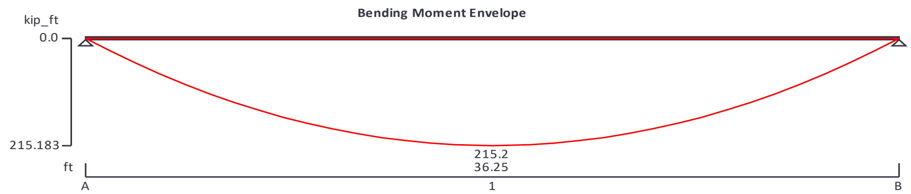


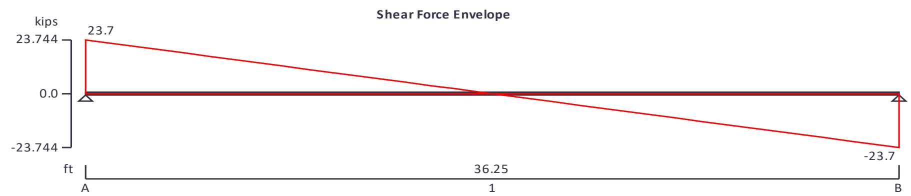












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 0.345 kips/ft
	Live full UDL 0.23 kips/ft
	Snow full UDL 0.161 kips/ft
	Snow full UDL 0.119 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60

		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00

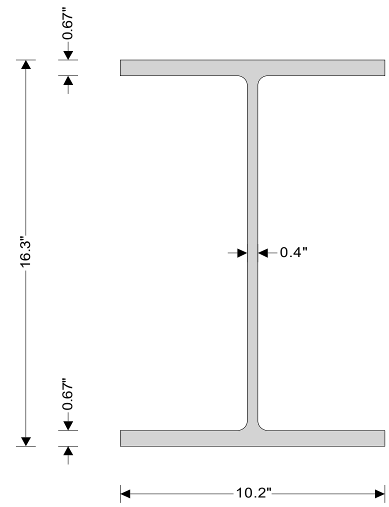
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **215.2** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **23.7** kips	V_{min} = **-23.7** kips
Deflection	δ_{max} = **1.3** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **23.7** kips	R_{A_min} = **18.2** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **7.5** kips	
Unfactored live load reaction at support A	R_{A_Live} = **4.2** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **5.1** kips	
Maximum reaction at support B	R_{B_max} = **23.7** kips	R_{B_min} = **18.2** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **7.5** kips	
Unfactored live load reaction at support B	R_{B_Live} = **4.2** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **5.1** kips	

Section details

Section type	**W 16x67 (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



Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

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 $b_f / (2 \times t_f)$ = **7.67**

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 $(d - 2 \times k) / t_w$ = **35.85**

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 $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **23.744** kips

Web area $A_w = d \times t_w$ = **6.439** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **193.155** kips

Resistance factor for shear ϕ_v = **1.00**

Design shear strength $V_c = \phi_v \times V_n$ = **193.155** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **215.183** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **541.667** kips_ft

Nominal flexural strength $M_n = M_{nyld}$ = **541.667** kips_ft

Design flexural strength $M_c = \phi_b \times M_n$ = **487.500** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240$ = **1.813** in

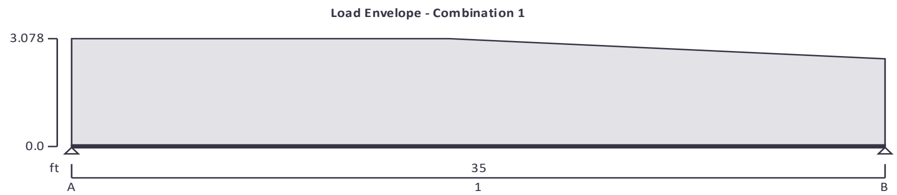
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min}))$ = **1.294** in

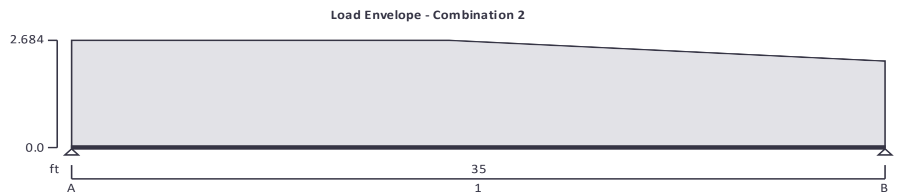
PASS - Maximum deflection does not exceed deflection limit

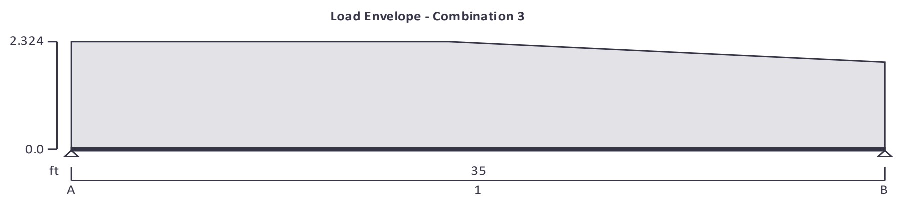
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

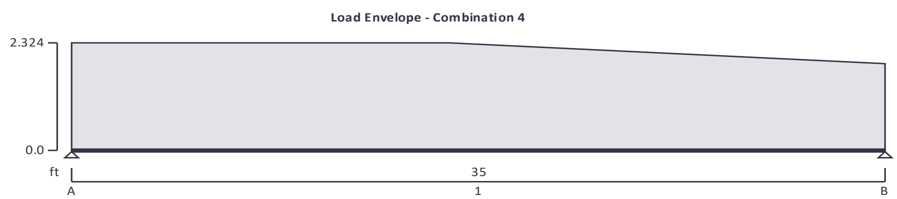
In accordance with AISC360-16 using the LRFD method

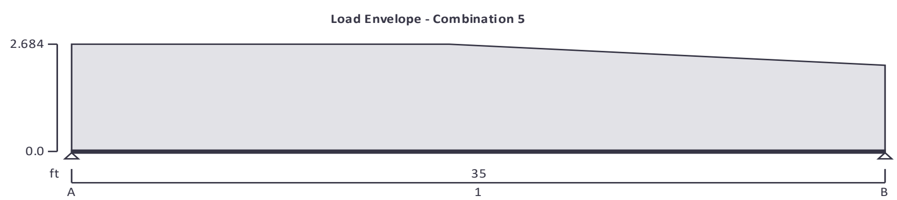
Tedds calculation version 3.0.14

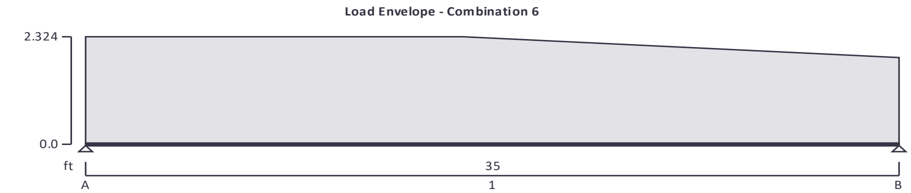


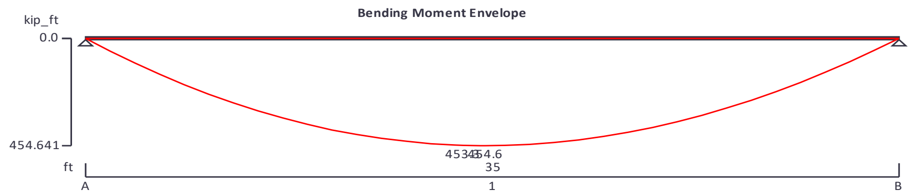


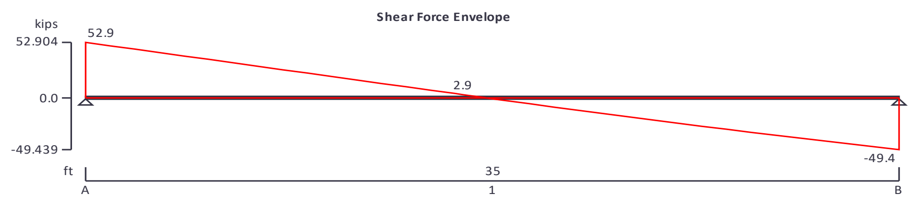












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead partial UDL 0.789 kips/ft from 0.00 in to 195.00 in
	Live partial UDL 0.6 kips/ft from 0.00 in to 195.00 in
	Snow partial UDL 0.42 kips/ft from 0.00 in to 195.00 in
	Snow full UDL 0.237 kips/ft
	Dead partial VDL 0.789 kips/ft at 195.00 in to 0.598 kips/ft at 420.00 in
	Live partial VDL 0.6 kips/ft at 195.00 in to 0.473 kips/ft at 420.00 in
	Snow partial VDL 0.42 kips/ft at 195.00 in to 0.331 kips/ft at 420.00 in

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60

		Wind $\times$ 1.60
		Snow $\times$ 1.60
	Support B	Dead $\times$ 1.20
		Live $\times$ 1.60
		Wind $\times$ 1.60
		Snow $\times$ 1.60
Load combination 2	Support A	Dead $\times$ 1.20
		Live $\times$ 1.60
		Wind $\times$ 1.00
		Snow $\times$ 1.00
		Dead $\times$ 1.20
		Live $\times$ 1.60
		Wind $\times$ 1.00
		Snow $\times$ 1.00
	Support B	Dead $\times$ 1.20
		Live $\times$ 1.60
		Wind $\times$ 1.00
		Snow $\times$ 1.00
Load combination 3	Support A	Dead $\times$ 1.20
		Live $\times$ 1.00
		Wind $\times$ 1.60
		Snow $\times$ 1.00
		Dead $\times$ 1.20
		Live $\times$ 1.00
		Wind $\times$ 1.60
		Snow $\times$ 1.00
	Support B	Dead $\times$ 1.20
		Live $\times$ 1.00
		Wind $\times$ 1.60
		Snow $\times$ 1.00
Load combination 4	Support A	Dead $\times$ 1.20
		Live $\times$ 1.00
		Wind $\times$ 1.00
		Snow $\times$ 1.00
		Dead $\times$ 1.20
		Live $\times$ 1.00
		Wind $\times$ 1.00
		Snow $\times$ 1.00
	Support B	Dead $\times$ 1.20
		Live $\times$ 1.00
		Wind $\times$ 1.00
		Snow $\times$ 1.00
Load combination 5	Support A	Dead $\times$ 1.20
		Live $\times$ 1.60
		Wind $\times$ 1.60

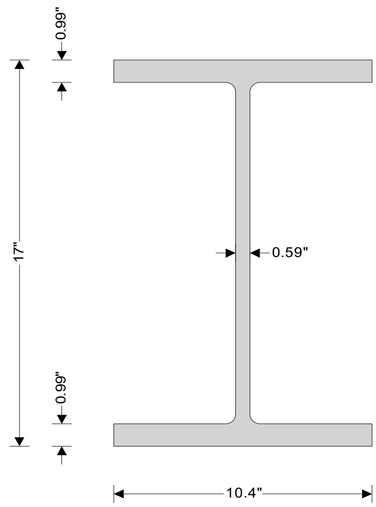
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **454.6** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **52.9** kips	V_{min} = **-49.4** kips
Deflection	δ_{max} = **1.2** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **52.9** kips	R_{A_min} = **39.9** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **15.2** kips	
Unfactored live load reaction at support A	R_{A_Live} = **10.3** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **11.3** kips	
Maximum reaction at support B	R_{B_max} = **49.4** kips	R_{B_min} = **37.2** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **14.1** kips	
Unfactored live load reaction at support B	R_{B_Live} = **9.5** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **10.8** kips	

Section details

Section type	**W 16x100 (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



Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

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

Width to thickness ratio $b_f / (2 \times t_f)$ = **5.28**

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 $(d - 2 \times k) / t_w$ = **24.31**

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 $V_r = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **52.904** kips

Web area $A_w = d \times t_w$ = **9.945** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **298.350** kips

Resistance factor for shear ϕ_v = **1.00**

Design shear strength $V_c = \phi_v \times V_n$ = **298.350** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength $M_r = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **454.641** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{nyld} = M_p = F_y \times Z_x$ = **825** kips_ft

Nominal flexural strength $M_n = M_{nyld}$ = **825.000** kips_ft

Design flexural strength $M_c = \phi_b \times M_n$ = **742.500** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240$ = **1.75** in

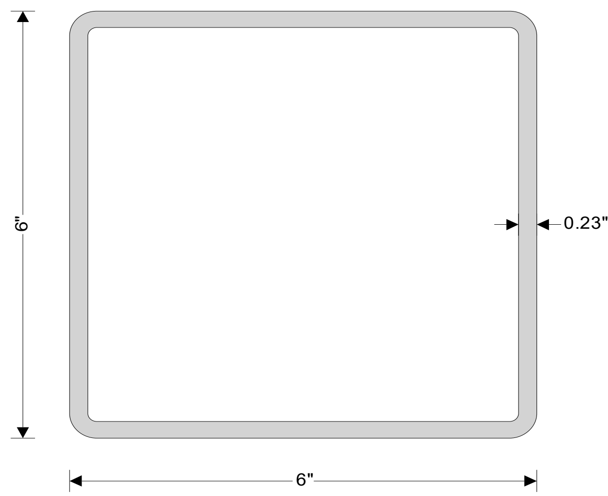
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **1.164** in

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN

In accordance with AISC360-16 and the LRFD 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 = **120** 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 = **156** in
For buckling about y axis	L_y = **156** in
For torsional buckling	L_z = **156** in

Effective length factors

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

Effective unbraced lengths

For buckling about x axis	$L_{cx} = L_x \times K_x$ = **156** in
For buckling about y axis	$L_{cy} = L_y \times K_y$ = **156** in
For torsional buckling	$L_{cz} = L_z \times K_z$ = **156** 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 $SR_x = L_{cx} / r_x =$ **66.7**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y =$ **66.7**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2 =$ **64.4** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **34.1** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A =$ **178.8** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **64.4** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y =$ **34.1** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A =$ **178.8** kips

Design compressive strength (cl.E1)

Resistance factor for compression $\phi_c =$ **0.90**

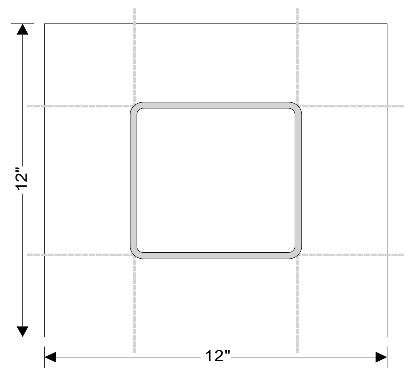
Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny}) =$ **160.9** kips

PASS - The design compressive strength exceeds the required compressive strength

COLUMN BASE PLATE DESIGN

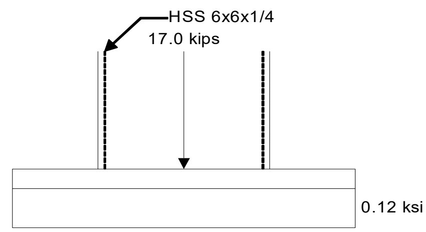
In accordance with AISC Steel Design Guide 1 and AISC 360-16

Tedds calculation version 2.1.02



Plan on baseplate

Flange/base weld - 0.3"
Web/base weld - 0.3"



Elevation on baseplate

Design forces and moments

Axial force	P_u = **17.0** 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 = **36.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **21.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **21.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **21.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **21.000** in
Minimum tensile strength, base plate	F_y = **36** ksi
Minimum tensile strength, column	F_{yCol} = **50** ksi
Compressive strength of concrete	f'_c = **3** ksi

Strength reduction factors

Compression	ϕ_c = **0.65**
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	$A_2 = (N + 2 \times l_{min}) \times (B + 2 \times l_{min}) =$ **1764.000** 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 — $t_{p,req} = l \times \sqrt{(2 \times P_u) / (\phi_b \times F_y \times B \times N)} =$ **0.269** 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 — $\phi_c P_p =$ **477.36** kips

PASS - Allowable bearing stress exceeds applied bearing stress

Flange weld

Flange weld leg length — $t_{wf} =$ **0.2500** in

Tension capacity of flange — $P_{tf} = b_f \times t \times F_{yCol} =$ **69.9** kips

Force in tension flange — $F_{tf} = M_u / (d - t) - P_u \times (b_f \times t) / A_{col} =$ **-4.5** 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 — $\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5}) =$ **47.250**ksi

Design strength of weld per in — $\phi R_{nf} = \phi F_{nw} \times t_{wf} / \sqrt{2} =$ **8.4** kips/in

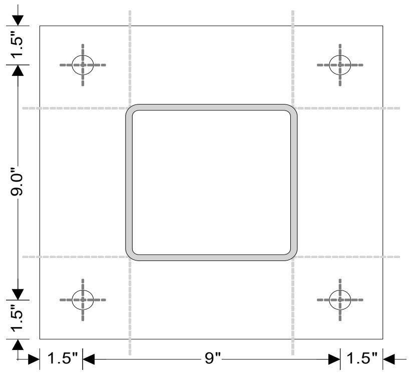
PASS - Available strength of flange weld exceeds force in flange weld

COLUMN BASE PLATE DESIGN

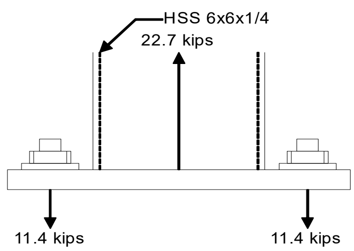
In accordance with AISC Steel Design Guide 1 and AISC 360-16

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 = **-22.7** 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 = **36.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **21.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **21.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **21.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **21.000** 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} = **50** ksi
Compressive strength of concrete	f'_c = **3** ksi

Strength reduction factors

Compression	ϕ_c = **0.65**

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

Bolt tension force

Tension force in one half of bolts	$T_u = ABS(P_u) / 2 =$ **11.4** kips
Max tension is single bolt	$T_{rod} = T_u / N_{bolty} =$ **5.7** 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	$t_{p,req} = 2.11 \times \sqrt{((T_u \times x)/(B \times F_y))} =$ **0.439** 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	$R_{wf} = T_{rod} / l_{Tweld,eff}$ = **2.8** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5})$ = **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_{nw} \times t_{wf} / \sqrt{(2)}$ = **8.4** kips/in

PASS - Available strength of weld exceeds force in tension weld

Local stress on flange	$f_{T,local} = (T_{rod}) / (l_{Tweld,eff} \times t_f)$ = **12.178** ksi
Column flange allowable stress	$F_{yCol} / 1.67$ = **29.940** ksi

PASS - Local column capacity exceeds local column stress

ANCHOR BOLT DESIGN

In accordance with ACI318-14

Tedds calculation version 2.1.02

Anchor bolt geometry

Type of anchor bolt	Cast-in headed end bolt 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
Number of threads per inch	n_t = **10**
Effective cross-sectional area of anchor	$A_{se} = \pi / 4 \times (d_a - 0.9743 \text{ in} / n_t)^2$ = **0.334** 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} = **58** ksi
Compressive strength of concrete	f'_c = **3** ksi
Concrete modification factor	λ = **1.00**

Modification factor for cast-in anchor concrete failure

$\lambda_a = 1.0 \times \lambda$= **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**

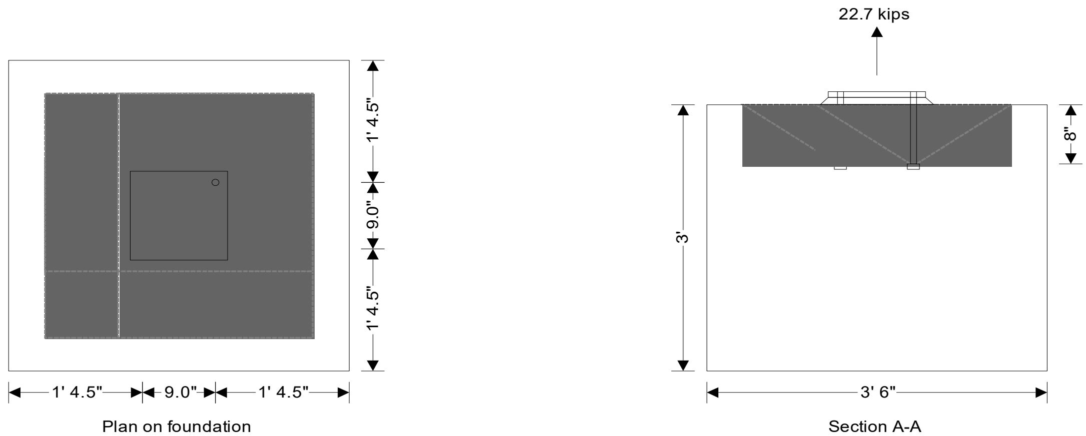
Steel strength of anchor in tension (17.4.1)

Nominal strength of anchor in tension $N_{sa} = A_{se} \times f_{uta}$ = **19.40** kips

Steel strength of anchor in tension $\phi N_{sa} = \phi_{t,s} \times N_{sa}$ = **14.55** kips

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

Check concrete breakout strength of anchor bolt in tension (17.4.2)



Concrete breakout - tension

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

Breakout strength for single anchor in tension $N_b = k_c \times \lambda_a \times \sqrt{(f'_c \times 1 \text{ psi})} \times h_{ef}^{1.5} \times 1 \text{ in}^{0.5}$ = **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 $c_{a,min}$ = **16.5** 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 (17.4.3)

Net bearing area of the head of anchor A_{brg} = **1.125** in^2

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

Pullout strength for single anchor $N_p = 8 \times A_{brg} \times f'_c$ = **27.00** kips

Nominal pullout strength of single anchor $N_{pn} = \psi_{c,P} \times N_p$ = **27.00** kips

Pullout strength of single anchor $\phi N_{pn} = \phi_{t,cB} \times N_{pn}$ = **18.90** kips

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

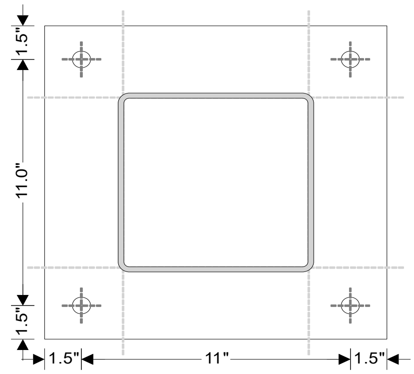
Side face blowout strength (17.4.4)

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

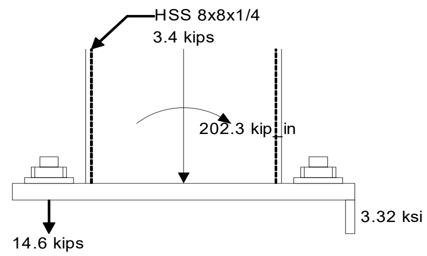
COLUMN BASE PLATE DESIGN

In accordance with AISC Steel Design Guide 1 and AISC 360-16

Tedds calculation version 2.1.02



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force	P_u = **3.4** kips (Compression)
Bending moment	M_u = **202.3** kip_in
Shear force	F_v = **0.0** kips
Eccentricity	e = ABS(M_u / P_u) = **59.326** in
Anchor bolt to center of plate	f = N/2 - e_1 = **5.500** in

Column details

Column section	HSS 8x8x1/4
Depth	d = **8.000** in
Breadth	b_f = **8.000** in
Thickness	t = **0.233** in

Baseplate details

Depth	N = **14.000** in
Breadth	B = **14.000** in
Thickness	t_p = **0.750** in
Design strength	F_y = **36.0** ksi

Foundation geometry

Member thickness	h_a = **36.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **33.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **33.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **33.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **33.000** 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} = **11.000** in
Edge distance	e_1 = **1.500** in
Minimum tensile strength, base plate	F_y = **36** ksi
Minimum tensile strength, column	F_{yCol} = **50** ksi
Compressive strength of concrete	f'_c = **3** ksi

Strength reduction factors

Compression	ϕ_c = **0.65**

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

Plate cantilever dimensions

Area of base plate	$A_1 = B \times N$ = **196.000** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min}) \times (B + 2 \times l_{min})$ = **4356.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **999.6** kips
Bending line cantilever distance m	$m = (N - 0.95 \times d) / 2$ = **3.200** in
Bending line cantilever distance n	$n = (B - 0.95 \times b_f) / 2$ = **3.200** in
Maximum bending line cantilever	$l = \max(m, n)$ = **3.200** in

Check eccentricity

Maximum bearing stress	$f_{p,max} = 0.85 \times f'_c \times \phi_c \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **3.32** ksi
Maximum bearing pressure	$q_{max} = f_{p,max} \times B$ = **46.41** kips/in
Critical eccentricity	$e_{crit} = N / 2 - P_u / (2 \times q_{max})$ = **6.963** in

e > e_{crit} so loads cannot be resisted by bearing alone. Therefore consider as a large moment

Plate dimensions adequate as $(f + N/2)^2 >= (2 \times P_u \times (e + f))/q_{max}$ and a real solution for bearing length exists

Bearing length - quadratic solution 1	$Y_1 = (f + N/2) + \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f))/q_{max})}$ = **24.613** in
Bearing length - quadratic solution 2	$Y_2 = (f + N/2) - \sqrt{((f + N/2)^2 - (2 \times P_u \times (e + f))/q_{max})}$ = **0.387** in
Bearing length	$Y = \min(Y_1, Y_2)$ = **0.387** in
Tension force in bolts	$T_u = q_{max} \times Y - P_u$ = **14.6** kips
Max tension in single bolt	$T_{rod} = T_u / (N_{bolt}/2)$ = **7.3** kips

Base plate yielding limit at bearing interface

Required plate thickness	$t_{p,req} = \sqrt{((4 \times f_{p,max} \times Y \times (l - Y/2))/(\phi_b \times F_y))}$ = **0.690** in

PASS - Thickness of plate exceeds required thickness

Base plate yielding limit at tension interface

Distance from bolt CL to plate bending lines	$x = \mathrm{abs}(m - e_1)$ = **1.700** in
Plate thickness required	$t_{p,req} = 2.11 \times \sqrt{((T_u \times x)/(B \times F_y))}$ = **0.467** 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	$R_{wf} = T_{rod} / l_{Tweld,eff}$ = **3.6** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5})$ = **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_{nw} \times t_{wf} / \sqrt{(2)}$ = **8.4** kips/in

PASS - Available strength of weld exceeds force in tension weld

Local stress on flange	$f_{T,local} = (T_{rod}) / (l_{Tweld,eff} \times t_f)$ = **15.614** ksi
Column flange allowable stress	$F_{yCol} / 1.67$ = **29.940** ksi

PASS - Local column capacity exceeds local column stress

ANCHOR BOLT DESIGN

In accordance with ACI318-14

Tedds calculation version 2.1.02

Anchor bolt geometry

Type of anchor bolt	Cast-in headed end bolt 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 $n_{tens} = (N_{boltN} \times 2) + (N_{bolty} - 2) =$ **2**
Spacing of bolts in x direction $s_{boltx} =$ **11** in
Spacing of bolts in y direction $s_{bolty} =$ **11** in
Number of threads per inch $n_t =$ **10**
Effective cross-sectional area of anchor $A_{se} = \pi / 4 \times (d_a - 0.9743 \text{ in} / n_t)^2 =$ **0.334** 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 $f_{uta} =$ **58** ksi
Compressive strength of concrete $f'_c =$ **3** ksi
Concrete modification factor $\lambda =$ **1.00**
Modification factor for cast-in anchor concrete failure
$\lambda_a = 1.0 \times \lambda =$ **1.00**

Strength reduction factors

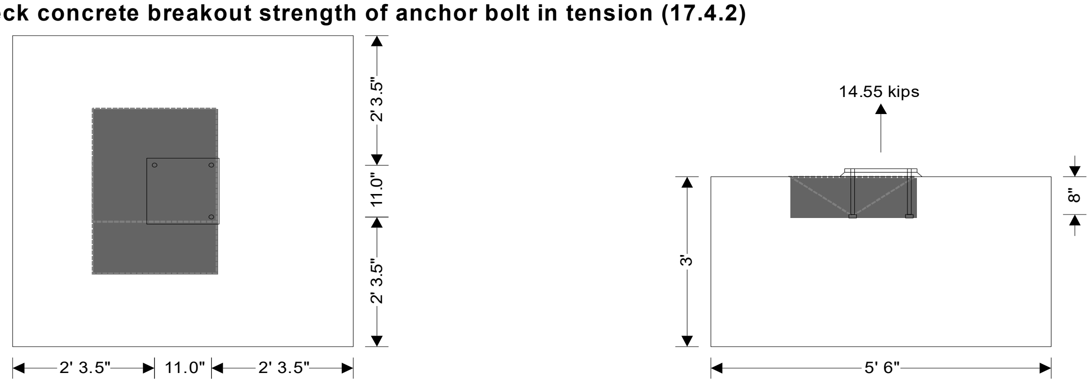
Tension of steel element $\phi_{t,s} =$ **0.75**
Shear of steel element $\phi_{v,s} =$ **0.65**
Concrete tension $\phi_{t,c} =$ **0.65**
Concrete shear $\phi_{v,c} =$ **0.70**
Concrete tension for pullout $\phi_{t,cB} =$ **0.70**
Concrete shear for pryout $\phi_{v,cB} =$ **0.70**

Steel strength of anchor in tension (17.4.1)

Nominal strength of anchor in tension $N_{sa} = A_{se} \times f_{uta} =$ **19.40** kips
Steel strength of anchor in tension $\phi N_{sa} = \phi_{t,s} \times N_{sa} =$ **14.55** kips

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

Check concrete breakout strength of anchor bolt in tension (17.4.2)



Plan on foundation
Section A-A
Concrete breakout - tension

Coeff for basic breakout strength in tension $k_c =$ **24**
Breakout strength for single anchor in tension $N_b = k_c \times \lambda_a \times \sqrt{(f'_c \times 1 \text{ psi})} \times h_{ef}^{1.5} \times 1 \text{ in}^{0.5} =$ **29.74** kips
Projected area for groups of anchors $A_{Nc} =$ **840** 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 $c_{a,min} =$ **27.5** 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 $N_{cbg} = A_{Nc} / A_{Nco} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$ = **43.38** kips

Concrete breakout strength $\phi N_{cbg} = \phi_{t,c} \times N_{cbg}$ = **28.20** kips

PASS - Breakout strength exceeds tension in bolts

Pullout strength (17.4.3)

Net bearing area of the head of anchor A_{brg} = **1.125** in^2

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

Pullout strength for single anchor $N_p = 8 \times A_{brg} \times f'_c$ = **27.00** kips

Nominal pullout strength of single anchor $N_{pn} = \psi_{c,P} \times N_p$ = **27.00** kips

Pullout strength of single anchor $\phi N_{pn} = \phi_{t,cB} \times N_{pn}$ = **18.90** kips

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

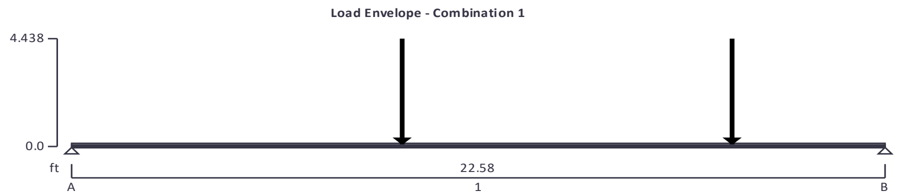
Side face blowout strength (17.4.4)

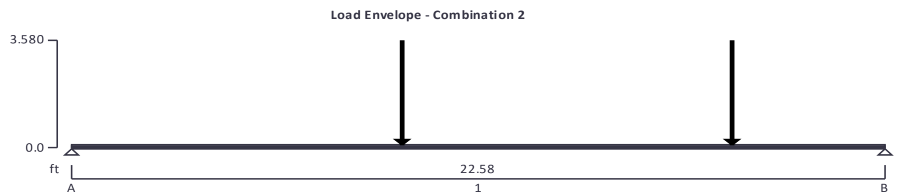
As $h_{ef} <= 2.5 \times min(c_{a1}, c_{a2})$ the edge distance is considered to be far from an edge and blowout strength need not be considered

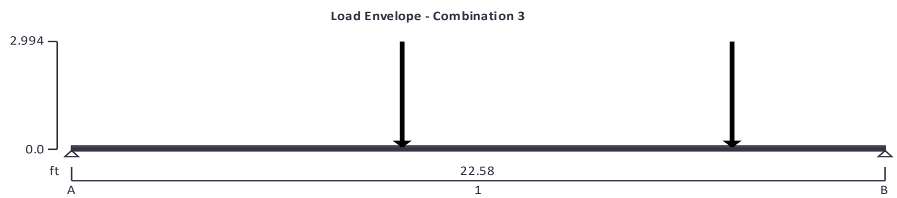
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

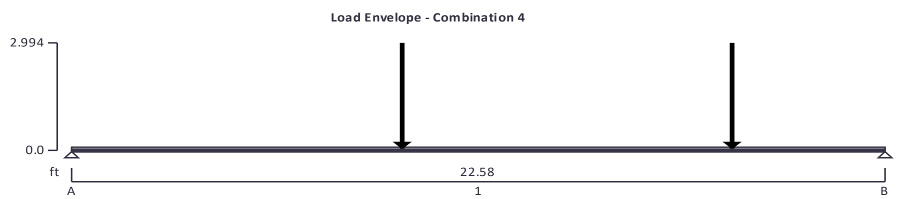
In accordance with AISC360-16 using the LRFD method

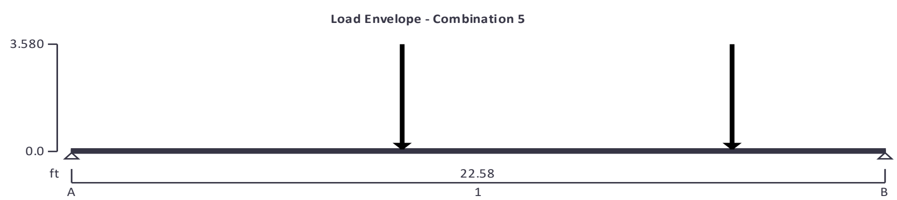
Tedds calculation version 3.0.14

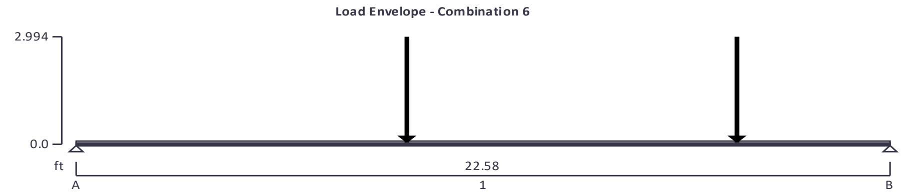


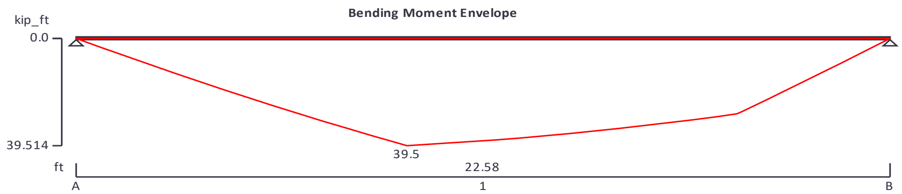


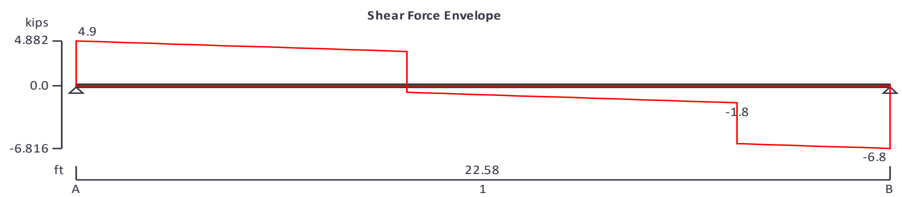












Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead point load 0.489 kips at 110.04 in
	Dead point load 0.489 kips at 219.96 in
	Live point load 0.977 kips at 110.04 in
	Live point load 0.977 kips at 219.96 in
	Snow point load 1.43 kips at 110.04 in
	Snow point load 1.43 kips at 219.96 in
	Snow full UDL 0.06 kips/ft

Load combinations

Load combination 1	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
		Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60

	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.60
Load combination 2	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.00
		Snow × 1.00
Load combination 3	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
Load combination 4	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.00
		Snow × 1.00
Load combination 5	Support A	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20

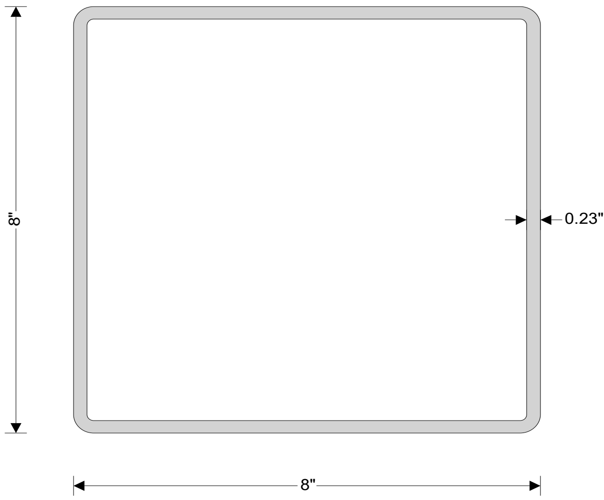
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.60
		Wind × 1.60
		Snow × 1.00
Load combination 6	Support A	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
		Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00
	Support B	Dead × 1.20
		Live × 1.00
		Wind × 1.60
		Snow × 1.00

Analysis results

Maximum moment	M_{max} = **39.5** kips_ft	M_{min} = **0** kips_ft
Maximum shear	V_{max} = **4.9** kips	V_{min} = **-6.8** kips
Deflection	δ_{max} = **1.1** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **4.9** kips	R_{A_min} = **3.3** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **0.7** kips	
Unfactored live load reaction at support A	R_{A_Live} = **0.8** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **1.8** kips	
Maximum reaction at support B	R_{B_max} = **6.8** kips	R_{B_min} = **4.7** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **0.9** kips	
Unfactored live load reaction at support B	R_{B_Live} = **1.2** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **2.4** kips	

Section details

Section type	**HSS 8x8x1/4 (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



Resistance factors

Resistance factor for tensile yielding	ϕ_{ty} = **0.90**
Resistance factor for tensile rupture	ϕ_{tr} = **0.75**
Resistance factor for compression	ϕ_c = **0.90**
Resistance factor for flexure	ϕ_b = **0.90**

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	$(b_f - 3 \times t) / t$ = **31.33**	
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**	Nonslender

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

Width to thickness ratio	$(d - 3 \times t) / t$ = **31.33**	
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 nonslender in flexure

Design of members for shear - Chapter G

Required shear strength	$V_r = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **6.816** kips
Web area	$A_w = 2 \times (d - 3 \times t) \times t$ = **3.402** in^2
Web plate buckling coefficient	k_v = **5**
Web shear coefficient - eq G2-9	C_{v2} = **1**
Nominal shear strength – eq G6-1	$V_n = 0.6 \times F_y \times A_w \times C_{v2}$ = **93.903** kips
Resistance factor for shear	ϕ_v = **0.90**
Design shear strength	$V_c = \phi_v \times V_n$ = **84.512** kips

PASS - Design shear strength exceeds required shear strength

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

Required flexural strength	$M_r = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min}))$ = **39.514** kips_ft

Yielding - Section F7.1

Nominal flexural strength for yielding - eq F7-1 $M_{nyld} = M_p = F_y \times Z_x =$ **78.583** kips_ft

Compression flange local buckling - Section F7.2

Clear width inside radii $b = b_f - 3 \times t =$ **7.301** in

Nominal flexural strength for compression flange local buckling - eq F7-2

$M_{ncfb} = [M_p - (M_p - F_y \times S_x) \times (3.57 \times b / t \times \sqrt{[F_y / E]} - 4.0)] =$ **73.697** kips_ft

Nominal flexural strength $M_n = \min(M_{nyld}, M_{nltb}, M_{ncfb}, M_{nwb}) =$ **73.697** kips_ft

Design flexural strength $M_c = \phi_b \times M_n =$ **66.327** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live and snow loads

Limiting deflection $\delta_{lim} = L_{s1} / 240 =$ **1.129** in

Maximum deflection span 1 $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min})) =$ **1.111** in

PASS - Maximum deflection does not exceed deflection limit

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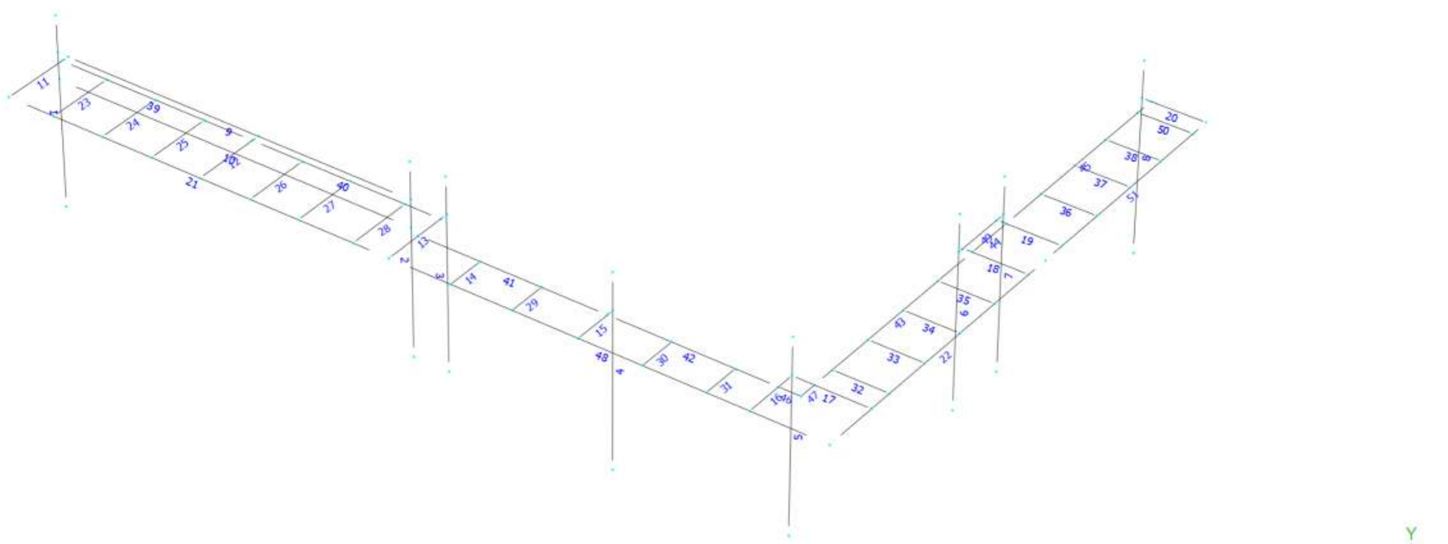

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Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

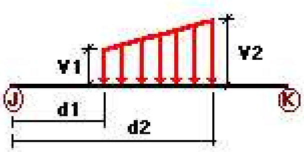
Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
LL	Live Load	No	LL
W	Wind	No	WIND

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
LL	2	0.00	-0.20	0.00	0.00	0.00	0.00
	21	0.00	-0.20	0.00	0.00	0.00	0.00
	36	0.00	-0.20	0.00	0.00	0.00	0.00
	55	0.00	-0.20	0.00	0.00	0.00	0.00

Distributed force on members



Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
DL	17	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	18	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	19	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	20	Y	-0.001	-0.001	0.00	Yes	100.00	Yes
	21	Y	-0.002	-0.002	0.00	Yes	100.00	Yes
	32	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	33	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	34	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	35	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	36	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	37	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	38	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
	39	Y	-0.002	-0.002	0.00	Yes	100.00	Yes

	40	Y	-0.002	-0.002	0.00	Yes	100.00	Yes
	41	Y	-0.0011	-0.0011	0.00	Yes	100.00	Yes
	42	Y	-0.0011	-0.0011	0.00	Yes	100.00	Yes
	46	Y	-0.0011	-0.0011	0.00	Yes	100.00	Yes
	48	Y	-0.0011	-0.0011	0.00	Yes	100.00	Yes
	50	Y	-0.0027	-0.0027	0.00	Yes	100.00	Yes
W	11	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	12	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	13	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	14	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	15	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	16	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	17	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	18	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	19	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	20	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	21	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.008	-0.008	0.00	Yes	100.00	Yes
	22	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	23	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	24	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	25	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	26	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	27	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	28	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	29	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	30	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	31	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	32	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	33	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	34	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	35	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	36	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	37	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	38	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
		Y	-0.012	-0.012	0.00	Yes	100.00	Yes
	39	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.008	-0.008	0.00	Yes	100.00	Yes
	40	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.008	-0.008	0.00	Yes	100.00	Yes
	41	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.0047	-0.0047	0.00	Yes	100.00	Yes
	42	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.0047	-0.0047	0.00	Yes	100.00	Yes
	43	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	44	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	45	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	46	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
		Y	-0.0047	-0.0047	0.00	Yes	100.00	Yes
	47	Y	-0.006	-0.006	0.00	Yes	100.00	Yes

48	Y	-0.006	-0.006	0.00	Yes	100.00	Yes
	Y	-0.0047	-0.0047	0.00	Yes	100.00	Yes
50	Y	-0.0077	-0.0077	0.00	Yes	100.00	Yes
	Y	-0.012	-0.012	0.00	Yes	100.00	Yes
51	Y	-0.006	-0.006	0.00	Yes	100.00	Yes

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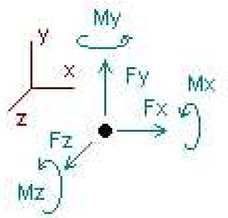
Analysis result

Reactions



Direction of positive forces and moments

	Forces [Kip]			Moments [Kip*ft]		
Node	**FX**	**FY**	**FZ**	**MX**	**MY**	**MZ**
Condition **DL=Dead Load**						
23	0.03364	1.68237	0.09612	0.00000	0.00000	0.00000
24	-0.03367	1.42825	0.05153	0.00000	0.00000	0.00000
25	-0.00549	0.84260	0.14043	0.00000	0.00000	0.00000
26	0.02560	0.71753	0.05804	0.00000	0.00000	0.00000
27	0.05695	0.98077	0.03409	0.00000	0.00000	0.00000
28	0.06753	0.78824	0.00030	0.00000	0.00000	0.00000
29	0.06414	0.62178	0.00588	0.00000	0.00000	0.00000
30	0.04843	0.56637	0.00393	0.00000	0.00000	0.00000
64	-0.53461	0.00000	-0.09150	0.00000	0.00000	0.00000
65	-0.02265	0.00000	-0.14334	0.00000	0.00000	0.00000
66	0.59684	0.00000	-0.05464	0.00000	0.00000	0.00000
67	0.10792	0.00000	-0.05647	0.00000	0.00000	0.00000
68	-0.22288	0.00000	-0.18098	0.00000	0.00000	0.00000
69	-0.08433	0.00000	0.10517	0.00000	0.00000	0.00000
70	-0.04885	0.00000	0.01520	0.00000	0.00000	0.00000
71	-0.04856	0.00000	0.01625	0.00000	0.00000	0.00000
SUM	0.00000	7.62791	0.00000	0.00000	0.00000	0.00000
Condition **LL=Live Load**						
23	0.00000	0.21127	0.08044	0.00000	0.00000	0.00000
24	0.00115	0.12086	0.04966	0.00000	0.00000	0.00000
25	-0.00374	0.09783	0.03832	0.00000	0.00000	0.00000
26	0.02164	-0.07190	-0.01668	0.00000	0.00000	0.00000
27	0.05257	0.34480	0.01963	0.00000	0.00000	0.00000
28	0.02296	0.13536	0.00199	0.00000	0.00000	0.00000
29	0.01185	-0.02651	0.00250	0.00000	0.00000	0.00000
30	-0.00374	-0.01172	0.00230	0.00000	0.00000	0.00000
64	-0.07983	0.00000	-0.07689	0.00000	0.00000	0.00000
65	-0.01536	0.00000	-0.03928	0.00000	0.00000	0.00000
66	0.12567	0.00000	-0.05308	0.00000	0.00000	0.00000
67	0.09105	0.00000	0.01833	0.00000	0.00000	0.00000
68	-0.19234	0.00000	-0.13034	0.00000	0.00000	0.00000
69	-0.03749	0.00000	0.05086	0.00000	0.00000	0.00000
70	0.00196	0.00000	0.04251	0.00000	0.00000	0.00000
71	0.00367	0.00000	0.00972	0.00000	0.00000	0.00000
SUM	0.00000	0.80000	0.00000	0.00000	0.00000	0.00000

Condition **W=Wind**

23	-0.00143	0.46675	0.09422	0.00000	0.00000	0.00000
24	0.00120	0.26826	0.05526	0.00000	0.00000	0.00000
25	-0.00431	0.46969	0.12008	0.00000	0.00000	0.00000
26	0.02146	0.32513	0.04641	0.00000	0.00000	0.00000
27	0.05006	0.56272	0.02425	0.00000	0.00000	0.00000
28	0.06264	0.39522	0.00066	0.00000	0.00000	0.00000
29	0.05939	0.26078	0.00427	0.00000	0.00000	0.00000
30	0.05151	0.30586	0.00300	0.00000	0.00000	0.00000
64	-0.20526	0.00000	-0.09046	0.00000	0.00000	0.00000
65	-0.01770	0.00000	-0.12225	0.00000	0.00000	0.00000
66	0.25600	0.00000	-0.05793	0.00000	0.00000	0.00000
67	0.09046	0.00000	-0.04530	0.00000	0.00000	0.00000
68	-0.18919	0.00000	-0.14078	0.00000	0.00000	0.00000
69	-0.07673	0.00000	0.07722	0.00000	0.00000	0.00000
70	-0.04647	0.00000	0.01904	0.00000	0.00000	0.00000
71	-0.05163	0.00000	0.01231	0.00000	0.00000	0.00000
SUM	0.00000	3.05440	0.00000	0.00000	0.00000	0.00000

Bentley

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Steel Code Check

Report: Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+1.6LL
D3=1.2DL+0.5W
D4=1.2DL+W
D5=1.2DL+W+LL
D6=0.9DL+W

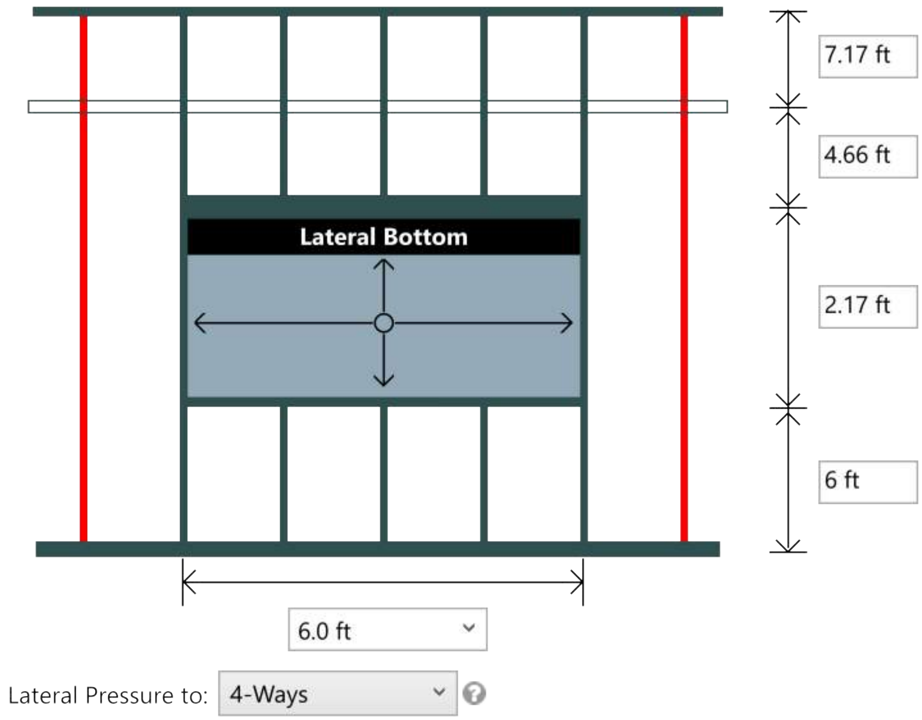
Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
1	*HSS_SQR 6X6X1_4*	1	D5 at 79.17%	**0.15**	**OK**	
10	*HSS_RECT 12X4X3_8*	10	D1 at 0.00%	**0.02**	**OK**	
11	*C 8X11.5*	11	D5 at 0.00%	0.14	OK	
12		12	D5 at 100.00%	0.11	OK	
13		13	D5 at 0.00%	**0.16**	**OK**	
14	*WT 4X5*	14	D5 at 50.00%	**0.00**	**OK**	
15	*C 8X11.5*	15	D5 at 84.38%	0.06	OK	
16		16	D5 at 100.00%	0.09	OK	
17		17	D5 at 0.00%	**0.12**	**OK**	
18		18	D5 at 100.00%	0.09	OK	
19		19	D5 at 100.00%	0.07	OK	
2	*HSS_SQR 6X6X1_4*	2	D5 at 79.17%	**0.11**	**OK**	
20	*C 8X11.5*	20	D4 at 100.00%	0.05	OK	
21		21	D5 at 100.00%	**0.16**	**OK**	
22		22	D5 at 91.96%	0.07	With warnings	
23	*WT 4X5*	23	D5 at 50.00%	**0.01**	**OK**	
24		24	D5 at 50.00%	0.01	OK	
25		25	D4 at 50.00%	0.01	OK	
26		26	D4 at 50.00%	0.01	OK	
27		27	D5 at 50.00%	0.01	OK	
28		28	D5 at 50.00%	0.01	OK	
29		29	D5 at 50.00%	0.00	OK	
3	*HSS_SQR 6X6X1_4*	3	D5 at 78.13%	**0.09**	**OK**	
30	*WT 4X5*	30	D5 at 50.00%	0.00	OK	
31		31	D4 at 50.00%	0.00	OK	
32		32	D5 at 50.00%	**0.01**	**OK**	
33		33	D5 at 50.00%	0.01	OK	
34		34	D5 at 50.00%	0.01	OK	
35		35	D5 at 50.00%	0.01	OK	
36		36	D5 at 50.00%	0.01	OK	
37		37	D5 at 50.00%	0.01	OK	
38		38	D5 at 50.00%	0.01	OK	
39	*C 12X20.7*	39	D5 at 40.63%	**0.02**	**OK**	

4	***HSS_SQR 6X6X1_4***	**4**	D5 at 78.13%	**0.05**	**OK**
40	***C 12X20.7***	**40**	D5 at 53.13%	0.02	OK
41		**41**	D5 at 39.58%	0.01	OK
42		**42**	D5 at 45.83%	0.01	OK
43		**43**	D5 at 47.50%	0.02	OK
44		**44**	D5 at 0.00%	0.00	OK
45		**45**	D4 at 47.50%	0.02	OK
46		**46**	D5 at 100.00%	**0.03**	**OK**
47		**47**	D5 at 100.00%	0.03	OK
48	***C 8X11.5***	**48**	D5 at 0.00%	**0.14**	**With warnings**
49	***HSS_RECT 8X4X3_8***	**49**	D5 at 0.00%	**0.01**	**OK**
5	***HSS_SQR 6X6X1_4***	**5**	D5 at 78.13%	**0.07**	**OK**
50	***WT 4X5***	**50**	D4 at 50.00%	**0.01**	**OK**
51	***C 8X11.5***	**51**	D5 at 0.00%	**0.06**	**With warnings**
6	***HSS_SQR 6X6X1_4***	**6**	D5 at 78.13%	**0.05**	**OK**
7		**7**	D5 at 78.13%	0.04	OK
8		**8**	D4 at 78.13%	0.03	OK
9	***HSS_RECT 12X4X3_8***	**9**	D5 at 58.33%	**0.05**	**OK**

Summary

The wall framing system of the project consists of cold-formed steel wall studs. Headers are provided at each opening and span to jamb studs that transfer vertical and lateral loads into the floor slab. Lateral loads are distributed by gypsum sheathed walls.

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



Design Loads

Wall Lateral Pressure :	**17.8 psf**
RO Lateral Pressure :	**4-Ways**
Parapet Lateral Pressure :	**32.5 psf**

Lateral Element Forces multiplied by 1 for strength checks

Lateral Forces multiplied by 0.7 for deflection determination

Header:	**Box (lateral top, bottom)**
Gravity Load at Header:	**12 psf**

+ Additional load from each cripple of 133 lbs at 16 in o.c.

Brace Settings

Component(s)	Members(s)	Flexural Bracing (in)	Axial KyLy (in)	Axial KtLt (in)	Distortional K-Phi(lb-in/in)	Distortional LM(in)	Interconnection Spacing(in)
Wall Studs	600S162-54(50), Single@16 in o/c	60 in	None	None	0	None	N/A
Jamb Studs	600S162-54(50), Boxed	60 in	None	None	0	None	12 in
Vertical Header	800S200-54(50), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Sill	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Summary Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max. Moment (Ft-Lb)	Max. Shear(lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	600S162-54(50), Single@16 in o/c	133.0	1113.9	310.7	65.4	549.8
Jamb Studs	600S162-54(50), Boxed	725.1	708.1	305.7	19.7	150.6
Vertical Header	800S200-54(50), Boxed	N/A	1087.7	725.1	N/A	725.1
Lat. Top Head	600T125-54(50), Single	N/A	620.1	413.4	N/A	413.4
Lat. Bottom Head	600T125-54(50), Single	N/A	83.1	47.5	N/A	47.5
Sill	600T125-54(50), Single	N/A	323.4	207.7	N/A	207.7

Summary Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	Bending +Axial Interaction	Shear Interaction	Web Stiffners	Design OK
Wall Studs	600S162-54(50), Single@16 in o/c	L/2615	L/358	0.768	0.45	NA	Yes
Jamb Studs	600S162-54(50), Boxed	L/3132	L/873	0.235	0.16	NA	Yes
Vertical Header	800S200-54(50), Boxed	L/3961	NA	0.15	0.17	No	Yes
Lat. Top Head	600T125-54(50), Single	L/1692	NA	0.42	0.15	No	Yes
Lat. Bottom Head	600T125-54(50), Single	L/12731	NA	0.06	0.02	No	Yes
Sill	600T125-54(50), Single	L/3251	NA	0.22	0.08	No	Yes

Simpson Strong-Tie® Connectors @ Studs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	549.77	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	72.34 %	49.31 %
R1	65.43	453.00	600T125-54 (50) & (1) 1/2" x 4" Titen HD to 2500 psi min concrete	7.03 %	8.90 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	150.55	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	19.81 %	13.50 %
R1	19.74	950.49	600T125-54 (50) & (1) 1/2" x 4" Titen HD to 2500 psi min concrete	1.06 %	2.69 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

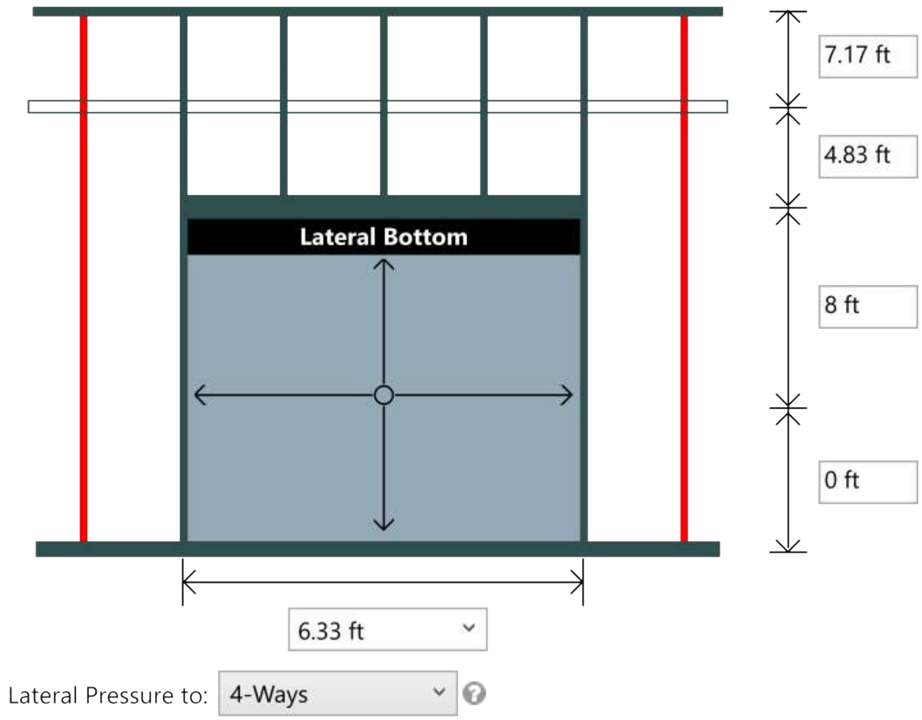
Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Top CantiLever	Span	N/A	0.00	N/A	N/A	N/A	N/A	N/A	N/A
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Top CantiLever	Span	N/A	0.00	N/A	N/A	N/A	N/A	N/A	N/A
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back,box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.



Design Loads

Wall Lateral Pressure :	**17.8 psf**
RO Lateral Pressure :	**4-Ways**
Parapet Lateral Pressure :	**32.5 psf**
Lateral Element Forces multiplied by 1 for strength checks	
Lateral Forces multiplied by 0.7 for deflection determination	
Header:	**Box (lateral top, bottom)**
Gravity Load at Header:	**12 psf**

+ Additional load from each cripple of 133 lbs at 16 in o.c.

Brace Settings

Component(s)	Members(s)	Flexural Bracing (in)	Axial KyLy (in)	Axial KtLt (in)	Distortional K-Phi(lb-in/in)	Distortional LM(in)	Interconnection Spacing(in)
Wall Studs	600S162-54(50), Single@16 in o/c	60 in	None	None	0	None	N/A
Jamb Studs	600S162-54(50), Boxed	60 in	None	None	0	None	12 in
Vertical Header	800S200-54(50), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Summary Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max. Moment (Ft-Lb)	Max. Shear(lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	600S162-54(50), Single@16 in o/c	133.0	1113.9	310.7	65.4	549.8
Jamb Studs	600S162-54(50), Boxed	771.5	678.2	268.4	188.0	158.9
Vertical Header	800S200-54(50), Boxed	N/A	1220.8	771.5	N/A	771.5
Lat. Top Head	600T125-54(50), Single	N/A	651.0	411.4	N/A	411.4
Lat. Bottom Head	600T125-54(50), Single	N/A	188.1	89.2	N/A	89.2

Summary Design Results

		Deflection					
Component(s)	Members(s)	Span	Parapet	Bending +Axial Interaction	Shear Interaction	Web Stiffners	Design OK

Wall Studs	600S162-54(50), Single@16 in o/c	L/2615	L/358	0.768	0.45	NA	Yes
Jamb Studs	600S162-54(50), Boxed	L/3324	L/900	0.235	0.15	NA	Yes
Vertical Header	800S200-54(50), Boxed	L/3345	NA	0.16	0.18	No	Yes
Lat. Top Head	600T125-54(50), Single	L/1528	NA	0.44	0.15	No	Yes
Lat. Bottom Head	600T125-54(50), Single	L/5507	NA	0.13	0.03	No	Yes

Simpson Strong-Tie® Connectors @ Studs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	549.77	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	72.34 %	49.31 %
R1	65.43	453.00	600T125-54 (50) & (1) 1/2" x 4" Titen HD to 2500 psi min concrete	7.03 %	8.90 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	158.90	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	20.91 %	14.25 %
R1	188.04	995.47	600T125-54 (50) & (1) 1/2" x 4" Titen HD to 2500 psi min concrete	13.47 %	25.58 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

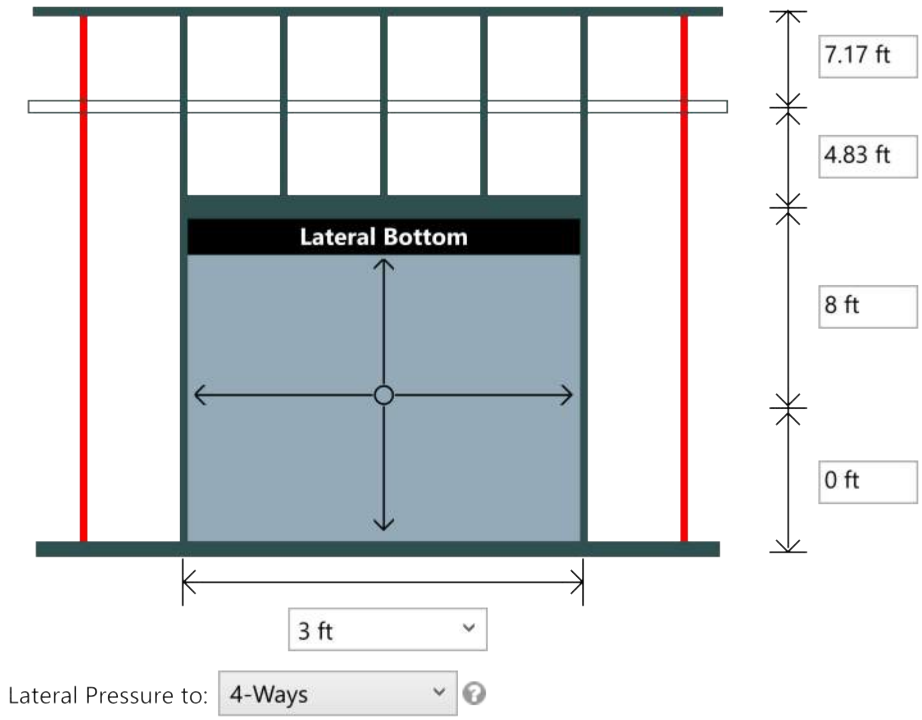
Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Top CantiLever	Span	N/A	0.00	N/A	N/A	N/A	N/A	N/A	N/A
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)[1]	LSUBH (Max)[1]	SUBH (Min)[1]	SUBH (Max)[1]	MSUBH (Min)[1]	MSUBH (Max)[1]
Top CantiLever	Span	N/A	0.00	N/A	N/A	N/A	N/A	N/A	N/A
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back,box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.



Design Loads

Wall Lateral Pressure :	**17.8 psf**
RO Lateral Pressure :	**4-Ways**
Parapet Lateral Pressure :	**32.5 psf**

Lateral Element Forces multiplied by 1 for strength checks

Lateral Forces multiplied by 0.7 for deflection determination

Header:	**Box (lateral top, bottom)**
Gravity Load at Header:	**12 psf**

+ Additional load from each cripple of 133 lbs at 16 in o.c.

Brace Settings

Component(s)	Members(s)	Flexural Bracing (in)	Axial KyLy (in)	Axial KtLt (in)	Distortional K-Phi(lb-in/in)	Distortional LM(in)	Interconnection Spacing(in)
Wall Studs	600S162-54(50), Single@16 in o/c	60 in	None	None	0	None	N/A
Jamb Studs	600S162-54(50), Single	60 in	None	None	0	None	N/A
Vertical Header	600S162-54(50), Boxed	Full	N/A	N/A	0	None	N/A
Lat. Top Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A
Lat. Bottom Head	600T125-54(50), Single	Full	N/A	N/A	0	None	N/A

Summary Analysis Results

Component(s)	Members(s)	Axial Load (lb)	Max. Moment (Ft-Lb)	Max. Shear(lb)	Bottom Reaction (lb)	Top or End Reaction (lb)
Wall Studs	600S162-54(50), Single@16 in o/c	133.0	1113.9	310.7	65.4	549.8
Jamb Studs	600S162-54(50), Single	365.6	556.9	182.1	106.3	219.9
Vertical Header	600S162-54(50), Boxed	N/A	274.2	365.6	N/A	365.6
Lat. Top Head	600T125-54(50), Single	N/A	146.2	195.0	N/A	195.0
Lat. Bottom Head	600T125-54(50), Single	N/A	20.0	20.0	N/A	20.0

Summary Design Results

Component(s)	Members(s)	Deflection Span	Deflection Parapet	Bending +Axial Interaction	Shear Interaction	Web Stiffners	Design OK

Wall Studs	600S162-54(50), Single@16 in o/c	L/2615	L/358	0.768	0.45	NA	Yes
Jamb Studs	600S162-54(50), Single	L/2809	L/572	0.717	0.23	NA	Yes
Vertical Header	600S162-54(50), Boxed	L/13676	NA	0.05	0.06	No	Yes
Lat. Top Head	600T125-54(50), Single	L/14352	NA	0.10	0.07	No	Yes
Lat. Bottom Head	600T125-54(50), Single	L/109165	NA	0.01	0.01	No	Yes

Simpson Strong-Tie® Connectors @ Studs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	549.77	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	72.34 %	49.31 %
R1	65.43	453.00	600T125-54 (50) & (1) 1/2" x 4" Titen HD to 2500 psi min concrete	7.03 %	8.90 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Connectors @ Jambs

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie® Connector	Connector Interaction	Anchor Interaction
R2	219.91	0.00	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	28.94 %	19.72 %
R1	106.33	589.63	600T125-54 (50) & (1) 1/2" x 4" Titen HD to 2500 psi min concrete	22.86 %	14.47 %

* Reference catalog for connector and anchor requirement notes as well as screw placements requirement

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Studs

Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)¹	LSUBH (Max)¹	SUBH (Min)¹	SUBH (Max)¹	MSUBH (Min)¹	MSUBH (Max)¹
Top CantiLever	Span	N/A	0.00	N/A	N/A	N/A	N/A	N/A	N/A
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Simpson Strong-Tie® Wall Stud Bridging Connectors @ Jambs

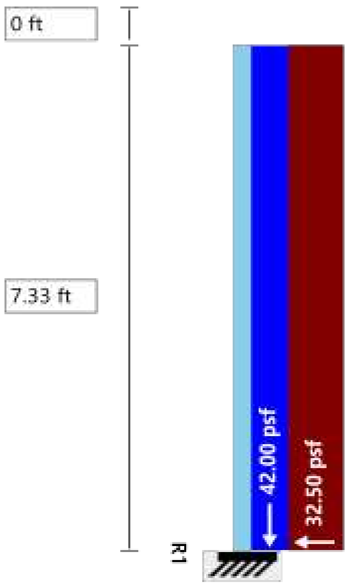
Span/CantiLever	Bracing Length (in.)	Design Number of Braces	Pn(lb.)	LSUBH (Min)¹	LSUBH (Max)¹	SUBH (Min)¹	SUBH (Max)¹	MSUBH (Min)¹	MSUBH (Max)¹
Top CantiLever	Span	N/A	0.00	N/A	N/A	N/A	N/A	N/A	N/A
Span	Varies	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

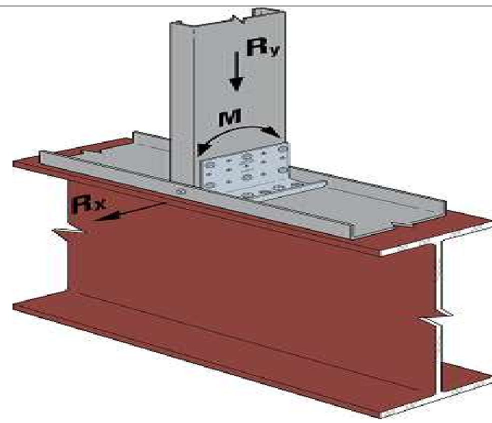
Notes:

1) Values in parentheses are stress ratios.
2) Bridging connectors are not designed for back-back,box, or built-up sections.
3) Reference www.strongtie.com for latest load data, important information, and general notes
4) CFS Designer will not select bridging connectors unless all flexural and axial bracing settings are the same.
5) If the bracing length is larger than the span length, bridging connectors are not designed.

Project Name: CFS
Model: KneeWall –1
Code: 2012 NASPEC [AISI S100-2012]

Date: 08/06/2021
Simpson Strong-Tie® CFS Designer™ 3.4.6.0





Vertical

F2= 410.48 lb

Live

F4= 317.63 lb

M= 1164.13 ft-lbs

Connector : RCKW5.5 @16 (in) oc w/ (6)#12 Screws

Anchor : (4) #12 Screws to Steel

Steel Connector Stiffness: 266100 in-lb/rad

Section : 600S200-54 (50 ksi) Single C Stud @ 16 "o.c.

Maxo= 2532.9 Ft-Lb **Va=** 2822.9 lb Pa = 3823.0 lbs Moment of Inertia, I=3.32 in^4

Loads have NOT been modified for strength checks
Loads have NOT been modified for deflection calculations
Stud Bracing (KyLy,KtLt) Distance: 88 " o.c.
Distortional Buckling Inputs: kφ = 0 lb-in/in; Lm = None

		Live	**Allowed**	**Interaction**
Member	Axial, lbs	410.48	3823.0	10.74%
(ASD)	Shear, lbs	317.63	1947.4	16.31%
	Moment, ft-lbs	1164.13	1733.0	67.17%
	Shear/Moment	0.473	1.000	47.30%
	Axial/Moment	0.795	1.000	79.50%
	Deflection Member, in	0.276		
	Deflection Connector, in	4.618		
	Total Deflection, in	4.894		
	L/	L/36		
Connector	Shear, lbs	317.63	1295	24.53%
& Anchorage	Moment, in-lbs	13969.51	6430	217.26%
	Shear/Moment			218.64%

*Loads for anchors converted to LRFD for design per ACI 318-14 chapter 17

Summary

The gravity structure system of the project referenced above consists primarily of steel beams and non load-bearing cold-formed metal stud walls. The walls are supported at grade by continuous footings. Headers are used at all opening locations to carry the gravity loads to the jambs of the openings. Additional studs are provided at the jambs to carry the concentrated loads at the ends of the headers. All jamb studs are supported at grade by footings, either continuous or spread depending on the magnitude of the load that is being resisted. All steel columns are supported by spread footings. The locations of all footings are indicated on the structural framing plans.

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. Refer to the “Wood” and “Steel” sections of these calculations for the design of the members being supported by the continuous and spread footings.

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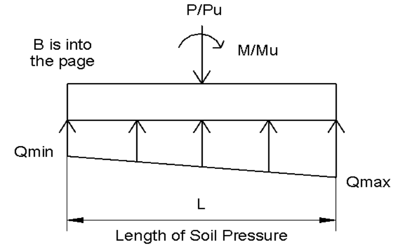
Project Paragon HUB | Project No. 21-036
Calc. By DN | Checked By TAJ | Date 08/06/21

Footing Designation: F2

Footing Location: 4E



General Information:

Footing Length, L =	4	ft			
Footing Width, B =	4	ft			
Footing Depth, H =	18	in			
Location =	Corner	in			
Steel Depth, d =	14.0625	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 =	1.5	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 =	4.63	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	3.8	k	Live =	1.6	
Applied Live Load =	3.09	k	Uplift=	1.6	
ASD Total Load, P =	11.51875	k			
LRFD Total Load, Pu =	15.0575	k			
ASD Uplift Load =	3.86	k			
LRFD Uplift Load =	6.175	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	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 =	0.720	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.720	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 =	0.941	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.941	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	0.941	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.039	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 =	3.91	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	55.46	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 =	23.06	in	(Critical Section B + d)
b2 =	23.06	in	(Column Height L + d)
b0 =	92.25	in	(2*b1 + 2*b2)
Vu2 =	11.58	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	319.74	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	269.05	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	213.16	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	3.859375	k	(From Above)
Required Dead Load =	6.43	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	8.43125	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 =	8.43	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 =	0.21	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.001	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.004	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 =	0.51	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 =	0.003	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.011	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 =	0.324	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

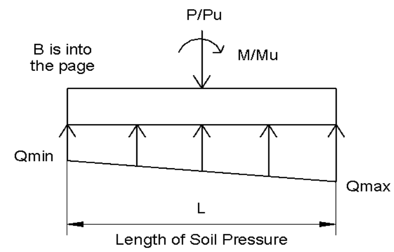
Final Footing Design:

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

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

Footing Designation: F2

Footing Location: 3E



General Information:

Footing Length, L =	4	ft		
Footing Width, B =	4	ft		
Footing Depth, H =	18	in		
Location =	Edge	in		
Steel Depth, d =	14.0625	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 =	1.5	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 =	9.92	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	4	k	Live =	1.6	
Applied Live Load =	6.62	k	Uplift=	1.6	
ASD Total Load, P =	20.539584	k			
LRFD Total Load, Pu =	27.29383424	k			
ASD Uplift Load =	8.27	k			
LRFD Uplift Load =	13.2316672	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	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 =	1.284	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.284	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 =	1.706	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.706	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.706	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.039	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 =	7.09	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	55.46	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 =	23.06	in	(Critical Section B + d)
b2 =	23.06	in	(Column Height L + d)
b0 =	92.25	in	(2*b1 + 2*b2)
Vu2 =	20.99	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	30		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	319.74	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	350.29	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	213.16	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	8.269792	k	(From Above)
Required Dead Load =	13.78	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	13.9237504	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 =	13.92	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 =	0.45	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.009	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 =	0.92	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 =	0.005	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.019	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 =	0.324	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

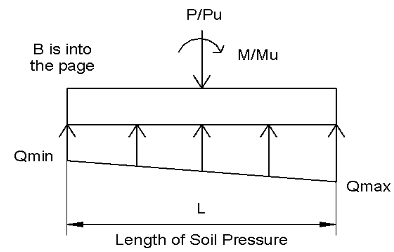
Final Footing Design:

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

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

Footing Designation: F2

Footing Location: 2D



General Information:

Footing Length, L =	4	ft		
Footing Width, B =	4	ft		
Footing Depth, H =	18	in		
Location =	Corner	in		
Steel Depth, d =	14.0625	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 =	1.5	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 =	9.67	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	3.8	k	Live =	1.6	
Applied Live Load =	6.45	k	Uplift=	1.6	
ASD Total Load, P =	19.9164069	k			
LRFD Total Load, Pu =	26.47831338	k			
ASD Uplift Load =	8.06	k			
LRFD Uplift Load =	12.89312552	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	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 =	1.245	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.245	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 =	1.655	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.655	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.655	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.039	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	6.88	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	55.46	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 =	23.06	in	(Critical Section B + d)
b2 =	23.06	in	(Column Height L + d)
b0 =	92.25	in	(2*b1 + 2*b2)
Vu2 =	20.37	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	319.74	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	269.05	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	213.16	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	8.058203448	k	(From Above)
Required Dead Load =	13.43	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	13.46984414	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 =	13.47	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 =	0.44	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.002	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.009	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 =	0.89	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 =	0.005	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.019	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 =	0.324	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

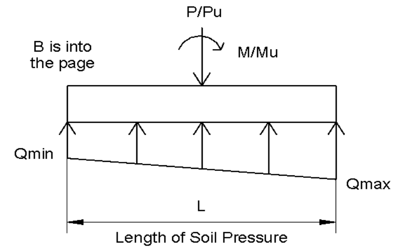
Final Footing Design:

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

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

Footing Designation: F2

Footing Location: 1C



General Information:

Footing Length, L =	4	ft			
Footing Width, B =	4	ft			
Footing Depth, H =	18	in			
Location =	Corner	in			
Steel Depth, d =	14.0625	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 =	1.5	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 =	4.38	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	3.8	k	Live =	1.6	
Applied Live Load =	2.92	k	Uplift=	1.6	
ASD Total Load, P =	11.09557289	k			
LRFD Total Load, Pu =	14.48197914	k			
ASD Uplift Load =	3.65	k			
LRFD Uplift Load =	5.836458316	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	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 =	0.693	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.693	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 =	0.905	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.905	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	0.905	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.039	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 =	3.76	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	55.46	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 =	23.06	in	(Critical Section B + d)
b2 =	23.06	in	(Column Height L + d)
b0 =	92.25	in	(2*b1 + 2*b2)
Vu2 =	11.14	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	319.74	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	269.05	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	213.16	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	3.647786447	k	(From Above)
Required Dead Load =	6.08	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	8.177343737	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 =	8.18	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 =	0.20	k-ft / ft		(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529			(m = fy/(0.85*f'c))
Ru =	0.001	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.004	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 =	0.49	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 =	0.003	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0001			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0001			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.010	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	12	in		
As Provided =	0.31	in^2/ft	=	5 Bars in B Direction
			=	5 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.324	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.31	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.62	in^2/ft	
T&S Steel Provided?	**YES**		

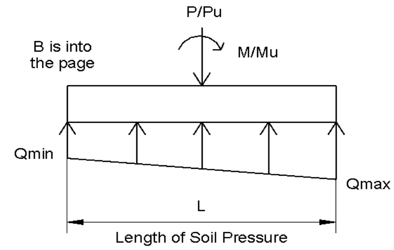
Final Footing Design:

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

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

Footing Designation: F4

Footing Location: 1B



General Information:

Footing Length, L =	3.5	ft		
Footing Width, B =	3.5	ft		
Footing Depth, H =	58	in		
Location =	Corner	in		
Steel Depth, d =	54.0625	in		(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in		
Slab Depth Above Footing =	4	in		
Area of Footing =	12.25	ft^2		(B*L)
Soil Bearing Pressure =	1.5	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 =	4.37	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	9.034375	k	Live =	1.6	
Applied Live Load =	2.92	k	Uplift=	1.6	
ASD Total Load, P =	16.325	k			
LRFD Total Load, Pu =	20.7565	k			
ASD Uplift Load =	3.65	k			
LRFD Uplift Load =	5.8325	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.583	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.333	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.333	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 =	0.583	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.694	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.694	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.694	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	-0.878	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 =	-5.20	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	186.55	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 =	63.06	in	(Critical Section B + d)
b2 =	63.06	in	(Column Height L + d)
b0 =	252.25	in	(2*b1 + 2*b2)
Vu2 =	-26.04	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	3361.25	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	3521.70	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	2240.83	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	3.6453125	k	(From Above)
Required Dead Load =	6.08	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	13.40875	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 =	13.41	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 =	0.18	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.001	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	6	in	
As Provided =	0.62	in^2/ft	= 8 Bars in B Direction
			= 8 Bars in L Direction

Bottom Steel:

Mu =	0.65	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.004	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	6	in	
As Provided =	0.62	in^2/ft	= 8 Bars in B Direction
			= 8 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	1.188	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.62	in^2/ft	
As Provided Bott =	0.62	in^2/ft	
As Provided Total =	1.24	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

Footing Width, B =	**3.5**	ft
Footing Length L =	**3.5**	ft
Footing Depth, H =	**58**	in

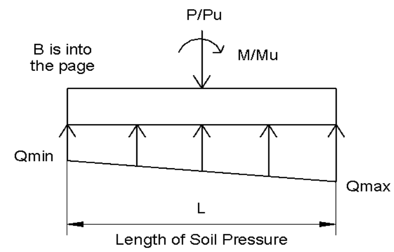
Top Steel =	**#5 bars**	**@6 inches O.C.**
Bottom Steel =	**#5 bars**	**@6 inches O.C.**

BSE STRUCTURAL ENGINEERS

Project: Paragon HUB | Project No.: 21-036
Calc. By: DN | Checked By: TAJ | Date: 08/06/21

Footing Designation: F3

Footing Location: 2B



General Information:

Footing Length, L =	5.5	ft		
Footing Width, B =	5.5	ft		
Footing Depth, H =	34	in		
Location =	Interior	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 =	30.25	ft^2		(B*L)
Soil Bearing Pressure =	1.5	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 =	17.01	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	14.36875	k	Live =	1.6	
Applied Live Load =	11.34	k	Uplift=	1.6	
ASD Total Load, P =	42.7214875	k			
LRFD Total Load, Pu =	55.802223	k			
ASD Uplift Load =	14.18	k			
LRFD Uplift Load =	22.68219	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.917	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.412	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.412	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 =	0.917	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.845	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.845	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.845	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.122	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 =	11.39	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	163.01	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 =	36.26	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1157.76	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1870.94	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	771.84	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	14.17636875	k	(From Above)
Required Dead Load =	23.63	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	31.3803925	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 =	31.38	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 =	0.47	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.001	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.005	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	11	in	
As Provided =	0.34	in^2/ft	= 7 Bars in B Direction
			= 7 Bars in L Direction

Bottom Steel:

Mu =	1.16	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 =	0.001	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.011	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	11	in	
As Provided =	0.34	in^2/ft	= 7 Bars in B Direction
			= 7 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.6696	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.34	in^2/ft	
As Provided Bott =	0.34	in^2/ft	
As Provided Total =	0.68	in^2/ft	
T&S Steel Provided?	**YES**		

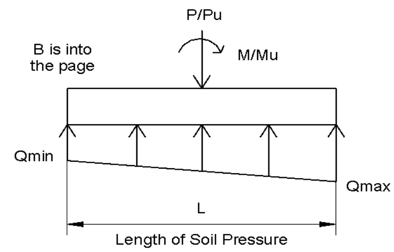
Final Footing Design:

Footing Width, B =	**5.5**	ft
Footing Length L =	**5.5**	ft
Footing Depth, H =	**34**	in

Top Steel =	**#5 bars**	**@11 inches O.C.**
Bottom Steel =	**#5 bars**	**@11 inches O.C.**

Footing Designation: F1

Footing Location: 2A, 5A



General Information:

Footing Length, L =	3.5	ft	
Footing Width, B =	3.5	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 =	8	in	
Area of Footing =	12.25	ft^2	(B*L)
Soil Bearing Pressure =	1.5	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 =	6.15	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	5.5125	k	Live =	1.6	
Applied Live Load =	4.10	k	Uplift=	1.6	
ASD Total Load, P =	15.76835938	k			
LRFD Total Load, Pu =	20.56296875	k			
ASD Uplift Load =	5.13	k			
LRFD Uplift Load =	8.2046875	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.583	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.287	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.287	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 =	0.583	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	3.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.679	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.679	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.679	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	0.122	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	0.72	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	103.74	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 =	2.78	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	20		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1157.76	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1128.43	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	771.84	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	5.127929688	k	(From Above)
Required Dead Load =	8.55	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	11.66601563	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 =	11.67	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.01	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	11	in	
As Provided =	0.34	in^2/ft	= 5 Bars in B Direction
			= 5 Bars in L Direction

Bottom Steel:

Mu =	0.01	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	11	in	
As Provided =	0.34	in^2/ft	= 5 Bars in B Direction
			= 5 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.6696	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.34	in^2/ft	
As Provided Bott =	0.34	in^2/ft	
As Provided Total =	0.68	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

Footing Width, B =	**3.5**	ft
Footing Length L =	**3.5**	ft
Footing Depth, H =	**34**	in

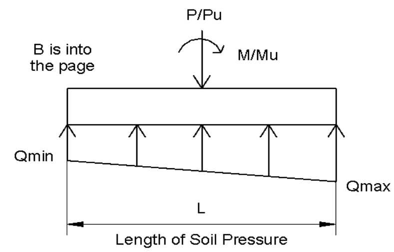
Top Steel =	**#5 bars**	**@11 inches O.C.**
Bottom Steel =	**#5 bars**	**@11 inches O.C.**

BSE STRUCTURAL ENGINEERS

Project Paragon HUB Project No. 21-036
Calc. By DN Checked By TAJ Date 08/06/21

Footing Designation: F3

Footing Location: 4B



General Information:

Footing Length, L =	5.5	ft		
Footing Width, B =	5.5	ft		
Footing Depth, H =	34	in		
Location =	Interior	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 =	30.25	ft^2		(B*L)
Soil Bearing Pressure =	1.5	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 =	15.76	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	14.36875	k	Live =	1.6	
Applied Live Load =	10.51	k	Uplift=	1.6	
ASD Total Load, P =	40.63398438	k			
LRFD Total Load, Pu =	52.96321875	k	***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accor**		
ASD Uplift Load =	13.13	k			
LRFD Uplift Load =	21.0121875	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 =	0.917	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.343	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.343	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 =	0.917	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.751	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.751	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.751	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.122	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 =	10.81	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	163.01	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 =	34.41	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1157.76	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1870.94	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	771.84	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	13.13261719	k	(From Above)
Required Dead Load =	21.89	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	30.12789063	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 =	30.13	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 =	0.44	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.001	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.004	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	11	in	
As Provided =	0.34	in^2/ft	= 7 Bars in B Direction
			= 7 Bars in L Direction

Bottom Steel:

Mu =	1.10	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 =	0.001	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.011	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	11	in	
As Provided =	0.34	in^2/ft	= 7 Bars in B Direction
			= 7 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.6696	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.34	in^2/ft	
As Provided Bott =	0.34	in^2/ft	
As Provided Total =	0.68	in^2/ft	
T&S Steel Provided?	**YES**		

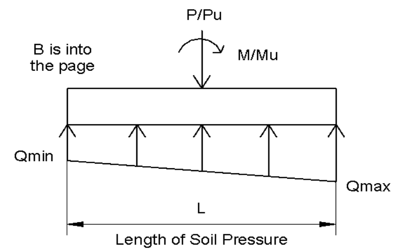
Final Footing Design:

Footing Width, B =	**5.5**	ft
Footing Length L =	**5.5**	ft
Footing Depth, H =	**34**	in

Top Steel =	**#5 bars**	**@11 inches O.C.**
Bottom Steel =	**#5 bars**	**@11 inches O.C.**

Footing Designation: GRADE BEAM ADEQUATE

Footing Location: HSS BEAM



General Information:

Footing Length, L =	2	ft	
Footing Width, B =	2.5	ft	
Footing Depth, H =	36	in	
Location =	Edge	in	
Steel Depth, d =	32.0625	in	(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in	
Slab Depth Above Footing =	4	in	
Area of Footing =	5	ft^2	(B*L)
Soil Bearing Pressure =	1.5	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 =	4.16	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	2.375	k	Live =	1.6	
Applied Live Load =	0.00	k	Uplift=	1.6	
ASD Total Load, P =	6.5375	k			
LRFD Total Load, Pu =	7.845	k			
ASD Uplift Load =	0.00	k			
LRFD Uplift Load =	0	k			

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

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.333	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	2.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.308	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.308	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 =	0.333	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	2.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.569	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.569	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.569	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	-0.711	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 =	-2.79	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	79.03	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 =	41.06	in	(Critical Section B + d)
b2 =	41.06	in	(Column Height L + d)
b0 =	164.25	in	(2*b1 + 2*b2)
Vu2 =	-10.53	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	30		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1124.94	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1699.55	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	865.34	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	477.36	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	0	k	(From Above)
Required Dead Load =	0.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	6.5375	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 =	6.54	k	(Applied Dead + Slab + Wall Weight + Add. Slab + Cont. Ftg)
Adequate for Uplift?	**Footing is Adequate to Resist Uplift**		(Calculation assumes wall is above the cont. ftg.)

Top Steel:

Mu =	0.00	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	9	in	
As Provided =	0.41	in^2/ft	= 5 Bars in B Direction
			= 4 Bars in L Direction

Bottom Steel:

Mu =	0.40	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.004	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	9	in	
As Provided =	0.41	in^2/ft	= 5 Bars in B Direction
			= 4 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.7128	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.41	in^2/ft	
As Provided Bott =	0.41	in^2/ft	
As Provided Total =	0.83	in^2/ft	
T&S Steel Provided?	**YES**		

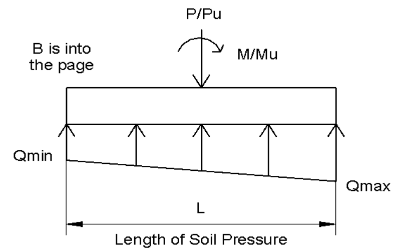
Final Footing Design:

Footing Width, B =	**2.5**	ft
Footing Length L =	**2**	ft
Footing Depth, H =	**36**	in

Top Steel =	**#5 bars**	**@9 inches O.C.**
Bottom Steel =	**#5 bars**	**@9 inches O.C.**

Footing Designation: F3

Footing Location: TRASH



General Information:

Footing Length, L =	5.5	ft			
Footing Width, B =	5.5	ft			
Footing Depth, H =	34	in			
Location =	Edge	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 =	30.25	ft^2			(B*L)
Soil Bearing Pressure =	1.5	ksf			
Allowable or Effective SBC?	Allowable				
Concrete Strength =	3	ksi			
	B Direction			L Direction	
Column Size =	8.00 in		X	8.00 in	
Base Plate Size =	14.00 in		X	14.00 in	
Critical Section =	11.00 in		X	11.00 in	

Loading:

Vertical Loads:			LRFD Factors:		
Applied Dead Load =	1.45	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	13.6125	k	Live =	1.6	
Applied Live Load =	1.96	k	Uplift=	1.6	
ASD Total Load, P =	17.02372675	k			
LRFD Total Load, Pu =	21.2110099	k	***Note: Uplift Pressure check below is for ASD pressures. Adjust loads accor**		
ASD Uplift Load =	2.45	k			
LRFD Uplift Load =	3.912689	k			

Moments:			LRFD Factors:		
Dead Load Moment =	2.5	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	8.66	k-ft	Live =	1.6	
ASD Total Moment, M =	11.16	k-ft			
LRFD Total Moment, Mu =	16.856	k-ft			

ASD Soil Pressures:

e =	0.656	ft	(ASD M / P)
Kern =	0.917	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	0.160	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	0.965	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 =	0.917	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.500	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	0.093	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.309	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.079	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.039	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	6.82	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	163.01	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 =	41.06	in	(Critical Section B + d)
b2 =	41.06	in	(Column Height L + d)
b0 =	164.25	in	(2*b1 + 2*b2)
Vu2 =	13.00	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	30		(ACI 318-11 Section 11.11.2.1)
β =	1		(ACI 318-11 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1217.04	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1519.44	k	(ACI 318-11 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	811.36	k	(ACI 318-11 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	649.74	k	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	2.445430625	k	(From Above)
Required Dead Load =	4.08	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	15.06738225	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 =	15.07	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 =	0.07	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.001	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	11	in		
As Provided =	0.34	in^2/ft	=	7 Bars in B Direction
			=	7 Bars in L Direction

Bottom Steel:

Mu =	0.67	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 =	0.001	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000			(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000			(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.007	in^2/ft		(As = Governing ρ*12 inches*d)
Bar # =	5			
Bar Spacing =	11	in		
As Provided =	0.34	in^2/ft	=	7 Bars in B Direction
			=	7 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.6696	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.34	in^2/ft	
As Provided Bott =	0.34	in^2/ft	
As Provided Total =	0.68	in^2/ft	
T&S Steel Provided?	**YES**		

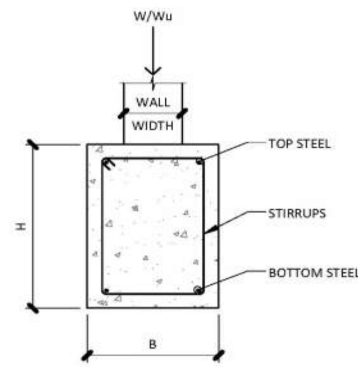
Final Footing Design:

Footing Width, B =	**5.5**	ft
Footing Length L =	**5.5**	ft
Footing Depth, H =	**34**	in

Top Steel =	**#5 bars**	**@11 inches O.C.**
Bottom Steel =	**#5 bars**	**@11 inches O.C.**

Grade Beam Design

Grade Beam Location:



General Information:

Footing Width, B =	24	in	
Footing Depth, H =	34	in	
Steel Depth, d =	30.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	6	in	
Soil Bearing Pressure =	1.5	ksf	
Allowable or Effective SBC?	Allowable		
Footing Concrete Strength =	3	ksi	
Wall Concrete Strength =	3	ksi	

Loading:

Vertical Loads:

Applied Dead Load =	0.65049	klf	
Wall Weight =	0	psf	
Wall Height =	40	ft	
Total Wall Weight =	0	klf	
Footing Weight =	0.85	klf	
Applied Live Load =	0	klf	
ASD Total Load, W =	1.50049	klf	
LRFD Total Load, Wu =	1.800588	klf	

LRFD Factors: (ASCE 7 Combo)

Dead = 1.2
Live = 1.6

ASD Soil Pressures:

Required Footing Width =	1.000326667	ft	(Footing Width = W / Soil Bearing Pressure)
Actual Soil Bearing Pressure =	750.245	psf	(Actual Soil Bearing Pressure = W / Footing Width)
Chosen Footing Width =	2	ft	
Assumed Footing Span =	2	ft	
Is Footing Width Adequate?	**YES**		

Plain Concrete Shear Check: Cantilevered Side of Footing

LRFDl Bearing Pressure =	900.294	psf	
h=	32	in	(h = H - 2 in.)
Cantilever =	9	in	(Cantilever = B/2 - Wall Width/2)
Vu1 =	0.68	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 =	9	in	(Cantilever = B/2 - Wall Width/2)
Mu =	0.25	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 / 1
Adequate in Flexure?	**YES**		

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

Vu1 =	1.80	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	59.28	klf	(ACI 318-11 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-11 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Oth

Use #3 Stirrups at 24 in. O.C. (Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	119.34	klf	(ACI 318-11 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall
Adequate in Bearing?	**YES**		

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

Mu =	0.90	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 318-11 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 2
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 Re
A's Required =	0.009	in^2	(As = Governing ρ*B*d)
Bar # =	5		
Number of Bars =	3	bars	
As Provided =	0.93	in^2	
Is Steel Adequate ?	**YES**		

Top Steel:

Bar # =	5	
Number of Bars =	3	bars
As Provided =	0.93	in^2

Temperature & Shrinkage Steel:

Minimum Steel =	1.4688	in^2/ft	(ACI 318-11 Section 7.12.2.1 T&S Steel = 0.0018*12 in
As Provided Top =	0.93	in^2/ft	
As Provided Bott =	0.93	in^2/ft	
As Provided Total =	1.86	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

Footing Width, B =	**24**	in
Footing Depth, H =	**34**	in

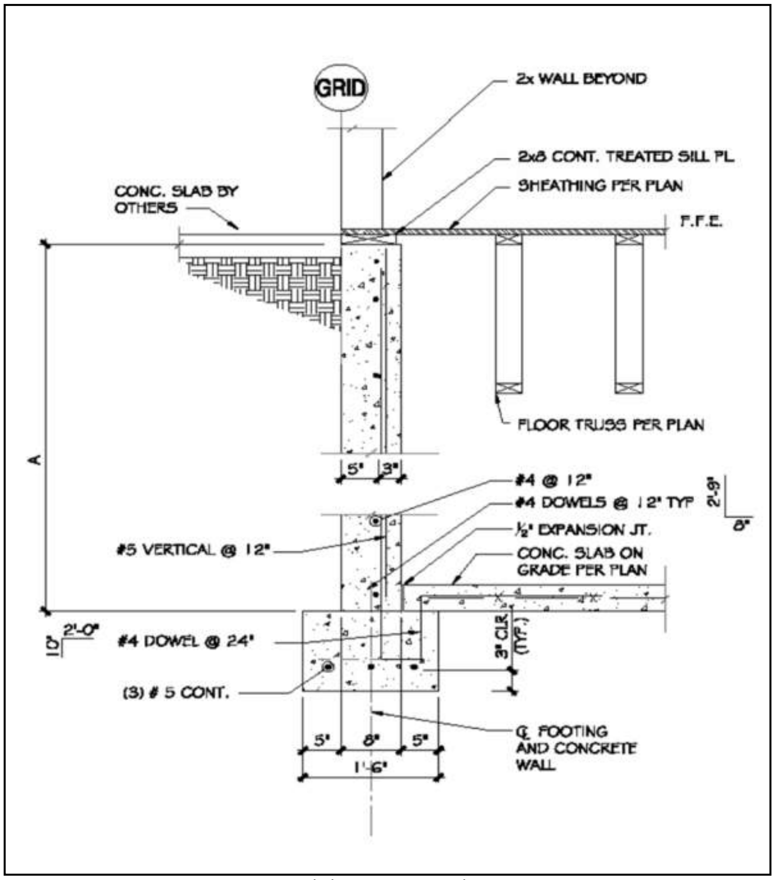
Top Steel = **(3) #5 bars**
Bottom Steel = **(3) #5 bars**
Stirrups = **#3 Stirrups at 24 in. O.C.**

STRUCTURAL ENGINEERS

Project Paragon HUB Project No. 21-036

Calc. By DN Checked By TAJ Date 08/06/21

Basement Wall Design:



Note: Detail shown is general in nature.

Wall Properties

Location =		
A (wall or soil height) =	4.00	ft
B (wall thickness) =	8.00	in
Footing Width =	2.50	ft
Footing Depth =	1.50	ft

Loading:

Soil wt =	120	pcf
At-Rest Pressure Coeff =	0.35	
Passive Pressure Coeff =	2	
Grade Vertical Live Load =	50	psf
Vertical Dead Load on Wall =	920	plf
At-Rest Pressure =	42.0	pcf
Surcharge Pressure =	17.5	psf

Wall Reinforcment:

Mu =	0.33	k-ft
d =	4	in.
b =	12	in.
f_y =	60000	psi
f'_c =	4000	psi
m =	17.647059	$= f_y / 0.85f'_c$
φ =	0.9	(for bending)
R_u =	23.003556	$= M_u / (0.9 \times b \times d^2)$
ρ_{req} =	0.0004	$= [1 - (1 - 2 \times m \times R_u/f_y)^{1/2}] / m$ ($A_{s,\ req}$ = 0.09 in^2)

Use # 4 bars @ 12 " o.c. (A_s = 0.20 in^2) **OK**

Wall Outkick at Base:

Does a slab pin the base? =	NO	**SEE FOLLOWING DESIGN**
F.S. against sliding =	1.5	
V (at top) =	260	lbs/ft (Resistance Outside Scope of Calculation)
V (at footing) =	472	lbs/ft (Outkick Force to Be Resisted)
V x F.S. =	707	lbs/ft
Coefficient of Friction (µ) =	0.3	
Wt of wall =	400	lbs/ft
Wt of footing =	563	lbs/ft
Wt of soil =	440	lbs/ft
Add. Soil Due to Surcharge =	50	lbs/ft
Superimposed DL	920	lbs/ft
Total =	2,373	lbs/ft
µ x Total =	712	lbs/ft
V x F.S. - µ x Total =	-4	lbs/ft to be resisted by passive pressure
Passive Pressure =	240	psf
Soil Above Footing Toe =	0	ft for additional passive resistance
Passive Resistance =	270	lbs/ft

OK Against Sliding

ANCHOR BOLT DESIGN

In accordance with ACI318-14

Tedds calculation version 2.1.02

Anchor bolt geometry

Type of anchor bolt — Cast-in headed end bolt anchor

Diameter of anchor bolt — d_a = **0.625** in

Total number of bolts — n_{total} = **1**

Total number of bolts in tension — n_{tens} = **1**

Number of threads per inch — n_t = **11**

Effective cross-sectional area of anchor — $A_{se} = \pi / 4 \times (d_a - 0.9743 \text{ in} / n_t)^2 = \mathbf{0.226} \text{ in}^2$

Embedded depth of each anchor bolt — h_{ef} = **8**in

Foundation geometry

Member thickness — h_a = **36** in

Dist center of baseplate to left edge foundation — x_{ce1} = **12** in

Dist center of baseplate to right edge foundation — x_{ce2} = **12** in

Dist center of baseplate to bot. edge foundation — y_{ce1} = **12** in

Dist center of baseplate to top edge foundation — y_{ce2} = **20** in

Material details

Minimum yield strength of steel — f_{ya} = **36** ksi

Nominal tensile strength of steel — f_{uta} = **58** ksi

Compressive strength of concrete — f'_c = **3** ksi

Concrete modification factor — λ = **1.00**

Modification factor for cast-in anchor concrete failure

$\lambda_a = 1.0 \times \lambda = \mathbf{1.00}$

Strength reduction factors

Tension of steel element — $\phi_{t,s}$ = **0.75**

Shear of steel element — $\phi_{v,s}$ = **0.65**

Concrete tension — $\phi_{t,c}$ = **0.75**

Concrete shear — $\phi_{v,c}$ = **0.75**

Concrete tension for pullout — $\phi_{t,cB}$ = **0.70**

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

Anchor forces

Number of bolt rows in tension — N_{boltN} = **1**

Axial force in bolts for row 1 — N_1 = **9.00** kips

Total axial force on bolt group — N_R = **9.00** kips

Maximum axial force to single bolt — $N_{max,s}$ = **9.00** kips

Eccentricity of axial load (from bolt group centroid) — e'_N = **0.00** in

Shear force applied to bolt group — V = **0.00** kips

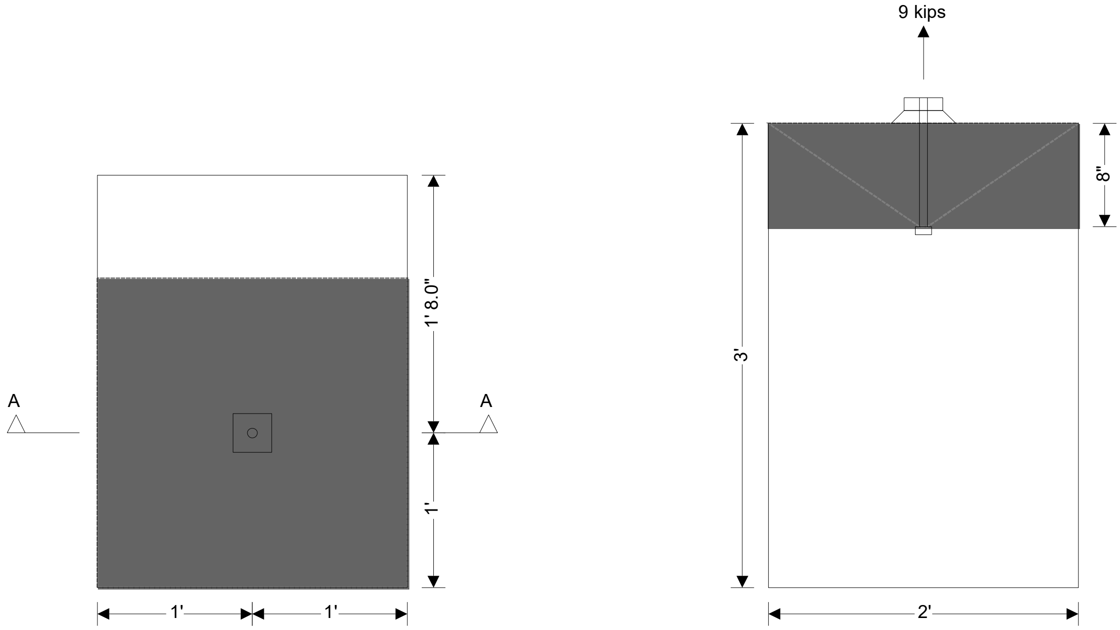
Steel strength of anchor in tension (17.4.1)

Nominal strength of anchor in tension — $N_{sa} = A_{se} \times f_{uta} = \mathbf{13.11}$ kips

Steel strength of anchor in tension — $\phi N_{sa} = \phi_{t,s} \times N_{sa} = \mathbf{9.83}$ kips

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

Check concrete breakout strength of anchor bolt in tension (17.4.2)



Plan on foundation

Section A-A

Concrete breakout - tension

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

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

Projected area for groups of anchors A_{Nc} = **576** 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 $c_{a,min}$ = **12** 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 $N_{cb} = A_{Nc} / A_{Nco} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$ = **29.74** kips

Concrete breakout strength $\phi N_{cb} = \phi_{t,c} \times N_{cb}$ = **22.31** kips

PASS - Breakout strength exceeds tension in bolts

Pullout strength (17.4.3)

Net bearing area of the head of anchor A_{brg} = **1** in^2

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

Pullout strength for single anchor $N_p = 8 \times A_{brg} \times f'_c$ = **24.00** kips

Nominal pullout strength of single anchor $N_{pn} = \psi_{c,P} \times N_p$ = **24.00** kips

Pullout strength of single anchor $\phi N_{pn} = \phi_{t,cB} \times N_{pn}$ = **16.80** kips

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

Side face blowout strength (17.4.4)

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

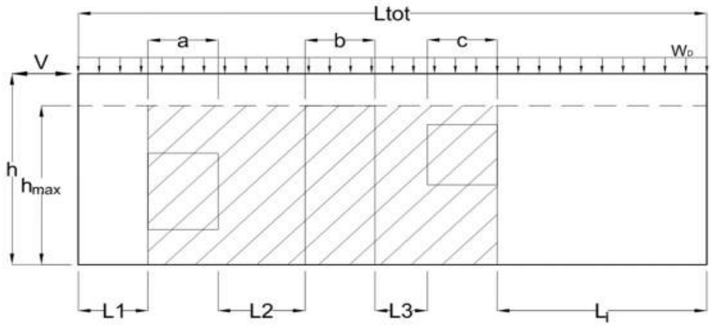
The lateral stability system of the project referenced above consists of metal deck diaphragms, and gypsum wallboard-sheathed cold-formed metal stud walls. All roof diaphragms are designed to transfer lateral loads from the exterior walls, and those induced into the deck by seismic loads, into the shear walls. Lateral resisting members directly transfer all 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. Refer to this section of these calculations for the shear wall design.

Cold-formed Shear Wall Design (ASD)

Code: AISI S213-07

Shear wall Location: Grid A



General Information:

Total applied **ASD** Wind load, V_w =	3280	lbs.		
Wall panel height, h =	13	ft		
Width of panel, L_{tot} =	39.2	ft		
Max height of openings, h_{max} =	8	ft		
Wall Self Weight, w_{self}	12	psf		
Applied Dead Load, w_D	100	plf	Ratio:	
Segment Length, L1 =	8.77	ft	1.5:1	OK
Segment Length, L2 =	1.33	ft	9.8:1	Segment exceeds MAX (h/w) ratio, omitted from Shear wall design
Segment Length, L3 =	10	ft	1.3:1	OK
Segment Length, L4 =	0	ft	-	
Segment Length, L5 =	0	ft	-	
Segment Length, L6 =	0	ft	-	

In-Plane Shear Design Check:

Sum of Perf. Shear Wall lengths, $\sum L_i$	18.77	ft	$(L_1 + L_2 + L_3 + \ldots)$
Total Length of openings	20.43	ft	(a + b + c + …)
Total Area of Openings, A_o	163.44	ft^2	(a*h + b*h + c*h + …)
Sheathing Area Ratio, r	0.60		$[1/ (1 + (Ao/h\sum L_i)]$
Percent Full-Height Sheathing	47.9%		$(\sum L_i/L_{tot})$
Maximum Opening Height Ratio	0.62		(h_{max}/h)
Shear Capacity Adjustment factor, C_a	0.69		$[r/(3-2*r)]*(L_{tot}/\sum L_i)$
ASD Shear wall force, v	**251.8**	plf	$(V/C_a*\sum L_i)$

Note: Area assumed to be total length of openings times max height

Sheathing Type:	1/2" Gypsum Board on one side of wall; Studs max 24" o.c.		
Screw Size:	#10 screws		
Pane Edge/Field Fastener Spacing:	7	inches	
Field Fastener Spacing:	7	inches	
Nominal Shear Strength (R_n)	290.0	plf	Table References
ASD Adjustment Factor,	2.0		AISI S213-07, Section C2.1
Number of Sides Sheathed	2.0		
ASD Shear Capacity, (R_n/Ω)*(# Sides)	**290.0**		**OK**

Chord Force at Shear Wall Ends

Stud Wall Spacing	24	in	
Boundary Chord Force, F	3273.61	lbs.	$(V*h)/(Ca*\sum L)$
Allowable Tension Force, T_a	1832.08	lbs.	$F - 0.6[(w_D+(\text{Wall Wt.})*(h)]*(\sum Li/2)$
Allowable Compression Force, C_a	3529.61	lbs.	$F + [(w_D+(\text{Wall Wt.})*(h)]*(s/2)$

*Refer to AISIWIN Calculation for Chord Design

Sill Plate: USE: 600T125-54

Chords: USE: (2)600S162-54

Sill Plate Anchor Force

Anchor Spacing	32	in
ASD Sill Plate Anchor Force	671.5	lbs./anchor

*Refer to Hilti Profis Calculation for Anchorage Design

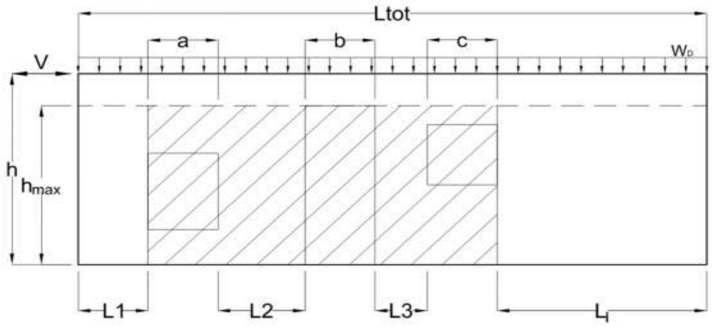
Hold-down: USE: S/HDU4

Engineer Note: Refer to Sections C2.2 Thru C2.2.4 of AISI S213-07 For additional framing requirements.

Cold-formed Shear Wall Design (ASD)

Code: AISI S213-07

Shear wall Location: Grid B



General Information:

Total applied **ASD** Wind load, V_w =	3786	lbs.		
Wall panel height, h =	13	ft		
Width of panel, L_{tot} =	23.33	ft		
Max height of openings, h_{max} =	0	ft		
Wall Self Weight, w_{self}	12	psf		
Applied Dead Load, w_D	100	plf		
			Ratio:	
Segment Length, L1 =	23.33	ft	0.6:1	OK
Segment Length, L2 =	0	ft	-	
Segment Length, L3 =	0	ft	-	
Segment Length, L4 =	0	ft	-	
Segment Length, L5 =	0	ft	-	
Segment Length, L6 =	0	ft	-	

In-Plane Shear Design Check:

Sum of Perf. Shear Wall lengths, $\sum L_i$	23.33	ft	$(L_1 + L_2 + L_3 + \ldots)$
Total Length of openings	0	ft	$(a + b + c + \ldots)$
Total Area of Openings, A_o	0.00	ft^2	$(a*h + b*h + c*h + \ldots)$
Sheathing Area Ratio, r	1.00		$[1/(1 + (Ao/h\sum L_i)]$
Percent Full-Height Sheathing	100.0%		$(\sum L_i/L_{tot})$
Maximum Opening Height Ratio	0.00		(h_{max}/h)
Shear Capacity Adjustment factor, C_a	1.00		$[r/(3-2*r)]*(L_{tot}/\sum L_i)$
ASD Shear wall force, v	**162.3**	plf	$(V/C_a*\sum L_i)$

Note: Area assumed to be total length of openings times max height

Sheathing Type:	1/2" Gypsum Board on one side of wall; Studs max 24" o.c.		
Screw Size:	#10 screws		
Pane Edge/Field Fastener Spacing:	7	inches	
Field Fastener Spacing:	7	inches	
Nominal Shear Strength (R_n)	290.0	plf	Table References
ASD Adjustment Factor,	2.0		AISI S213-07, Section C2.1
Number of Sides Sheathed	2.0		
ASD Shear Capacity, $(R_n/\Omega)*(\#\ Sides)$	**290.0**		**OK**

Chord Force at Shear Wall Ends

Stud Wall Spacing	24	in	
Boundary Chord Force, F	2109.42	lbs.	$(V*h)/(Ca*\sum L)$
Allowable Tension Force, T_a	317.67	lbs.	$F - 0.6[(w_D+(\text{Wall Wt.})*(h)]*(\sum Li/2)$
Allowable Compression Force, C_a	2365.42	lbs.	$F + [(w_D+(\text{Wall Wt.})*(h)]*(s/2)$

*Refer to AISIWIN Calculation for Chord Design

Sill Plate: USE: 600T125-54

Chords: USE: (2)600S162-54

Sill Plate Anchor Force

Anchor Spacing	32	in
ASD Sill Plate Anchor Force	432.7	lbs./anchor

*Refer to Hilti Profis Calculation for Anchorage Design

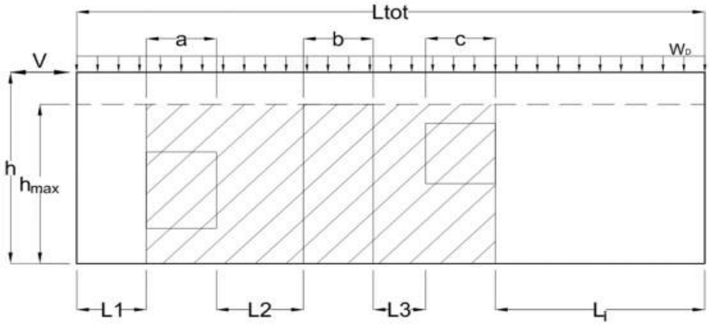
Hold-down: USE: S/HDU4

Engineer Note: Refer to Sections C2.2 Thru C2.2.4 of AISI S213-07 For additional framing requirements.

Cold-formed Shear Wall Design (ASD)

Code: AISI S213-07

Shear wall Location: Diagonal + Grid E



General Information:

Total applied **ASD** Wind load, V_w =	3263	lbs.		
Wall panel height, h =	13	ft		
Width of panel, L_{tot} =	42	ft		
Max height of openings, h_{max} =	0	ft		
Wall Self Weight, w_{self}	12	psf		
Applied Dead Load, w_D	100	plf	Ratio:	
Segment Length, L1 =	61	ft	0.2:1	OK
Segment Length, L2 =	0	ft	-	
Segment Length, L3 =	0	ft	-	
Segment Length, L4 =	0	ft	-	
Segment Length, L5 =	0	ft	-	
Segment Length, L6 =	0	ft	-	

In-Plane Shear Design Check:

Sum of Perf. Shear Wall lengths, $\sum L_i$	61	ft	$(L_1 + L_2 + L_3 + \ldots)$
Total Length of openings	-19	ft	(a + b + c + …)
Total Area of Openings, A_o	0.00	ft²	(a*h + b*h + c*h + …)
Sheathing Area Ratio, r	1.00		$[1/(1 + (Ao/h\sum L_i)]$
Percent Full-Height Sheathing	145.2%		$(\sum L_i/L_{tot})$
Maximum Opening Height Ratio	0.00		(h_{max}/h)
Shear Capacity Adjustment factor, C_a	0.69		$[r/(3-2*r)]*(L_{tot}/\sum L_i)$
ASD Shear wall force, v	**77.7**	plf	$(V/C_a*\sum L_i)$

Note: Area assumed to be total length of openings times max height

Sheathing Type:	1/2" Gypsum Board on one side of wall; Studs max 24" o.c.		
Screw Size:	#10 screws		
Pane Edge/Field Fastener Spacing:	7	inches	
Field Fastener Spacing:	7	inches	
Nominal Shear Strength (R_n)	290.0	plf	Table References
ASD Adjustment Factor,	2.0		AISI S213-07, Section C2.1
Number of Sides Sheathed	2.0		
ASD Shear Capacity, (R_n/Ω)*(# Sides)	**290.0**		**OK**

Chord Force at Shear Wall Ends

Stud Wall Spacing	24	in	
Boundary Chord Force, F	1009.89	lbs.	$(V*h)/(Ca*\sum L)$
Allowable Tension Force, T_a	-3674.91	lbs.	$F - 0.6[(w_D+(\text{Wall Wt.})*(h)]*(\sum Li/2)$
Allowable Compression Force, C_a	1265.89	lbs.	$F + [(w_D+(\text{Wall Wt.})*(h)]*(s/2)$

*Refer to AISIWIN Calculation for Chord Design
Sill Plate: USE: 600T125-54
Chords: USE: (2)600S162-54

Sill Plate Anchor Force

Anchor Spacing	32	in
ASD Sill Plate Anchor Force	207.2	lbs./anchor

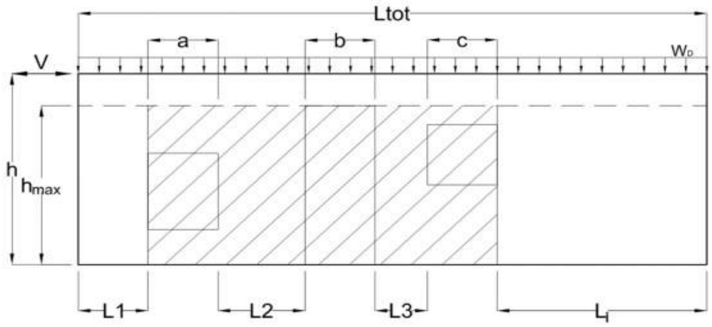
*Refer to Hilti Profis Calculation for Anchorage Design
Hold-down: USE: S/HDU4

Engineer Note: Refer to Sections C2.2 Thru C2.2.4 of AISI S213-07 For additional framing requirements.

Cold-formed Shear Wall Design (ASD)

Code: AISI S213-07

Shear wall Location: Grid 4/5



General Information:

Total applied **ASD** Wind load, V_w =	5945	lbs.		
Wall panel height, h =	13	ft		
Width of panel, L_{tot} =	62.33	ft		
Max height of openings, h_{max} =	8	ft		
Wall Self Weight, w_{self}	12	psf		
Applied Dead Load, w_D	100	plf	Ratio:	
Segment Length, L1 =	10.67	ft	1.2:1	OK
Segment Length, L2 =	3.8	ft	3.4:1	Segment exceeds MAX (h/w) ratio, omitted from Shear wall design
Segment Length, L3 =	3.75	ft	3.5:1	Segment exceeds MAX (h/w) ratio, omitted from Shear wall design
Segment Length, L4 =	12.25	ft	1.1:1	OK
Segment Length, L5 =	0	ft	-	
Segment Length, L6 =	0	ft	-	

In-Plane Shear Design Check:

Sum of Perf. Shear Wall lengths, $\sum L_i$	22.92	ft	$(L_1 + L_2 + L_3 + \ldots)$
Total Length of openings	39.41	ft	(a + b + c + …)
Total Area of Openings, A_o	315.28	ft^2	(a*h + b*h + c*h + …)
Sheathing Area Ratio, r	0.49		$[1/(1 + (Ao/h\sum L_i)]$
Percent Full-Height Sheathing	36.8%		$(\sum L_i/L_{tot})$
Maximum Opening Height Ratio	0.62		(h_{max}/h)
Shear Capacity Adjustment factor, C_a	0.65		$[r/(3-2*r)]*(L_{tot}/\sum L_i)$
ASD Shear wall force, v	**398.2**	plf	$(V/C_a*\sum L_i)$

Note: Area assumed to be total length of openings times max height

Sheathing Type:	1/2" Gypsum Board on one side of wall; Studs max 24" o.c.		
Screw Size:	#10 screws		
Pane Edge/Field Fastener Spacing:	4	inches	
Field Fastener Spacing:	4	inches	
Nominal Shear Strength (R_n)	425.0	plf	Table References
ASD Adjustment Factor,	2.0		AISI S213-07, Section C2.1
Number of Sides Sheathed	2.0		
ASD Shear Capacity, $(R_n/\Omega)*$(# Sides)	**425.0**		**OK**

Chord Force at Shear Wall Ends

Stud Wall Spacing	24	in	
Boundary Chord Force, F	5176.35	lbs.	$(V*h)/(Ca*\sum L)$
Allowable Tension Force, T_a	3416.09	lbs.	$F - 0.6[(w_D+(\text{Wall Wt.})*(h)]*(\sum Li/2)$
Allowable Compression Force, C_a	5432.35	lbs.	$F + [(w_D+(\text{Wall Wt.})*(h)]*(s/2)$

*Refer to AISIWIN Calculation for Chord Design
Sill Plate: USE: 600T125-54
Chords: USE: (2)600S162-54

Sill Plate Anchor Force

Anchor Spacing	32	in
ASD Sill Plate Anchor Force	1061.8	lbs./anchor

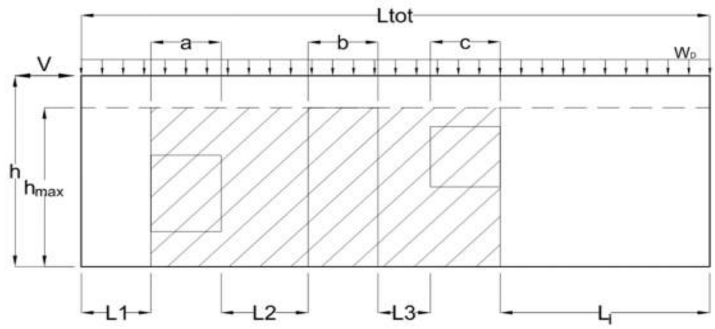
*Refer to Hilti Profis Calculation for Anchorage Design
Hold-down: USE: S/HDU6

Engineer Note: Refer to Sections C2.2 Thru C2.2.4 of AISI S213-07 For additional framing requirements.

Cold-formed Shear Wall Design (ASD)

Code: AISI S213-07

Shear wall Location: Grid 2



General Information:

Total applied **ASD** Wind load, V_w =	7018	lbs.		
Wall panel height, h =	13	ft		
Width of panel, L_{tot} =	22.25	ft		
Max height of openings, h_{max} =	0	ft		
Wall Self Weight, w_{self}	12	psf		
Applied Dead Load, w_D	100	plf	Ratio:	
Segment Length, L1 =	22.25	ft	0.6:1	OK
Segment Length, L2 =	0	ft	-	
Segment Length, L3 =	0	ft	-	
Segment Length, L4 =	0	ft	-	
Segment Length, L5 =	0	ft	-	
Segment Length, L6 =	0	ft	-	

In-Plane Shear Design Check:

Sum of Perf. Shear Wall lengths, $\sum L_i$	22.25	ft	$(L_1 + L_2 + L_3 + \ldots)$
Total Length of openings	0	ft	(a + b + c + ...)
Total Area of Openings, A_o	0.00	ft^2	(a*h + b*h + c*h + ...)
Sheathing Area Ratio, r	1.00		$[1/(1 + (A_o/h\sum L_i)]$
Percent Full-Height Sheathing	100.0%		$(\sum L_i/L_{tot})$
Maximum Opening Height Ratio	0.00		(h_{max}/h)
Shear Capacity Adjustment factor, C_a	1.00		$[r/(3-2*r)]*(L_{tot}/\sum L_i)$
ASD Shear wall force, v	**315.4**	plf	$(V/C_a*\sum L_i)$

Note: Area assumed to be total length of openings times max height

Sheathing Type:	1/2" Gypsum Board on one side of wall; Studs max 24" o.c.		
Screw Size:	#10 screws		
Pane Edge/Field Fastener Spacing:	4	inches	
Field Fastener Spacing:	4	inches	
Nominal Shear Strength (R_n)	425.0	plf	Table References
ASD Adjustment Factor,	2.0		AISI S213-07, Section C2.1
Number of Sides Sheathed	2.0		
ASD Shear Capacity, (R_n/Ω)*(# Sides)	**425.0**		**OK**

Chord Force at Shear Wall Ends

Stud Wall Spacing	24	in	
Boundary Chord Force, F	4100.14	lbs.	$(V*h)/(Ca*\sum L)$
Allowable Tension Force, T_a	2391.34	lbs.	$F - 0.6[(w_D+(\text{Wall Wt.})*(h)]*(\sum Li/2)$
Allowable Compression Force, C_a	4356.14	lbs.	$F + [(w_D+(\text{Wall Wt.})*(h)]*(s/2)$

*Refer to AISIWIN Calculation for Chord Design

Sill Plate: USE: 600T125-54

Chords: USE: (2)600S162-54

Sill Plate Anchor Force

Anchor Spacing	32	in
ASD Sill Plate Anchor Force	841.1	lbs./anchor

*Refer to Hilti Profis Calculation for Anchorage Design

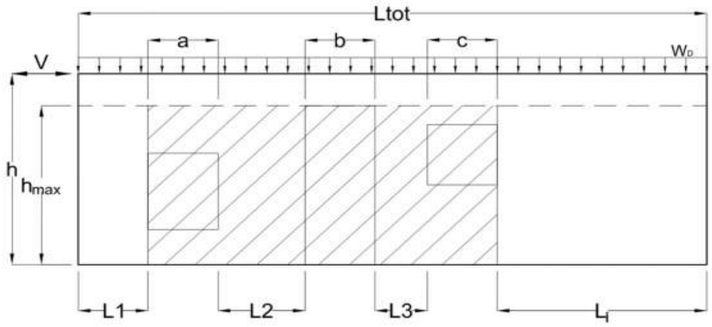
Hold-down: USE: S/HDU6

Engineer Note: Refer to Sections C2.2 Thru C2.2.4 of AISI S213-07 For additional framing requirements.

Cold-formed Shear Wall Design (ASD)

Code: AISI S213-07

Shear wall Location: Grid 1



General Information:

Total applied **ASD** Wind load, V_w =	1072	lbs.		
Wall panel height, h =	13	ft		
Width of panel, L_{tot} =	11	ft		
Max height of openings, h_{max} =	0	ft		
Wall Self Weight, w_{self}	12	psf		
Applied Dead Load, w_D	100	plf	Ratio:	
Segment Length, L1 =	11	ft	1.2:1	OK
Segment Length, L2 =	0	ft	-	
Segment Length, L3 =	0	ft	-	
Segment Length, L4 =	0	ft	-	
Segment Length, L5 =	0	ft	-	
Segment Length, L6 =	0	ft	-	

In-Plane Shear Design Check:

Sum of Perf. Shear Wall lengths, $\sum L_i$	11	ft	$(L_1 + L_2 + L_3 + \ldots)$
Total Length of openings	0	ft	(a + b + c + …)
Total Area of Openings, A_o	0.00	ft^2	(a*h + b*h + c*h + …)
Sheathing Area Ratio, r	1.00		$[1/(1 + (Ao/h\sum L_i)]$
Percent Full-Height Sheathing	100.0%		$(\sum L_i/L_{tot})$
Maximum Opening Height Ratio	0.00		(h_{max}/h)
Shear Capacity Adjustment factor, C_a	1.00		$[r/(3-2^*r)]^*(L_{tot}/\sum L_i)$
ASD Shear wall force, v	**97.5**	plf	$(V/C_a{}^*\sum L_i)$

Note: Area assumed to be total length of openings times max height

Sheathing Type:	1/2" Gypsum Board on one side of wall; Studs max 24" o.c.		
Screw Size:	#10 screws		
Pane Edge/Field Fastener Spacing:	7	inches	
Field Fastener Spacing:	7	inches	
Nominal Shear Strength (R_n)	290.0	plf	Table References
ASD Adjustment Factor,	2.0		AISI S213-07, Section C2.1
Number of Sides Sheathed	2.0		
ASD Shear Capacity, $(R_n/\Omega)^*$(# Sides)	**290.0**		**OK**

Chord Force at Shear Wall Ends

Stud Wall Spacing	24	in	
Boundary Chord Force, F	1267.04	lbs.	$(V^*h)/(Ca^*\sum L)$
Allowable Tension Force, T_a	422.24	lbs.	$F - 0.6[(w_D+(\text{Wall Wt.})^*(h)]^*(\sum Li/2)$
Allowable Compression Force, C_a	1523.04	lbs.	$F + [(w_D+(\text{Wall Wt.})^*(h)]^*(s/2)$

*Refer to AISIWIN Calculation for Chord Design
Sill Plate: USE: 600T125-54
Chords: USE: (2)600S162-54

Sill Plate Anchor Force

Anchor Spacing	32	in
ASD Sill Plate Anchor Force	259.9	lbs./anchor

*Refer to Hilti Profis Calculation for Anchorage Design
Hold-down: USE: S/HDU4

<u>Engineer Note:</u> Refer to Sections C2.2 Thru C2.2.4 of AISI S213-07 For additional framing requirements.