BSE STRUCTURAL ENGINEERS

Project Balderston Auto | Project No. 20-467
Calc. By RJS | Checked By JH | Date 1/11/2021

Structural Calculations For:

Balderston Auto

Lee's Summit, MO

STATE OF MISSOURI
JUSTIN E. BURGOON
NUMBER PE-2014015015
2021-03-01
PROFESSIONAL ENGINEER

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 **15 psf**

LIVE LOADING

LIVE LOAD CONSTRUCTION

Roof Live Load **20 psf**

IBC 2012, Table 1607.1

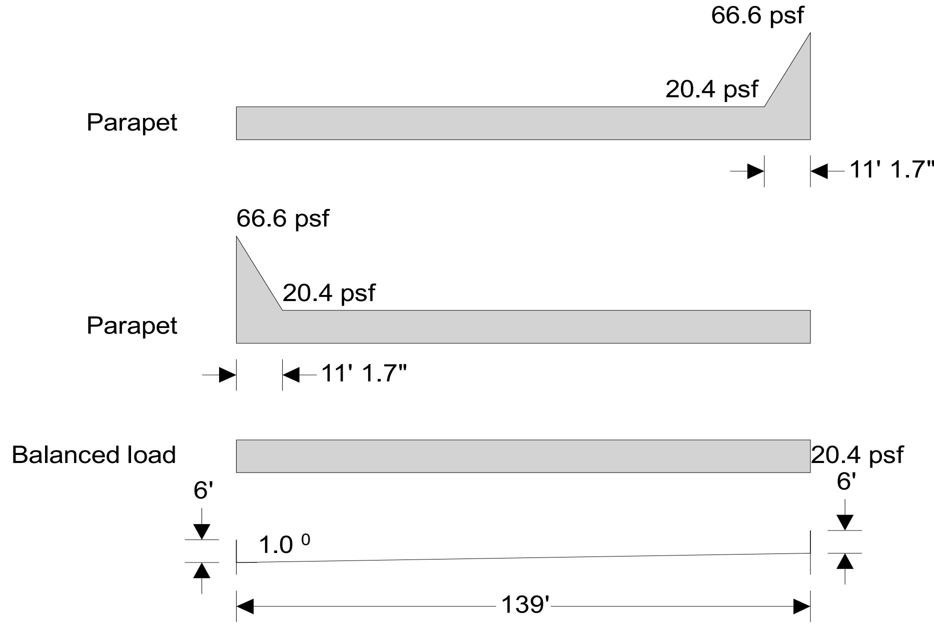
SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.09

Building details	
Roof type	Monopitch
Width of roof	b = **139.00** ft
Slope of roof 1	α = **1.00** deg
Ground snow load	
Ground snow load (Figure 7-1)	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 7-2)	Partially exposed
Exposure factor (Table 7-2)	C_e = **1.00**
Thermal condition (Table 7-3)	Structures kept just above freezing
Thermal factor (Table 7-3)	C_t = **1.10**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **20.00** lb/ft^2
Rain on snow surcharge (Sect 7.10)	p_{f_sur} = **5.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g + p_{f\_sur}$ = **20.40** lb/ft^2
Unbalanced flat roof snow load (Sect 7.3)	$p_{f\_unbal} = \max(p_{f\_min}, 0.7 \times C_e \times C_t \times I_s \times p_g)$ = **20.00** lb/ft^2
Cold roof slope factor (C_t > 1.0)	
Roof surface type	Slippery
Ventilation	Ventilated
Thermal resistance (R-value)	R = **30.00** °F h ft^2 / Btu
Roof slope factor Fig 7-2b (solid line)	C_s = **1.00**
Monoslope	
Sloped roof snow load (Cl.7.4)	$p_s = \max(C_s \times p_f, p_{f\_min})$ = **20.40** lb/ft^2
Left parapet	
Balanced snow load height	$h_b = p_f \times C_s / \gamma$ = **1.23** ft
Height of left parapet	h_{pptL} = **6.00** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **4.77** ft
Length of roof - left parapet	$l_{u\_pptL} = b_1$ = **139.00** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptL}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **2.78** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **2.78** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **11.14** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **46.23** lb/ft^2
Right parapet	
Height of right parapet	h_{pptR} = **6.00** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **4.77** ft
Length of roof - right parapet	$l_{u\_pptR} = b_1$ = **139.00** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = 0.75 \times (0.43 \times (\max(20\text{ ft}, l_{u\_pptR}) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **2.78** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **2.78** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **11.14** ft

Drift surcharge load - right parapet $p_{d\_pptR} = h_{d\_pptR} \times \gamma =$ **46.23** lb/ft^2



Roof elevation

SEISMIC FORCES (ASCE 7-10)

Tedds calculation version 3.1.00

Site parameters

Site class	D
Mapped acceleration parameters (Section 11.4.1)	
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

Risk category (Table 1.5-1) 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 = **17.2** 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)	$T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$ = **0.169** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.169** sec
Long-period transition period	T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	B_BUILDING_FRAME_SYSTEMS
	18. Ordinary reinforced masonry shear walls
Response modification factor (Table 12.2-1)	R = **2**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-3)	$C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0528**
Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e))$ = **0.3220**
Minimum (Eq 12.8-5)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0100**
Seismic response coefficient	C_s = **0.0528**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **375.0** kips

Seismic response coefficient C_s = **0.0528**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **19.8** kips

Vertical distribution of seismic forces (Sect 12.8.3)

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

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

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x
1	17.2;	375.0;	1.00;	1.000;	19.8

SNOW LOADING (ASCE7-10)

Tedds calculation version 1.0.06

Building details

Roof type	Flat
Width of roof	b = **220.00** ft

Ground snow load

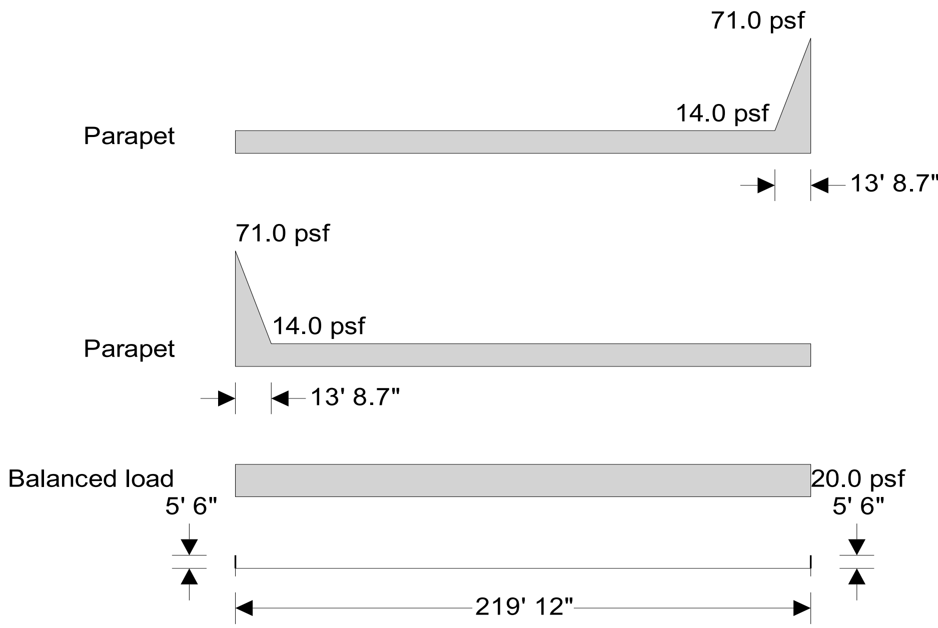
Ground snow load	p_g = **20.00** lb/ft^2
Density of snow	γ = min(0.13 × p_g / 1ft + 14lb/ft^3, 30lb/ft^3) = **16.60** lb/ft^3
Terrain type	C
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	C_e = **1.00**
Thermal condition (Table 7-3)	All
Thermal factor (Table 7-3)	C_t = **1.00**
Importance category (Table 1-1)	II
Importance factor (Table 7-4)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	p_{f_min} = I_s × p_g = **20.00** lb/ft^2
Flat roof snow load (Sect 7.3)	p_f = 0.7 × C_e × C_t × I_s × p_g = **14.00** lb/ft^2

Left parapet

Balanced snow load height	h_b = p_f / γ = **0.84** ft
Height of left parapet	h_{pptL} = **5.50** ft
Height from balance load to top of left parapet	h_{c_pptL} = h_{pptL} - h_b = **4.66** ft
Length of roof - left parapet	l_{u_pptL} = b = **220.00** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL}$ = 0.75 × (0.43 × (max(20 ft, l_{u_pptL}) × 1ft^2)$^{1/3}$ × (p_g / 1lb/ft^2 + 10)$^{1/4}$ - 1.5ft) = **3.43** ft
Drift height - left parapet	h_{d_pptL} = min($h_{d\_l\_pptL}$, h_{pptL} - h_b) = **3.43** ft
Drift width	W_{d_pptL} = min(4 × h_{d_pptL}, 8 × (h_{pptL} - h_b)) = **13.73** ft
Drift surcharge load - left parapet	p_{d_pptL} = h_{d_pptL} × γ = **56.96** lb/ft^2

Right parapet

Height of right parapet	h_{pptR} = **5.50** ft
Height from balance load to top of right parapet	h_{c_pptR} = h_{pptR} - h_b = **4.66** ft
Length of roof - right parapet	l_{u_pptR} = b = **220.00** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR}$ = 0.75 × (0.43 × (max(20 ft, l_{u_pptR}) × 1ft^2)$^{1/3}$ × (p_g / 1lb/ft^2 + 10)$^{1/4}$ - 1.5ft) = **3.43** ft
Drift height - right parapet	h_{d_pptR} = min($h_{d\_l\_pptR}$, h_{pptR} - h_b) = **3.43** ft
Drift width	W_{d_pptR} = min(4 × h_{d_pptR}, 8 × (h_{pptR} - h_b)) = **13.73** ft
Drift surcharge load - right parapet	p_{d_pptR} = h_{d_pptR} × γ = **56.96** lb/ft^2



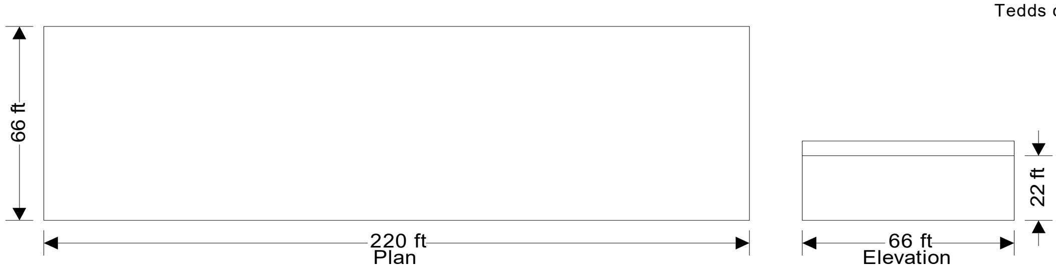
Roof elevation

WIND LOADING

In accordance with ASCE7-10

Using the components and cladding design method

Tedds calculation version 2.1.05



Building data

Type of roof	Flat
Length of building	b = **220.00** ft
Width of building	d = **66.00** ft
Height to eaves	H = **22.00** ft
Height of parapet	h_p = **5.00** ft
Mean height	h = **22.00** ft

General wind load requirements

Basic wind speed	V = **115.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	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 (T.30.3-1)	K_z = **0.92**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **26.4** psf

Velocity pressure at parapet

Velocity pressure coefficient (T.30.3-1)	K_z = **0.96**
Velocity pressure	$q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$ = **27.5** psf

Peak velocity pressure for internal pressure

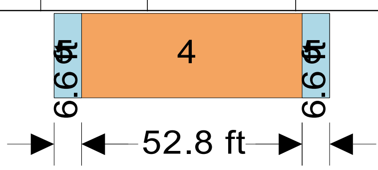
Peak velocity pressure – internal (as roof press.)	q_i = **26.36** psf

Equations used in tables

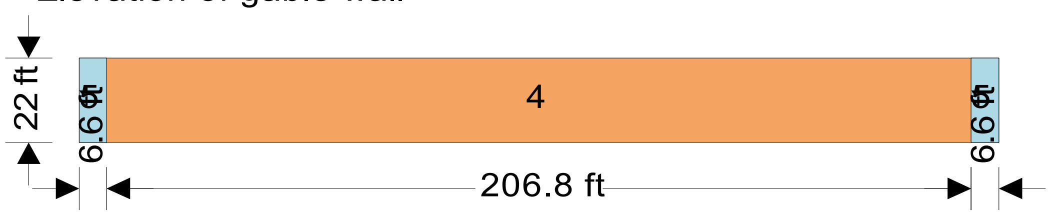
Net pressure	$p = q_h \times [GC_p - GC_{pi}]$
Parapet net pressure	$p = q_p \times [GC_p - GC_{pi\_p}]$

Components and cladding pressures - Wall (Table 30.4-1 and Figure 30.4-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	4	-	-	10.0	0.90	-0.99	28.5	-30.8
50sf	4	-	-	50.0	0.79	-0.88	25.5	-27.9
200sf	4	-	-	200.0	0.69	-0.78	23.0	-25.4
>500sf	4	-	-	500.0	0.63	-0.72	21.4	-23.7
<10sf	5	-	-	10.0	0.90	-1.26	28.5	-38.0
50sf	5	-	-	50.0	0.79	-1.04	25.5	-32.1
200sf	5	-	-	200.0	0.69	-0.85	23.0	-27.1
>500sf	5	-	-	500.0	0.63	-0.72	21.4	-23.7
20 sff	5	-	-	20.0	0.85	-1.16	27.2	-35.4
20 sff	4	-	-	20.0	0.85	-0.94	27.2	-29.6



Elevation of gable wall



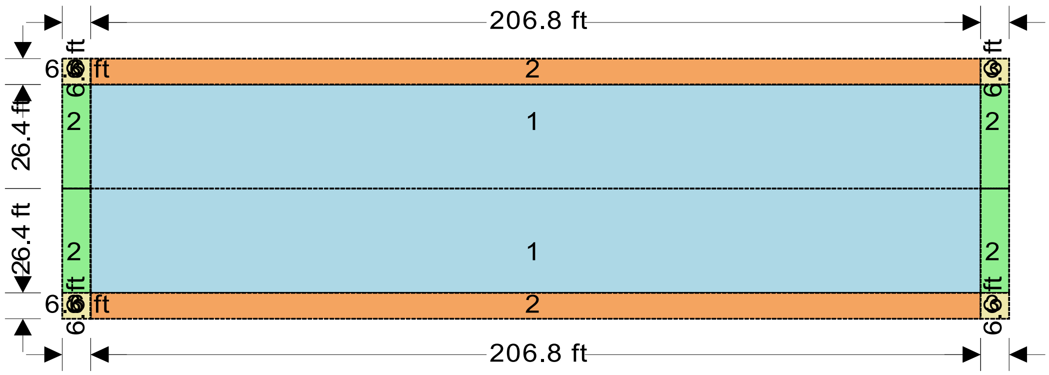
Elevation of side wall

Components and cladding pressures - Roof (Figure 30.4-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<10sf	1	-	-	10.0	0.30	-1.00	12.7 #	-31.1
25sf	1	-	-	25.0	0.26	-0.96	11.6 #	-30.1
50sf	1	-	-	50.0	0.23	-0.93	10.8 #	-29.3
>100sf	1	-	-	100.0	0.20	-0.90	10.0 #	-28.5
<10sf	2	-	-	10.0	0.90	-1.80	28.5	-52.2
25sf	2	-	-	25.0	0.84	-1.52	26.8	-44.9
50sf	2	-	-	50.0	0.79	-1.31	25.5	-39.3
>100sf	2	-	-	100.0	0.74	-1.10	24.3	-33.7
<10sf	3	-	-	10.0	0.90	-1.80	28.5	-52.2
25sf	3	-	-	25.0	0.84	-1.52	26.8	-44.9
50sf	3	-	-	50.0	0.79	-1.31	25.5	-39.3

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
>100sf	3	-	-	100.0	0.74	-1.10	24.3	-33.7

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



Plan on roof

Summary

The gravity structure system of the project referenced above consists primarily of steel joists and girders, load-bearing CMU walls and steel columns. Loading criteria and depth criteria are shown for all roof framing members. These members are to be designed by the Project joist manufacturer and reviewed by BSE. Interior steel framing is supported by steel columns. The locations of all roof framing and columns are indicated on the structural framing plans.

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

STEEL MEMBER ANALYSIS & DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

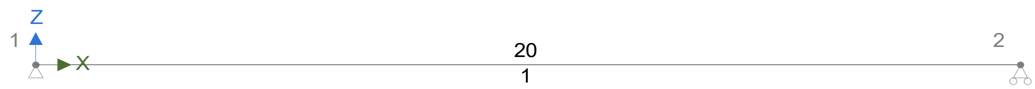
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

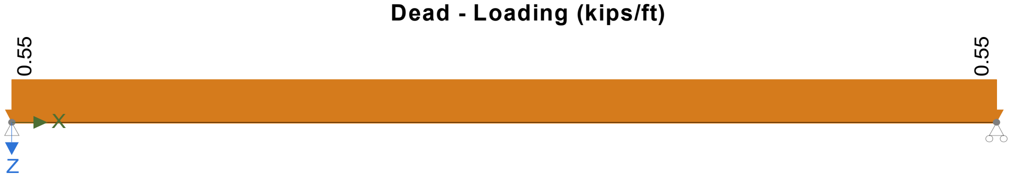
Geometry (ft) - Steel (AISC)



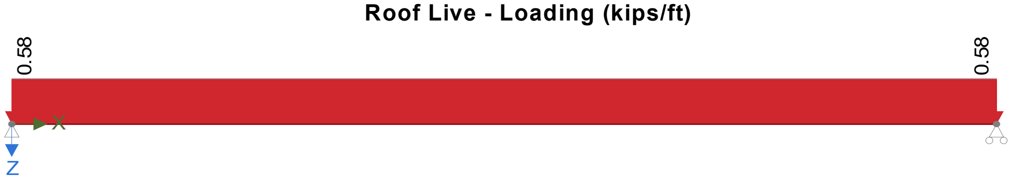
Loading

Self weight included

Dead - Loading (kips/ft)



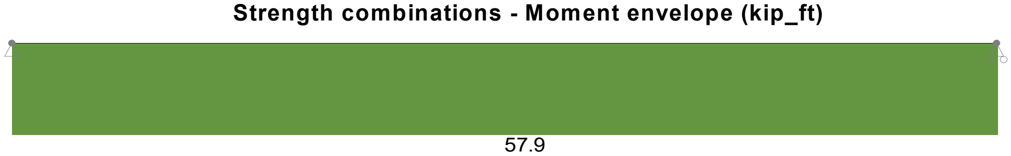
Roof Live - Loading (kips/ft)



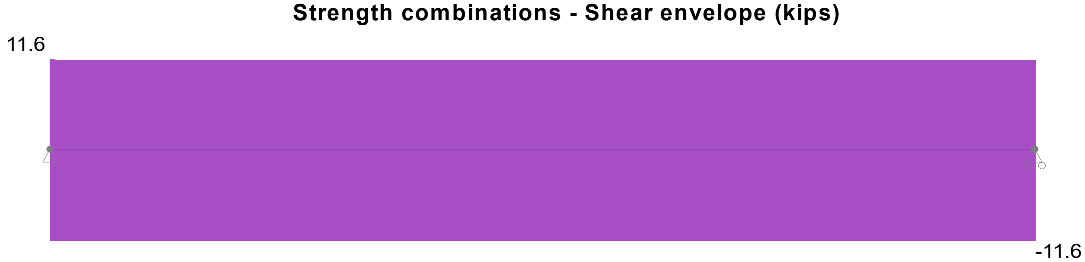
Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



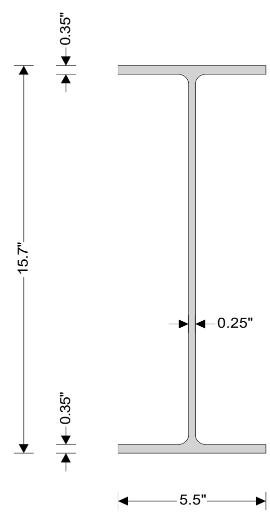
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Beam design

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



W 16x26 (AISC 15th Edn (v15.0))
Section depth, d, 15.7 in
Section breadth, b_f, 5.5 in
Weight of section, Weight, 26 lbf/ft
Flange thickness, t_f, 0.345 in
Web thickness, t_w, 0.25 in
Area of section, A, 7.7 in^2
Radius of gyration about x-axis, r_x, 6.26 in
Radius of gyration about y-axis, r_y, 1.12 in
Elastic section modulus about x-axis, S_x, 38.4 in^3
Elastic section modulus about y-axis, S_y, 3.49 in^3
Plastic section modulus about x-axis, Z_x, 44.2 in^3
Plastic section modulus about y-axis, Z_y, 5.48 in^3
Second moment of area about x-axis, I_x, 301 in^4
Second moment of area about y-axis, I_y, 9.59 in^4

Lateral restraint

Top flange has lateral restraint at supports plus 6ft, 12ft & 18ft

Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **11.6** kips

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

Web plate buckling coefficient k_v = **5**

$(d - 2 \times k) / t_w > 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-3 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **117.8** kips

Safety factor Ω_v = 1.67

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **70.5** kips

$V_{r,x} / V_{c,x}$ = **0.164**

PASS - Allowable shear strength exceeds required shear strength

Check design 10ft along span

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **57.9** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **184.2** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length L_b = **6** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)}$ = **3.956** ft

Distance between flange centroids h_o = **15.4** in

c = **1**

r_{ts} = **1.38** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)})}$ = **11.167** ft

Moment at quarter point of segment M_A = **54.2** kips_ft

Moment at center-line of segment M_B = **57.3** kips_ft

Moment at three quarter point of segment M_C = **57.7** kips_ft

Maximum moment in segment M_{max} = **57.9** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 \times M_{max} / (2.5 \times M_{max} + 3 \times M_A + 4 \times M_B + 3 \times M_C)$ = **1.019**

Nominal flexural strength for lateral-torsional buckling - eq F2-2

$M_{n,ltb,x} = \min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x})$

= **166.8** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,yld,x}, M_{n,ltb,x})$ = **166.8** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **99.9** kips_ft

$M_{r,x} / M_{c,x}$ = **0.580**

PASS - Allowable flexural strength exceeds required flexural strength

Check design 10ft along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.493** in

Allowable deflection $\delta_{x,Allowable} = L_{m1\_s1} / 360$ = **0.667** in

$\delta_x / \delta_{x,Allowable}$ = **0.74**

PASS - Allowable deflection exceeds design deflection

STEEL MEMBER ANALYSIS & DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

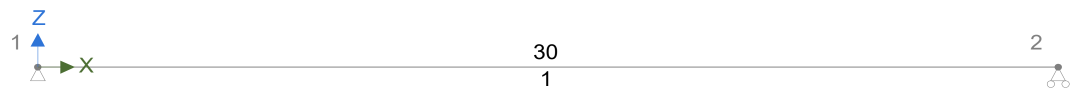
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

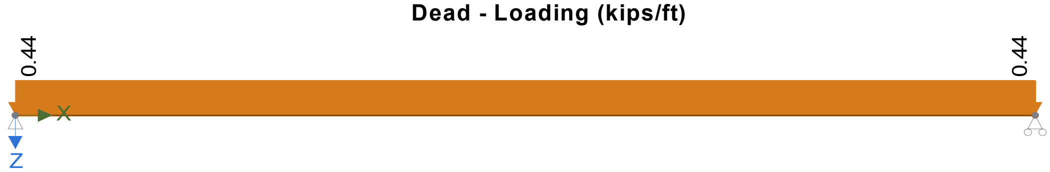
Geometry (ft) - Steel (AISC)



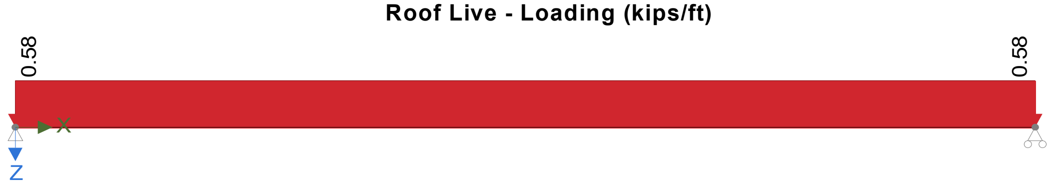
Loading

Self weight included

Dead - Loading (kips/ft)



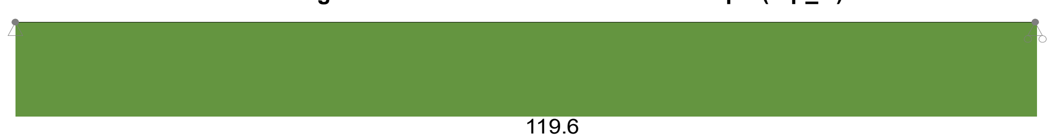
Roof Live - Loading (kips/ft)



Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

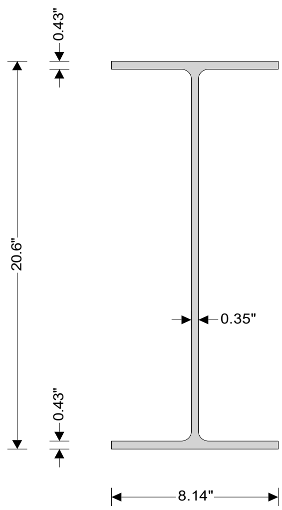
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Beam design

Section details

Section type	W 21x48 (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



W 21x48 (AISC 15th Edn (v15.0))
Section depth, d, 20.6 in
Section breadth, b_f, 8.14 in
Weight of section, Weight, 48 lbf/ft
Flange thickness, t_f, 0.43 in
Web thickness, t_w, 0.35 in
Area of section, A, 14.1 in^2
Radius of gyration about x-axis, r_x, 8.24 in
Radius of gyration about y-axis, r_y, 1.66 in
Elastic section modulus about x-axis, S_x, 93 in^3
Elastic section modulus about y-axis, S_y, 9.52 in^3
Plastic section modulus about x-axis, Z_x, 107 in^3
Plastic section modulus about y-axis, Z_y, 14.9 in^3
Second moment of area about x-axis, I_x, 959 in^4
Second moment of area about y-axis, I_y, 38.7 in^4

Lateral restraint

Top flange has lateral restraint at supports plus 6ft, 12ft & 18ft

Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **9.47**	
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**	Noncompact

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

Width to thickness ratio	$(d - 2 \times k) / t_w$ = **53.54**	
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 noncompact in flexure

Check design at start of span

Design of members for shear - Chapter G

Required shear strength	$V_{r,x}$ = **15.9** kips
Web area	$A_w = d \times t_w$ = **7.21** in^2
Web plate buckling coefficient	k_v = **5**
	$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$
Web shear coefficient - eq G2-2	C_v = **1.000**
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **216.3** kips
Safety factor	Ω_v = 1.50
Allowable shear strength	$V_{c,x} = V_{n,x} / \Omega_v$ = **144.2** kips
	$V_{r,x} / V_{c,x}$ = **0.111**

PASS - Allowable shear strength exceeds required shear strength

Check design 15ft along span

Design of members for flexure - Chapter F

Required flexural strength	$M_{r,x}$ = **119.6** kips_ft
Plastic moment - eq F2-1	$M_{p,x} = F_y \times Z_x$ = **445.8** kips_ft

Lateral-torsional buckling - Section F3.1

Unbraced length	L_b = **6** ft
Limiting unbraced length for yielding - eq F2-5	$L_p = 1.76 \times r_y \times \sqrt{(E / F_y)}$ = **5.863** ft
Distance between flange centroids	h_o = **20.2** in
	c = **1**
	r_{ts} = **2.05** in
Limiting unbraced length for inelastic LTB - eq F2-6	$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2))}}$ = **16.548** ft
Moment at quarter point of segment	M_A = **118.4** kips_ft
Moment at center-line of segment	M_B = **119.6** kips_ft
Moment at three quarter point of segment	M_C = **118.4** kips_ft
Maximum moment in segment	M_{max} = **119.6** kips_ft
LTB modification factor - eq F1-1	$C_b = 12.5 \times M_{max} / (2.5 \times M_{max} + 3 \times M_A + 4 \times M_B + 3 \times M_C)$ = **1.005**

Nominal flexural strength for lateral-torsional buckling - eq F2-2

$M_{n,ltb,x} = \min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **445.7** kips_ft

Compression flange local buckling - Section F3.2

$\lambda = b_f / (2 \times t_f)$ = **9.465**

Nominal flexural strength for compression flange local buckling - eq F3-1

$M_{n,flb,x} = M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (\lambda - \lambda_{pff}) / (\lambda_{rff} - \lambda_{pff})$ = **442.2** kips_ft

Allowable flexural strength - F1

Nominal flexural strength	$M_{n,x} = \min(M_{n,ltb,x}, M_{n,flb,x})$ = **442.2** kips_ft
Allowable flexural strength	$M_{c,x} = M_{n,x} / \Omega_b$ = **264.8** kips_ft
	$M_{r,x} / M_{c,x}$ = **0.452**

PASS - Allowable flexural strength exceeds required flexural strength

Check design 15ft along span

Design of members for x-x axis deflection

Maximum deflection	δ_x = **0.714** in
Allowable deflection	$\delta_{x,Allowable} = L_{m1\_s1} / 360$ = **1** in

δ_x / $\delta_{x,Allowable}$ = **0.714**

PASS - Allowable deflection exceeds design deflection

STEEL MEMBER ANALYSIS & DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

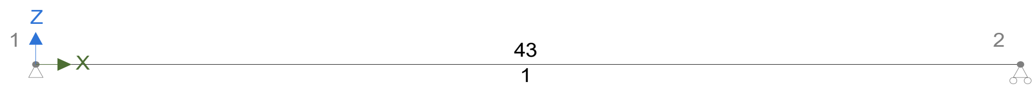
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

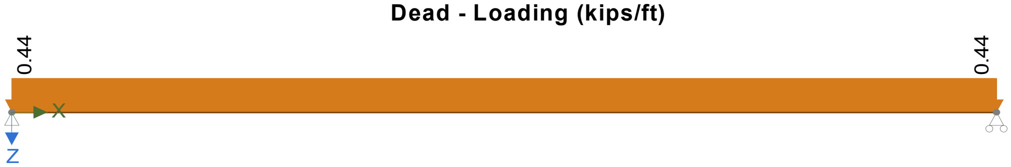
Geometry (ft) - Steel (AISC)



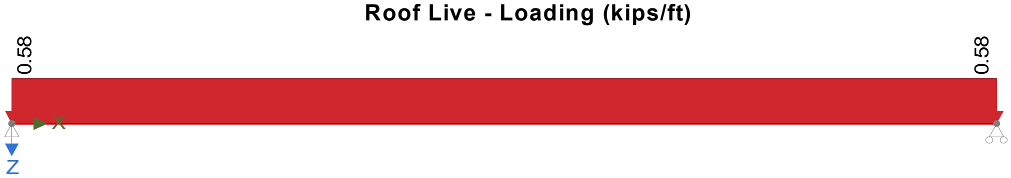
Loading

Self weight included

Dead - Loading (kips/ft)



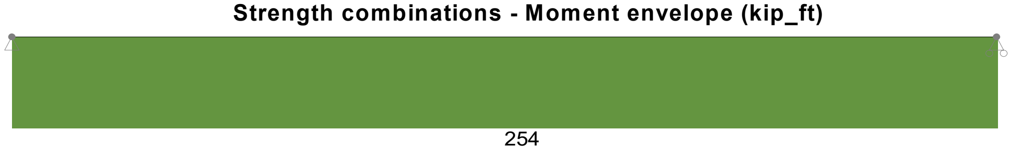
Roof Live - Loading (kips/ft)



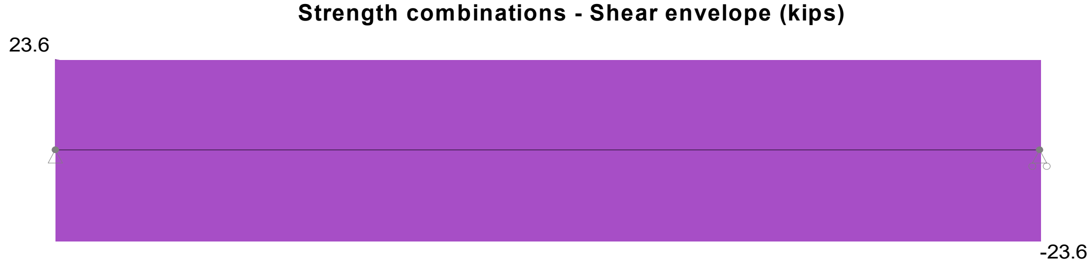
Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



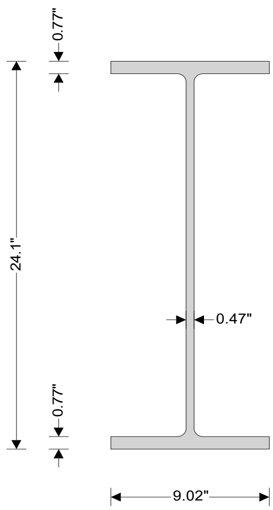
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Beam design

Section details

Section type	W 24x84 (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



W 24x84 (AISC 15th Edn (v15.0))
Section depth, d, 24.1 in
Section breadth, b_f, 9.02 in
Weight of section, Weight, 84 lbf/ft
Flange thickness, t_f, 0.77 in
Web thickness, t_w, 0.47 in
Area of section, A, 24.7 in^2
Radius of gyration about x-axis, r_x, 9.79 in
Radius of gyration about y-axis, r_y, 1.95 in
Elastic section modulus about x-axis, S_x, 196 in^3
Elastic section modulus about y-axis, S_y, 20.9 in^3
Plastic section modulus about x-axis, Z_x, 224 in^3
Plastic section modulus about y-axis, Z_y, 32.6 in^3
Second moment of area about x-axis, I_x, 2370 in^4
Second moment of area about y-axis, I_y, 94.4 in^4

Lateral restraint

Top flange has full lateral restraint

Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **5.86**	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08**	Compact

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

Width to thickness ratio	$(d - 2 \times k) / t_w$ = **45.87**	
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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **23.6** kips

Web area $A_w = d \times t_w$ = **11.327** in^2

Web plate buckling coefficient k_v = **5**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **339.8** kips

Safety factor Ω_v = 1.50

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **226.5** kips

$V_{r,x} / V_{c,x}$ = **0.104**

PASS - Allowable shear strength exceeds required shear strength

Check design 21ft 6in along span

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **254** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **933.3** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **933.3** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **558.9** kips_ft

$M_{r,x} / M_{c,x}$ = **0.454**

PASS - Allowable flexural strength exceeds required flexural strength

Check design 21ft 6in along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **1.254** in

Allowable deflection $\delta_{x,Allowable} = L_{m1\_s1} / 360$ = **1.433** in

$\delta_x / \delta_{x,Allowable}$ = **0.875**

PASS - Allowable deflection exceeds design deflection

FLEXURAL ANALYSIS OF ANGLES

AISC "F10"

b d t
L 5 x 5 x 3/8 Fy = 36 ksi

Cb = 1.00 (conservatively)
L = 48.00 in
Iz = 3.55 in^4
rz = 0.99 in
t = 0.38 in
βw = 0.00 (see table C-F10.1 in commentary)
S = 2.41 in^3 (in direction of bending)

1.) YEILDING

Mn/Ω =	77.93	K-in	=	6.49401	K-ft	Eq. F10-1

2.) LATERAL TORSIONAL BUCKLING (using geometric axis)

Is toe of angle in compression or tension (C or T)? T

Me =	4254.35	K-in	=	354.53	K-ft	Eq. F10-4a & Eq. F10-4b
Mn/Ω =	73.59	K-in	=	6.13228	K-ft	Eq. F10-2 & Eq. F10-3

3.) LATERAL TORSIONAL BUCKLING (using major principal axis)

Me =	977.05078	K-in	=	81.4209	K-ft	Eq. F10-6
Mn/Ω =	66.84	K-in	=	5.56982	K-ft	Eq. F10-2 & Eq. F10-3

4.) LEG LOCAL BUCKLING NEED THIS CALC

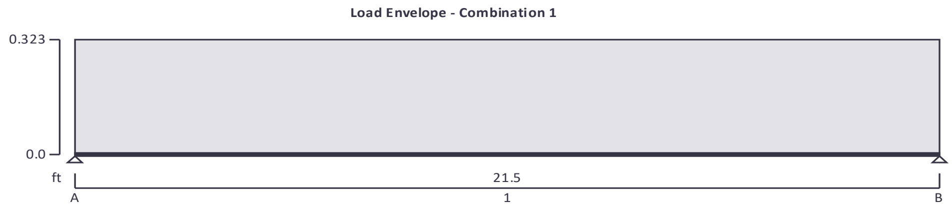
Mn/Ω =	67.412415	K-in	=	5.6177	K-ft	Eq. F10-7
Mn/Ω =	133.71171	K-in	=	11.1426	K-ft	Eq. F10-8

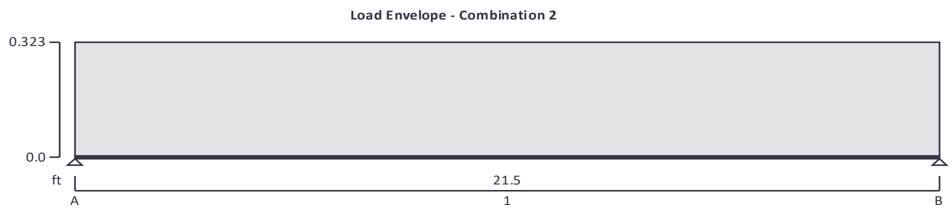
Mn/Ω = 5.57 K-ft = 66.84 K-in

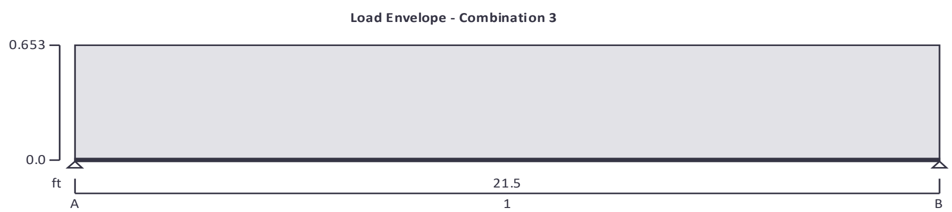
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

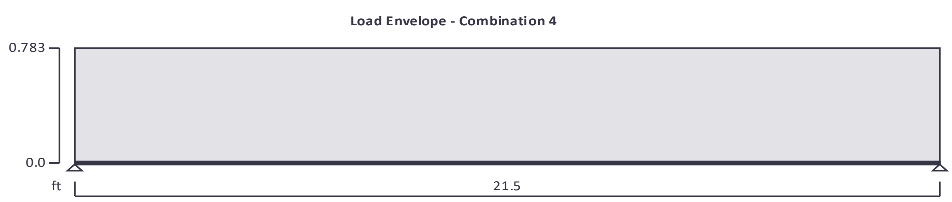
In accordance with AISC360-16 using the ASD method

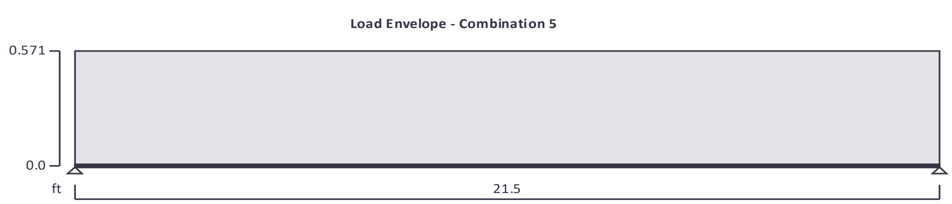
Tedds calculation version 3.0.14







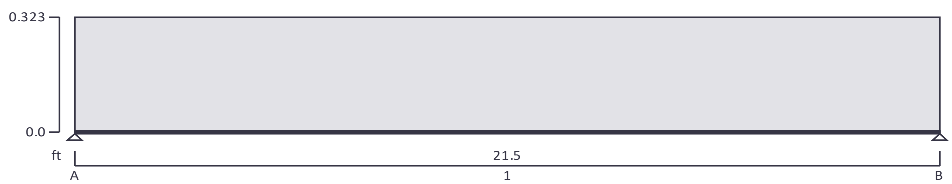




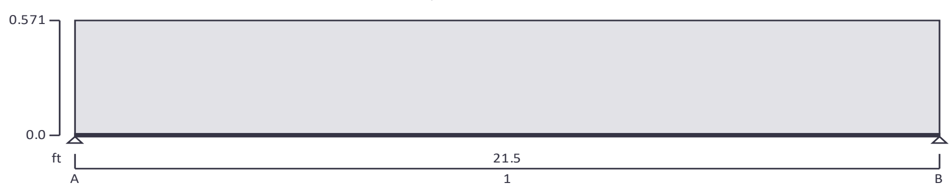
Load Envelope - Combination 6

Load Envelope - Combination 7

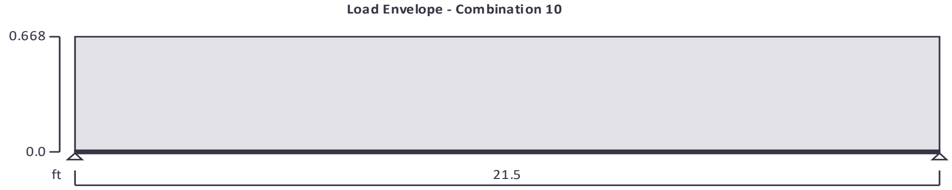
Load Envelope - Combination 8

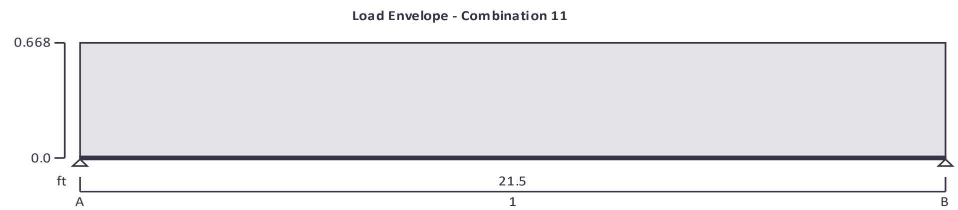


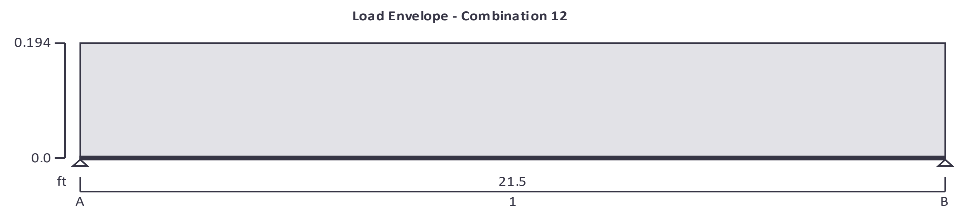
Load Envelope - Combination 9

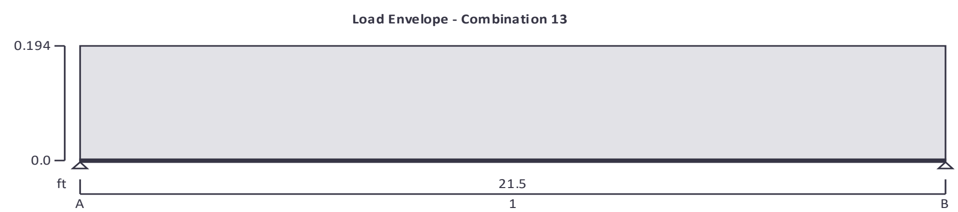


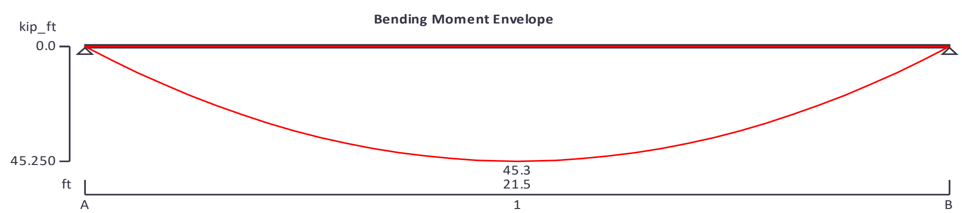
Load Envelope - Combination 10

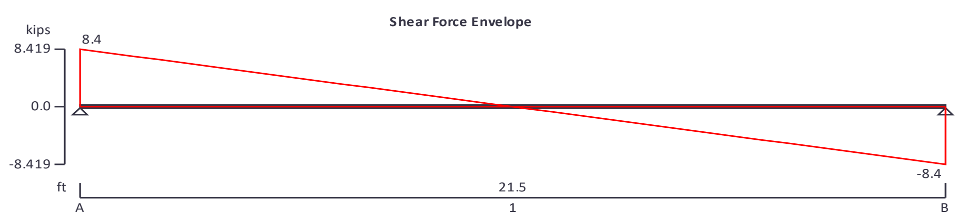












Support conditions

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

Applied loading

Beam loads	Dead full UDL 0.297 kips/ft
	Snow full UDL 0.46 kips/ft
	Roof live full UDL 0.33 kips/ft
	Dead self weight of beam × 1

Load combinations

Load combination 1 - D	Support A	Dead × 1.00
		Dead × 1.00
	Support B	Dead × 1.00
Load combination 2 - D + L	Support A	Dead × 1.00
		Live × 1.00
		Dead × 1.00
		Live × 1.00
	Support B	Dead × 1.00
		Live × 1.00
Load combination 3 - D + Lr	Support A	Dead × 1.00
		Roof live × 1.00
		Dead × 1.00
		Roof live × 1.00
	Support B	Dead × 1.00
		Roof live × 1.00
Load combination 4 - D + S	Support A	Dead × 1.00
		Snow × 1.00
		Dead × 1.00
		Snow × 1.00
	Support B	Dead × 1.00
		Snow × 1.00
Load combination 5 - D+0.75L+0.75Lr	Support A	Dead × 1.00
		Live × 0.75
		Roof live × 0.75
		Dead × 1.00
		Live × 0.75
		Roof live × 0.75
	Support B	Dead × 1.00
		Live × 0.75
		Roof live × 0.75
Load combination 6 - D+0.75L+0.75S	Support A	Dead × 1.00
		Live × 0.75
		Snow × 0.75
		Dead × 1.00
		Live × 0.75
		Snow × 0.75
	Support B	Dead × 1.00
		Live × 0.75
		Snow × 0.75

Load combination 7 - D+ 0.6W	Support A	Dead × 1.00
		Wind × 0.60
		Dead × 1.00
		Wind × 0.60
	Support B	Dead × 1.00
		Wind × 0.60
Load combination 8 - D+0.7E	Support A	Dead × 1.00
		Seismic × 0.70
		Dead × 1.00
		Seismic × 0.70
	Support B	Dead × 1.00
		Seismic × 0.70
Load combination 9 - D+0.75L+0.75(0.6W)+0.75Lr	Support A	Dead × 1.00
		Live × 0.75
		Roof live × 0.75
		Wind × 0.45
		Dead × 1.00
		Live × 0.75
		Roof live × 0.75
		Wind × 0.45
	Support B	Dead × 1.00
		Live × 0.75
		Roof live × 0.75
		Wind × 0.45
Load combination 10 - D+0.75L+0.75(0.6W)+0.75S	Support A	Dead × 1.00
		Live × 0.75
		Snow × 0.75
		Wind × 0.45
		Dead × 1.00
		Live × 0.75
		Snow × 0.75
		Wind × 0.45
	Support B	Dead × 1.00
		Live × 0.75
		Snow × 0.75
		Wind × 0.45
Load combination 11 - D+0.75L+0.75(0.7E)+0.75S	Support A	Dead × 1.00
		Live × 0.75
		Snow × 0.75
		Seismic × 0.53
		Dead × 1.00
		Live × 0.75
		Snow × 0.75
		Seismic × 0.53
	Support B	Dead × 1.00

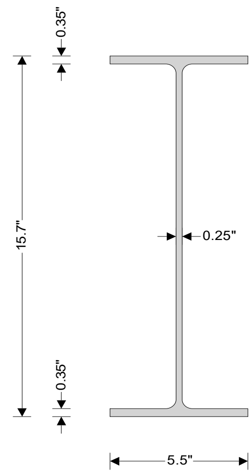
		Live × 0.75
		Snow × 0.75
		Seismic × 0.53
Load combination 12 - 0.6D + 0.6W	Support A	Dead × 0.60
		Wind × 0.60
		Dead × 0.60
		Wind × 0.60
	Support B	Dead × 0.60
		Wind × 0.60
Load combination 13 - 0.6D + 0.7E	Support A	Dead × 0.60
		Seismic × 0.70
		Dead × 0.60
		Seismic × 0.70
	Support B	Dead × 0.60
		Seismic × 0.70

Analysis results

Maximum moment	M_{max} = **45.3** kips_ft	M_{min} = **0** kips_ft
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **40.2** kips_ft	$M_{s1\_seg1\_min}$ = **0** kips_ft
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **45.3** kips_ft	$M_{s1\_seg2\_min}$ = **0** kips_ft
Maximum moment span 1 segment 3	$M_{s1\_seg3\_max}$ = **40.2** kips_ft	$M_{s1\_seg3\_min}$ = **0** kips_ft
Maximum shear	V_{max} = **8.4** kips	V_{min} = **-8.4** kips
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **8.4** kips	$V_{s1\_seg1\_min}$ = **0** kips
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **2.8** kips	$V_{s1\_seg2\_min}$ = **-2.8** kips
Maximum shear span 1 segment 3	$V_{s1\_seg3\_max}$ = **0** kips	$V_{s1\_seg3\_min}$ = **-8.4** kips
Deflection segment 4	δ_{max} = **0.6** in	δ_{min} = **0** in
Maximum reaction at support A	R_{A_max} = **8.4** kips	R_{A_min} = **2.1** kips
Unfactored dead load reaction at support A	R_{A_Dead} = **3.5** kips	
Unfactored roof live load reaction at support A	$R_{A\_Roof\ live}$ = **3.5** kips	
Unfactored snow load reaction at support A	R_{A_Snow} = **4.9** kips	
Maximum reaction at support B	R_{B_max} = **8.4** kips	R_{B_min} = **2.1** kips
Unfactored dead load reaction at support B	R_{B_Dead} = **3.5** kips	
Unfactored roof live load reaction at support B	$R_{B\_Roof\ live}$ = **3.5** kips	
Unfactored snow load reaction at support B	R_{B_Snow} = **4.9** 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



Safety factors

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

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

Safety factor for compression Ω_c = **1.67**

Safety factor for flexure Ω_b = **1.67**

Lateral bracing

Span 1 has lateral bracing at supports plus third points

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 $V_r = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **8.419** 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

Safety factor for shear Ω_v = **1.67**

Allowable shear strength $V_c = V_n / \Omega_v$ = **70.509** kips

PASS - Allowable shear strength exceeds required shear strength

Design of members for flexure in the major axis at span 1 segment 2 - Chapter F

Required flexural strength $M_r = \max(\mathrm{abs}(M_{s1\_seg2\_max}), \mathrm{abs}(M_{s1\_seg2\_min}))$ = **45.25** 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

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{s1\_seg2} =$ **86** in

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{[E / F_y]} =$ **47.473** in

Distance between flange centroids $h_o = d - t_f =$ **15.355** in

$c =$ **1**

$r_{ts} = \sqrt{[\sqrt{(I_y \times C_w)} / S_x]} =$ **1.385** in

Limiting unbraced length for inelastic LTB - eq F2-6

$L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{[(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)}]} =$ **134.473** in

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

Moment at quarter point of segment $M_A =$ **43.993** kips_ft

Moment at center-line of segment $M_B =$ **45.250** kips_ft

Moment at three quarter point of segment $M_C =$ **43.993** kips_ft

Maximum moment in segment $M_{abs} =$ **45.250** kips_ft

Lateral torsional buckling modification factor - eq F1-1 $C_b = 12.5 \times M_{abs} / [2.5 \times M_{abs} + 3 \times M_A + 4 \times M_B + 3 \times M_C] =$ **1.014**

Nominal flexural strength for lateral torsional buckling - eq F2-2 $M_{nltb} = C_b \times [M_p - (M_p - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)] =$ **154.265** kips_ft

Nominal flexural strength $M_n = \min(M_{nyld}, M_{nltb}) =$ **154.265** kips_ft

Allowable flexural strength $M_c = M_n / \Omega_b =$ **92.374** kips_ft

PASS - Allowable flexural strength exceeds required flexural strength

Design of members for vertical deflection

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

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **0.717** in

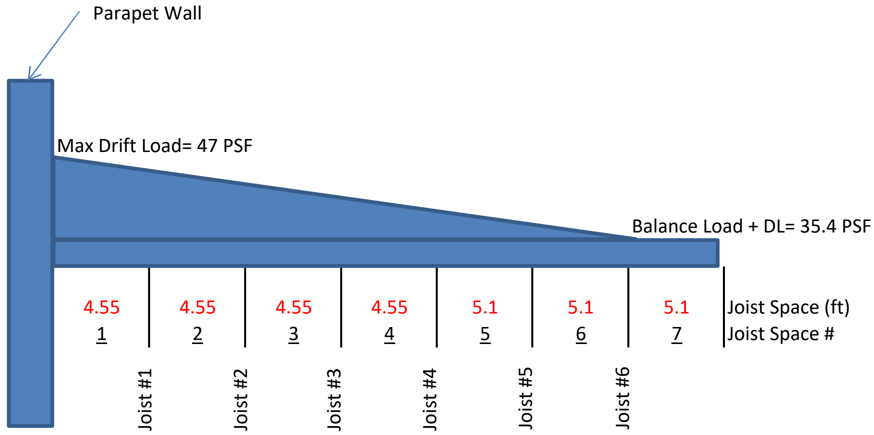
Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.613** in

PASS - Maximum deflection does not exceed deflection limit

Joists With Drift Along Length of Joist

Dead Load=	15	psf	**Area -**	**Interior Joists - Non-Drift Loaded Joists**
Balance Load=	20.4	psf		
Total Load=	35.4	psf (DL+Balance)		
Max. Drift Load	47	psf		
Length of Drift=	11	ft		

Joist Space #1=	4.6	ft
Joist Space #2=	4.6	ft
Joist Space #3=	4.6	ft
Joist Space #4=	4.6	ft
Joist Space #5=	5.1	ft
Joist Space #6=	5.1	ft
Joist Space #7=	5.1	ft



Pressure Load #1=	62.95909	psf
Pressure Load #2=	43.51818	psf
Pressure Load #3=	35.4	psf
Pressure Load #4=	35.4	psf
Pressure Load #5=	35.4	psf
Pressure Load #6=	35.4	psf

Uniform Load #1=	286.4639	lb/ft (Joist #1)
Uniform Load #2=	198.0077	lb/ft (Joist #2)
Uniform Load #3=	161.07	lb/ft (Joist #3)
Uniform Load #4=	170.805	lb/ft (Joist #4)
Uniform Load #5=	180.54	lb/ft (Joist #5)
Uniform Load #6=	180.54	lb/ft (Joist #6)

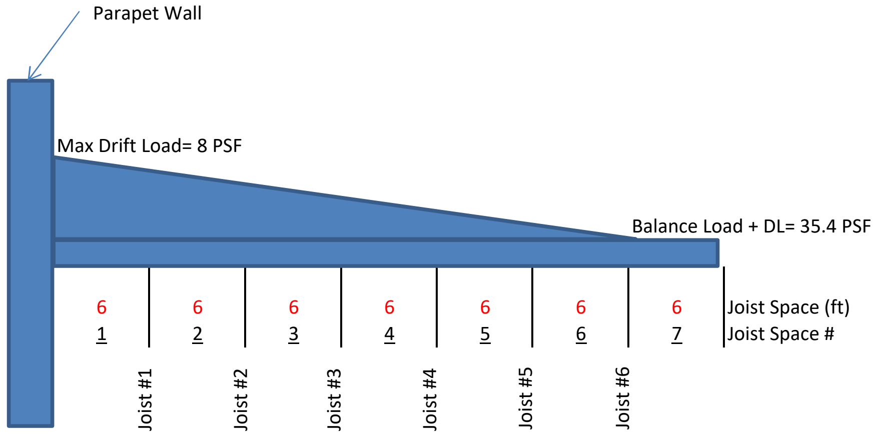
Max Loading =	286.4639	lb/ft
Joist Span =	29	ft
Joist Selection =	24K4	
Joist Capacity =	290	lb/ft
% Capacity =	98.781%	

OK use 24K4

Joists With Drift Along Length of Joist

Area - Interior Joists - Non-Drift Loaded Joists

Dead Load=	15	psf
Balance Load=	20.4	psf
Total Load=	35.4	psf (DL+Balance)
Max. Drift Load	8	psf
Length of Drift=	11.5	ft
Joist Space #1=	6.0	ft
Joist Space #2=	6.0	ft
Joist Space #3=	6.0	ft
Joist Space #4=	6.0	ft
Joist Space #5=	6.0	ft
Joist Space #6=	6.0	ft
Joist Space #7=	6.0	ft
Pressure Load #1=	39.22609	psf
Pressure Load #2=	35.4	psf
Pressure Load #3=	35.4	psf
Pressure Load #4=	35.4	psf
Pressure Load #5=	35.4	psf
Pressure Load #6=	35.4	psf
Uniform Load #1=	235.3565	lb/ft (Joist #1)
Uniform Load #2=	212.4	lb/ft (Joist #2)
Uniform Load #3=	212.4	lb/ft (Joist #3)
Uniform Load #4=	212.4	lb/ft (Joist #4)
Uniform Load #5=	212.4	lb/ft (Joist #5)
Uniform Load #6=	212.4	lb/ft (Joist #6)



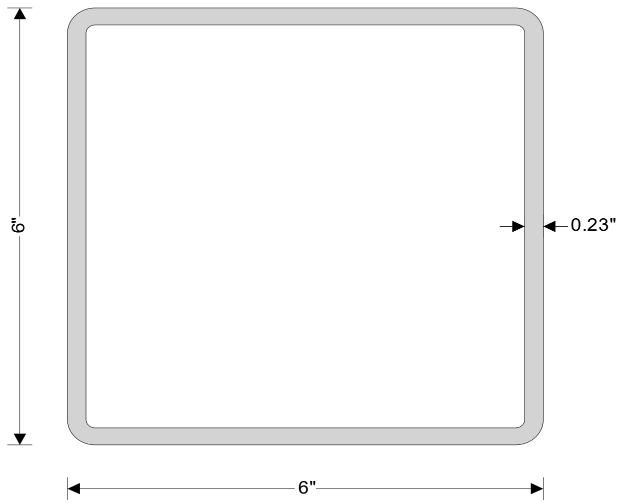
Max Loading =	235.3565	lb/ft
Joist Span =	29	ft
Joist Selection =	20K6	
Joist Capacity =	242	lb/ft
% Capacity =	97.255%	

OK use 20K6

STEEL COLUMN DESIGN

In accordance with AISC360-10 and the ASD method

Tedds calculation version 1.0.09



Column and loading details

Column details

Column section	**HSS 6x6x1/4**

Design loading

Required axial strength	P_r = **45** 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 = **198** in
For buckling about y axis	L_y = **198** in
For torsional buckling	L_z = **198** 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**

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) λ_{f_c} = b / t = **22.751**
Width to thickness ratio of web (compression) λ_{w_c} = h / t = **22.751**
Width to thickness ratio of flange (major flexure) λ_{f_fx} = b / t = **22.751**
Width to thickness ratio of web (major flexure) λ_{w_fx} = h / t = **22.751**
Width to thickness ratio of flange (minor flexure) λ_{f_fy} = h / t = **22.751**
Width to thickness ratio of web (minor flexure) λ_{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 $SR_x = K_x \times L_x / r_x$ = **84.6**
Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y$ = **84.6**

Reduction factor for slender elements

Reduction factor for slender elements (E7)

The section does not contain any slender elements therefore:-
Slender element reduction factor Q = **1.0**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (SR_x)^2$ = **40.0** ksi
Reduction factor Q_x = Q = **1.000**
Flexural buckling stress about x axis $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y$ = **28.4** ksi
Nominal flexural buckling strength $P_{nx} = F_{crx} \times A_g$ = **148.9** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (SR_y)^2$ = **40.0** ksi
Reduction factor Q_y = Q = **1.000**
Flexural buckling stress about y axis $F_{cry} = Q_y \times (0.658^{Q_y \times F_y / F_{ey}}) \times F_y$ = **28.4** ksi
Nominal flexural buckling strength $P_{ny} = F_{cry} \times A_g$ = **148.9** kips

Allowable compressive strength (cl. E1)

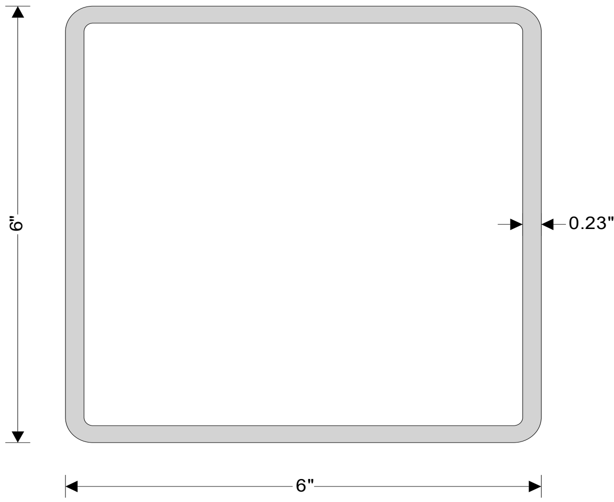
Safety factor for compression Ω_c = **1.67**
Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}) / \Omega_c$ = **89.2** kips

PASS - The allowable compressive strength exceeds the required compressive strength

STEEL COLUMN DESIGN

In accordance with AISC360-10 and the ASD method

Tedds calculation version 1.0.09



Column and loading details

Column details

Column section — **HSS 6x6x1/4**

Design loading

Required axial strength	P_r = **9** 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 = **198** in
For buckling about y axis	L_y = **198** in
For torsional buckling	L_z = **198** 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**

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) λ_{f_c} = b / t = **22.751**

Width to thickness ratio of web (compression) λ_{w_c} = h / t = **22.751**

Width to thickness ratio of flange (major flexure) λ_{f_fx} = b / t = **22.751**

Width to thickness ratio of web (major flexure) λ_{w_fx} = h / t = **22.751**

Width to thickness ratio of flange (minor flexure) λ_{f_fy} = h / t = **22.751**

Width to thickness ratio of web (minor flexure) λ_{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 $SR_x = K_x \times L_x / r_x$ = **84.6**

Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y$ = **84.6**

Reduction factor for slender elements

Reduction factor for slender elements (E7)

The section does not contain any slender elements therefore:-

Slender element reduction factor Q = **1.0**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (SR_x)^2$ = **40.0** ksi

Reduction factor Q_x = Q = **1.000**

Flexural buckling stress about x axis $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y$ = **28.4** ksi

Nominal flexural buckling strength $P_{nx} = F_{crx} \times A_g$ = **148.9** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (SR_y)^2$ = **40.0** ksi

Reduction factor Q_y = Q = **1.000**

Flexural buckling stress about y axis $F_{cry} = Q_y \times (0.658^{Q_y \times F_y / F_{ey}}) \times F_y$ = **28.4** ksi

Nominal flexural buckling strength $P_{ny} = F_{cry} \times A_g$ = **148.9** kips

Allowable compressive strength (cl. E1)

Safety factor for compression Ω_c = **1.67**

Allowable compressive strength $P_c = \min(P_{nx}, P_{ny}) / \Omega_c$ = **89.2** kips

PASS - The allowable compressive strength exceeds the required compressive strength

STEEL MEMBER ANALYSIS & DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

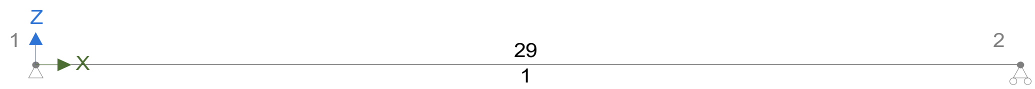
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

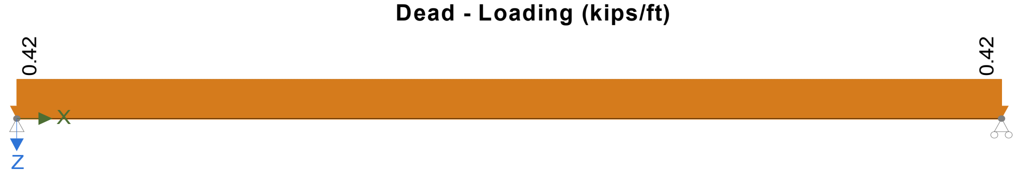
Geometry (ft) - Steel (AISC)



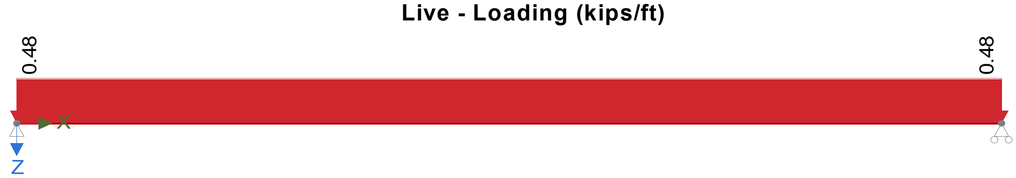
Loading

Self weight included

Dead - Loading (kips/ft)



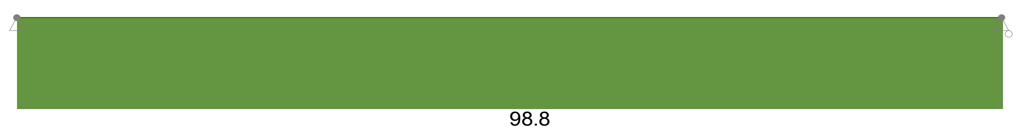
Live - Loading (kips/ft)



Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

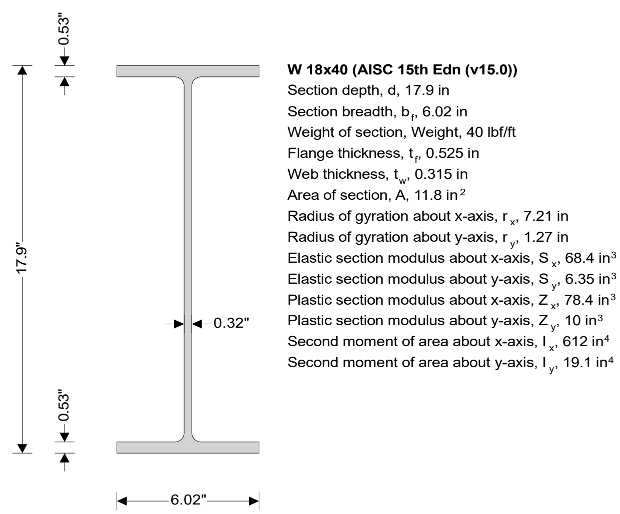
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Beam design

Section details

Section type	W 18x40 (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



Lateral restraint

Top flange has full lateral restraint

Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **5.73**	
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	$(d - 2 \times k) / t_w$ = **50.94**	
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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **13.6** kips

Web area $A_w = d \times t_w$ = **5.638** in^2

Web plate buckling coefficient k_v = **5**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **169.2** kips

Safety factor Ω_v = 1.50

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **112.8** kips

$V_{r,x} / V_{c,x}$ = **0.121**

PASS - Allowable shear strength exceeds required shear strength

Check design 14ft 6in along span

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **98.8** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **326.7** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **326.7** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **195.6** kips_ft

$M_{r,x} / M_{c,x}$ = **0.505**

PASS - Allowable flexural strength exceeds required flexural strength

Check design 14ft 6in along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.862** in

Allowable deflection $\delta_{x,Allowable} = L_{m1\_s1} / 360$ = **0.967** in

$\delta_x / \delta_{x,Allowable}$ = **0.891**

PASS - Allowable deflection exceeds design deflection

STEEL MEMBER ANALYSIS & DESIGN (AISC 360)

In accordance with AISC360 14th Edition published 2010 using the ASD method

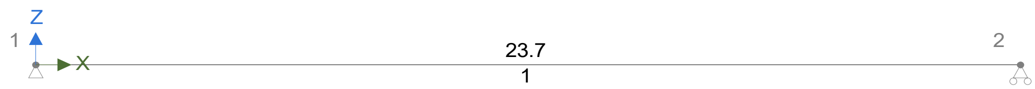
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

Geometry (ft) - Steel (AISC)



Loading

Self weight included

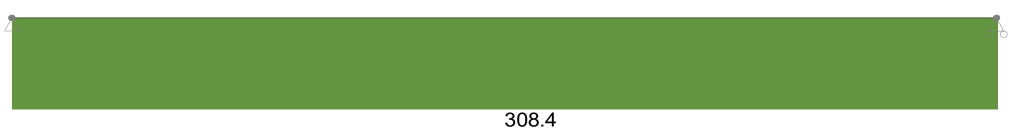
Dead - Loading (kips/ft)

Live - Loading (kips/ft)

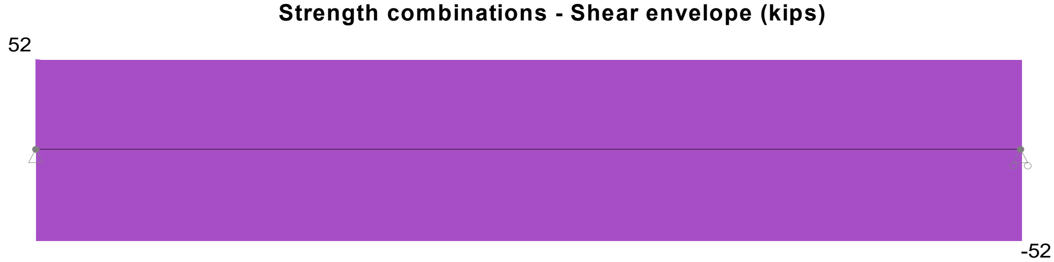
Results

Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)



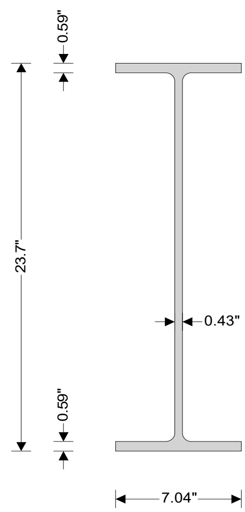
Safety factors

Shear	Ω_v = **1.67**
Flexure	Ω_b = **1.67**
Tensile yielding	$\Omega_{t,y}$ = **1.67**
Tensile rupture	$\Omega_{t,r}$ = **2.00**
Compression	Ω_c = **1.67**

Beam design

Section details

Section type	W 24x62 (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



W 24x62 (AISC 15th Edn (v15.0))
Section depth, d, 23.7 in
Section breadth, b_f, 7.04 in
Weight of section, Weight, 62 lbf/ft
Flange thickness, t_f, 0.59 in
Web thickness, t_w, 0.43 in
Area of section, A, 18.2 in^2
Radius of gyration about x-axis, r_x, 9.23 in
Radius of gyration about y-axis, r_y, 1.38 in
Elastic section modulus about x-axis, S_x, 131 in^3
Elastic section modulus about y-axis, S_y, 9.8 in^3
Plastic section modulus about x-axis, Z_x, 153 in^3
Plastic section modulus about y-axis, Z_y, 15.7 in^3
Second moment of area about x-axis, I_x, 1550 in^4
Second moment of area about y-axis, I_y, 34.5 in^4

Lateral restraint

Top flange has full lateral restraint

Bottom flange has lateral restraint at supports only

Consider Combination 1 - 1.0D + 1.0Lr (Strength)

Classification of sections for local buckling - Section B4

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

Width to thickness ratio	$b_f / (2 \times t_f)$ = **5.97**	
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	$(d - 2 \times k) / t_w$ = **50.05**	
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

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **52** kips

Web area $A_w = d \times t_w$ = **10.191** in^2

Web plate buckling coefficient k_v = **5**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_v = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_v$ = **305.7** kips

Safety factor Ω_v = 1.50

Allowable shear strength $V_{c,x} = V_{n,x} / \Omega_v$ = **203.8** kips

$V_{r,x} / V_{c,x}$ = **0.255**

PASS - Allowable shear strength exceeds required shear strength

Check design 11ft 10.2in along span

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **308.4** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x$ = **637.5** kips_ft

Allowable flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x}$ = **637.5** kips_ft

Allowable flexural strength $M_{c,x} = M_{n,x} / \Omega_b$ = **381.7** kips_ft

$M_{r,x} / M_{c,x}$ = **0.808**

PASS - Allowable flexural strength exceeds required flexural strength

Check design 11ft 10.2in along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.726** in

Allowable deflection $\delta_{x,Allowable} = L_{m1\_s1} / 360$ = **0.79** in

$\delta_x / \delta_{x,Allowable}$ = **0.919**

PASS - Allowable deflection exceeds design deflection

Summary

The gravity structure system of the project referenced above consists primarily of steel joists and girders, load-bearing CMU walls and steel columns. CMU walls support all gravity and lateral loads along the perimeter of the building. CMU wall designs are performed in RAM Elements with Tedds calculations, as required.

The following section of calculations covers the complete design of the tilt CMU 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 “Lateral Stability” section of these calculations for the determination of all wind and seismic loads.

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the strength design method

Tedds calculation version 2.2.04

Masonry wall panel details

Typical Exterior Wall - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **88** ft

Panel height h = **26** ft

Seismic properties

Seismic design category B

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **0.345**

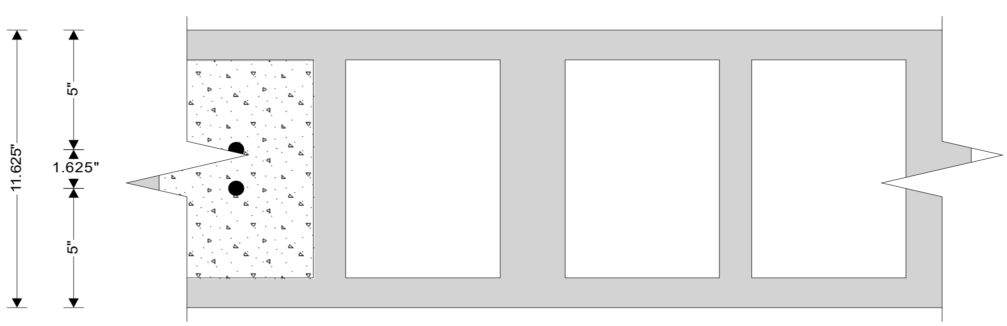
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **11.625** in



Masonry details

Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **1900** psi

Density of masonry units γ_{block} = **115** lb/ft^3

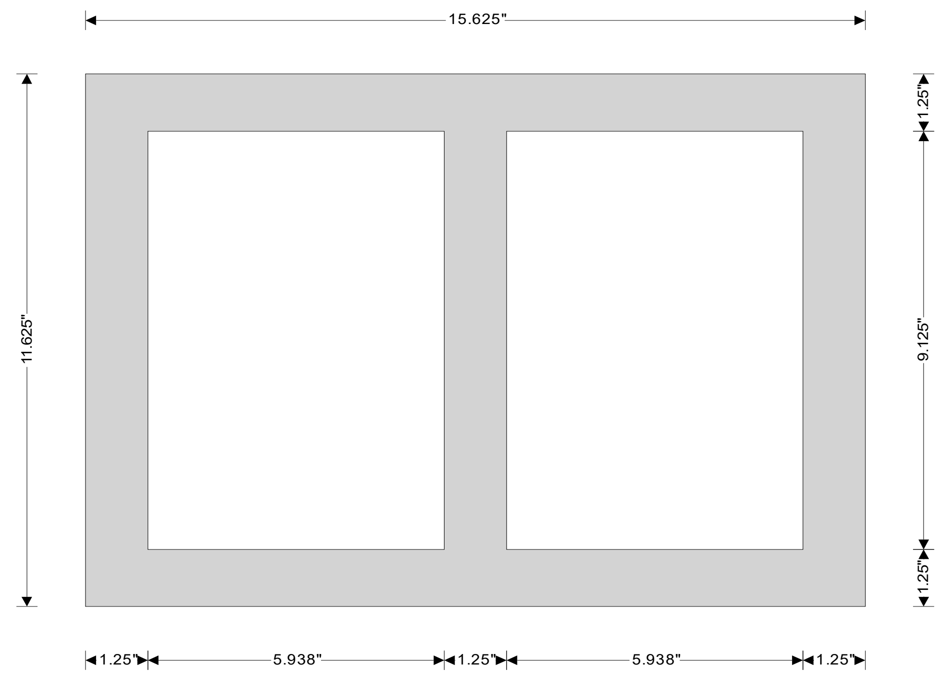
Height of masonry units h_b = **7.625** in

Length of masonry units l_b = **15.625** in

Number of internal webs N_{web} = **1**

Number of end webs N_{end} = **2**

Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **56.28** in^2/ft
Area of grout	$A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **14.15** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **58.7** psf



From TMS 602-11 Table 2 - Compressive strength of masonry

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

From TMS 402 -11 Table 3.1.8.2 - Modulus of rupture

Modulus of rupture normal to bed	f_{r_norm} = **80** psi
Modulus of rupture parallel to bed	f_{r_para} = **125** psi

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Allowable tensile stress in reinforcement	F_s = **32000** psi
Modulus of elasticity for reinforcement	E_s = **29000000** psi
Vertical reinforcement provided	No.5 bars at 48 in centers
Area of vertical reinforcement, per face	$A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in^2/ft

Lateral out-of-plane loads

Wind load on panel	W = **28** psf
Wind load on parapet	W_p = **70** psf
Seismic load factor (ASCE7 12.11.1)	$F_p = 0.4 \times S_{DS} \times I_e$ = **0.138**
Seismic load from wall	$E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **8.1** psf
Additional seimic load	E_{add} = **0** psf

Seismic lateral load on panel $E = E_{wall} + E_{add} = \mathbf{8.1}$ psf

Lateral in-plane loads

Wind shear load on wall $V_W = \mathbf{21800}$ lbs

Vertical loading details

Dead load at supported level $DL = \mathbf{276}$ lb/ft

Live load from above $LL_{above} = \mathbf{290}$ lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS} = \mathbf{0.069}$

From ASCE 7-10 cl.2.3.2 - Combining factored loads using strength design (Utilization)

Load combination no.1 $1.4 \times DL$ (0.061)

Load combination no.5 $1.2 \times DL + W + LL + 0.5 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.881)

Load combination no.7 $0.9 \times DL + W$ (0.942)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = \mathbf{70.4}$ in^2/ft

Properties for walls loaded out-of-plane:

Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) = \mathbf{1091.7}$ in^4/ft

Section modulus $S = I / c = \mathbf{187.8}$ in^3/ft

Radius of gyration $r = \sqrt{[I / A]} = \mathbf{3.937}$ in

Effective height factor $K = \mathbf{1}$

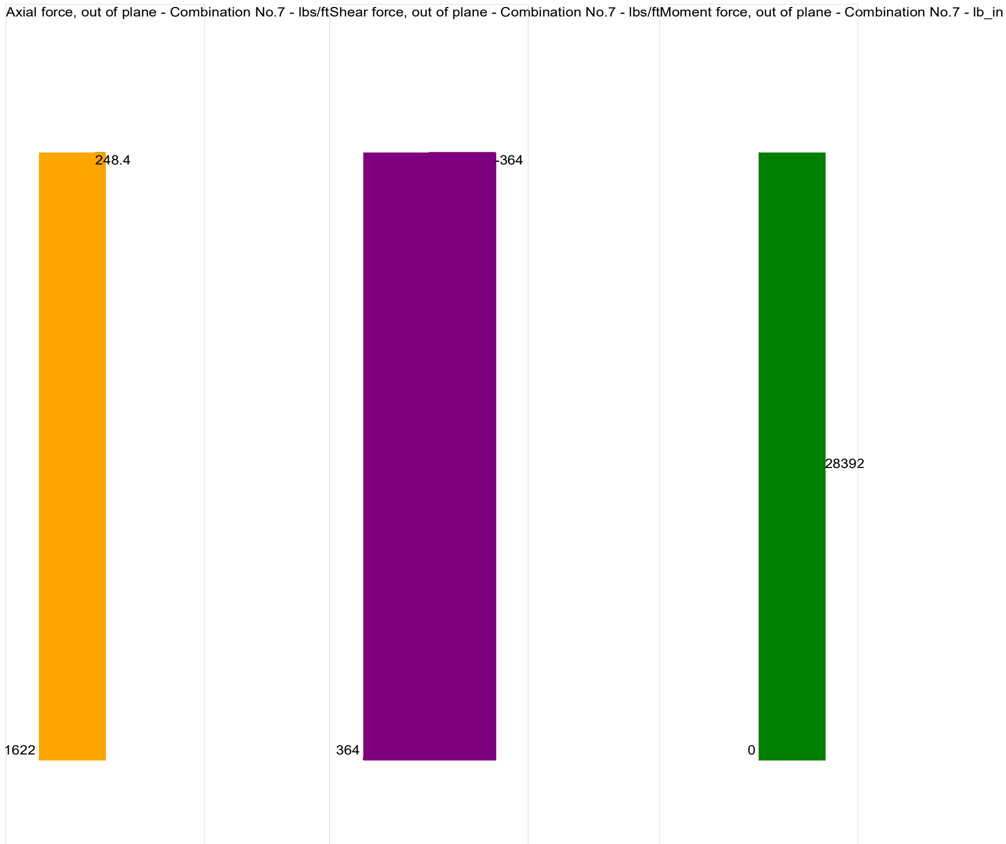
Properties for walls loaded in-plane:

Net moment of inertia

$$I_{x\_net} = t \times L^3 / 12 - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell3}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell9}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell10}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell11}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell13}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell14}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell15}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell16}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell17}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell19}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell20}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell21}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell22}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell23}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell25}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell26}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell27}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell28}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell29}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell31}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell32}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell33}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell34}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell35}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell37}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell38}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell39}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell40}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell41}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell43}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell44}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell45}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell46}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell47}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell49}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell50}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell51}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell52}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell53}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell55}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell56}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell57}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell58}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell59}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell61}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell62}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell63}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell64}^2) - 2 \times (I_{x\_cell} + A_{cell} \times x_{cell65}^2) = \mathbf{587318944} \text{ in}^4$$

Net section modulus $S_{x\_net} = I_{x\_net} / (L / 2) = \mathbf{1112346}$ in^3

Consider wall at maximum moment location under load combination no.7



Maximum moment location	13 ft
Axial load at mid-height of panel	P = **935** lb/ft
Slenderness ratio	$(K \times h) / r$ = **79.244** < 99
Nominal axial strength	$P_n = 0.8 \times (0.8 \times f'_m \times (A - 2 \times A_s) + 2 \times A_s \times 0 \text{ ksi}) \times [1 - ((K \times h) / (140 \times r))^2]$ = **45849** lb/ft
Strength reduction factor	ϕ = **0.9**
Design axial strength	$\phi \times P_n$ = **41264** lb/ft
	$P / (\phi \times P_n)$ = **0.023**

PASS - Nominal axial strength exceeds axial load

Factored axial stress	P / t = **7** psi
Factored axial stress limit	$0.2 \times f'_m$ = **300** psi

PASS - Allowable stress under out of plane loads exceeds factored axial stress

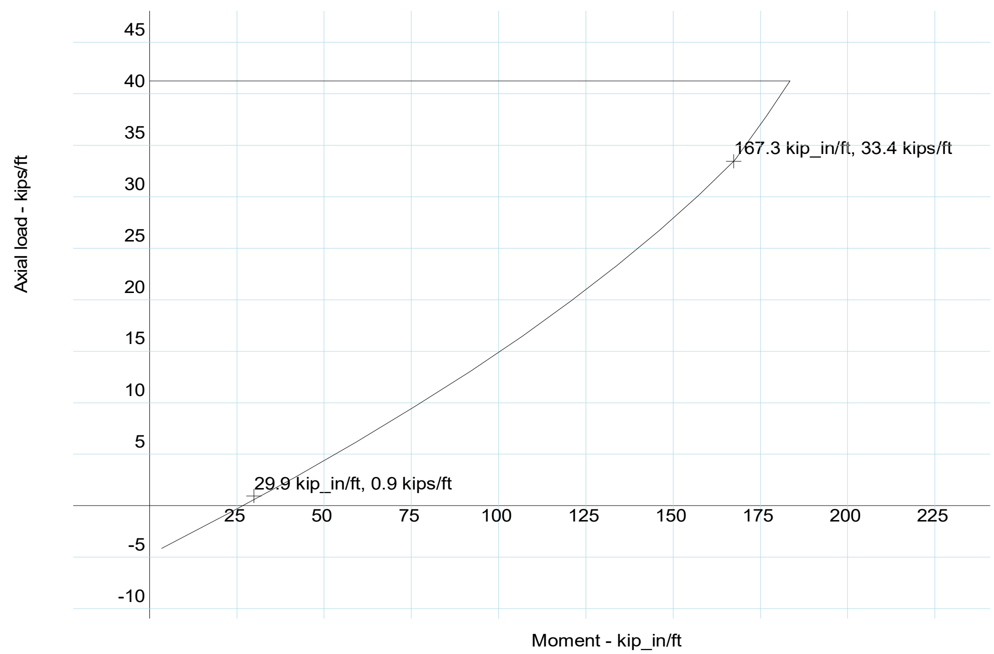
Nominal cracking moment strength	$M_{cr} = S \times f_{r\_norm}$ = **14963** lb_in/ft
Modular ratio	$n = E_s / E_m$ = **21.481**
Distance to neutral axis	$c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m)$ = **0.481** in
Moment of inertia of cracked section	$I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3$ = **73.7** in^4/ft
By iteration	$M_{u0} = M$ = **28392** lb_in/ft
	$\delta_{u0} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I) + 5 \times (M_{u0} - M_{cr}) \times h^2 / (48 \times E_m \times I_{cr})$ = **1.471** in
	$M_{u3} = M_{u0} + P \times \delta_{u2}$ = **29911** lb_in/ft

	$\delta_{u3} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I) + 5 \times (M_{u3} - M_{cr}) \times h^2 / (48 \times E_m \times I_{cr})$ = **1.626** in
Bending moment at mid-height of panel	$M = M_{u0} + P \times \delta_{u3}$ = **29912** lb_in/ft
Depth of reinforcement	d = **6.625** in
Strength reduction factor	ϕ = **0.9**
Tensile strain in reinforcement	$\varepsilon_s = f_y / E_s$ = **0.0021**
Maximum usable compressive strain of masonry	ε_{mu} = **0.0025**
Fiber of max.compressive strain to neutral axis	$c_{bal} = \varepsilon_{mu} \times d / (\varepsilon_{mu} + \varepsilon_s)$ = **3.625** in
Tensile force at balance point	$T_{bal} = A_s \times f_y$ = **4602** lb/ft
	β_1 = **0.8**
Compressive force at balance point	$C_{bal} = 0.8 \times f'_m \times \beta_1 \times c_{bal}$ = **41760** lb/ft
Design axial force at balance point	$P_{bal} = \phi \times (C_{bal} - T_{bal})$ = **33442** lb/ft
Design moment at balance point	$M_{bal} = \phi \times [T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - \beta_1 \times c_{bal} / 2)]$ = **167325** lb_in/ft
Maximum design moment from integration diagram	M_c = **31767** lb_in/ft
	M / M_c = **0.942**

PASS - Combination of applied axial load and flexure is acceptable

Maximum area of tensile reinforcement (3.3.3.5)	$A_{s\_max} = 0.64 \times f'_m \times [\varepsilon_{mu} / (\varepsilon_{mu} + 1.5 \times \varepsilon_s)] \times d / f_y$ = **0.568** in^2/ft

PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram

Consider wall at top under load combination no.7

Shear force	V = **364** lb/ft
Compressive force	N_u = **248** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **12.939** in^2/ft
Nominal shear strength	$V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_u, 6 \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **2067** lb/ft
Strength reduction factor	ϕ_v = **0.8**
Design shear strength	$\phi_v \times V_n$ = **1653** lb/ft

$V / (\phi_v \times V_n) =$ **0.220**

PASS - Design shear strength exceeds applied shear strength

MASONRY WALL PANEL DESIGN TO MSJC-11

Using the strength design method

Tedds calculation version 2.2.04

Masonry wall panel details

GL4 Wall - Reinforced single-wythe wall, the wall is pinned at the top and at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **62** ft

Panel height h = **26** ft

Seismic properties

Seismic design category B

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **0.345**

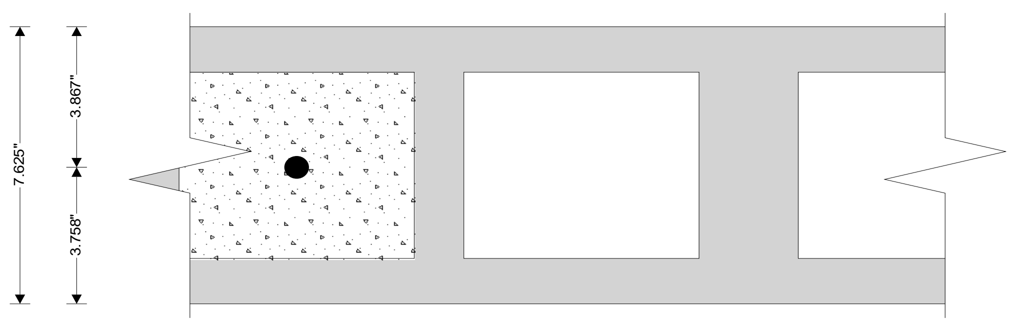
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in



Masonry details

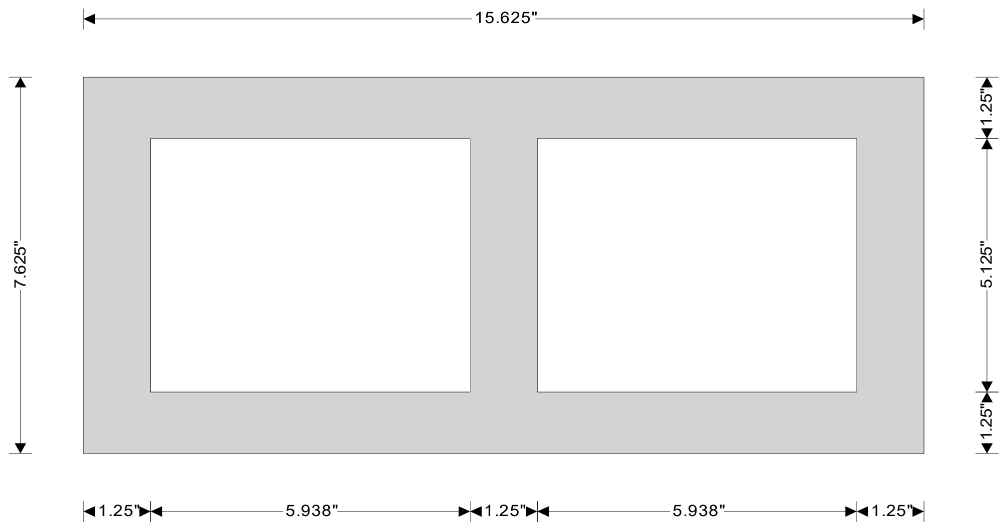
Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **1900** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-11 Table 2 - Compressive strength of masonry

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

From TMS 402 -11 Table 3.1.8.2 - Modulus of rupture

Modulus of rupture normal to bed	f_{r_norm} = **80** psi
Modulus of rupture parallel to bed	f_{r_para} = **125** psi

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Allowable tensile stress in reinforcement	F_s = **32000** psi
Modulus of elasticity for reinforcement	E_s = **29000000** psi
Vertical reinforcement provided	No.5 bars at 48 in centers
Area of vertical reinforcement	$A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in^2/ft

Lateral out-of-plane loads

Wind load on panel	W = **5** psf
Wind load on parapet	W_p = **70** psf
Seismic load factor (ASCE7 12.11.1)	$F_p = 0.4 \times S_{DS} \times I_e$ = **0.138**
Seismic load from wall	$E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **6** psf
Additional seimic load	E_{add} = **0** psf
Seismic lateral load on panel	$E = E_{wall} + E_{add}$ = **6** psf

Lateral in-plane loads

Vertical loading details

Dead load at supported level DL = **125** lb/ft

Live load from above LL_{above} = **250** lb/ft

Vertical seismic load factor applied to dead load $F_{Ev} = 0.2 \times S_{DS}$ = **0.069**

From ASCE 7-10 cl.2.3.2 - Combining factored loads using strength design (Utilization)

Load combination no.1 $1.4 \times DL$ (0.113)

Load combination no.5 $1.2 \times DL + W + LL + 0.5 \times (LL_r \text{ or } SL \text{ or } RL)$ (0.272)

Load combination no.7 $0.9 \times DL + W$ (0.294)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **52.7** in^2/ft

Properties for walls loaded out-of-plane:

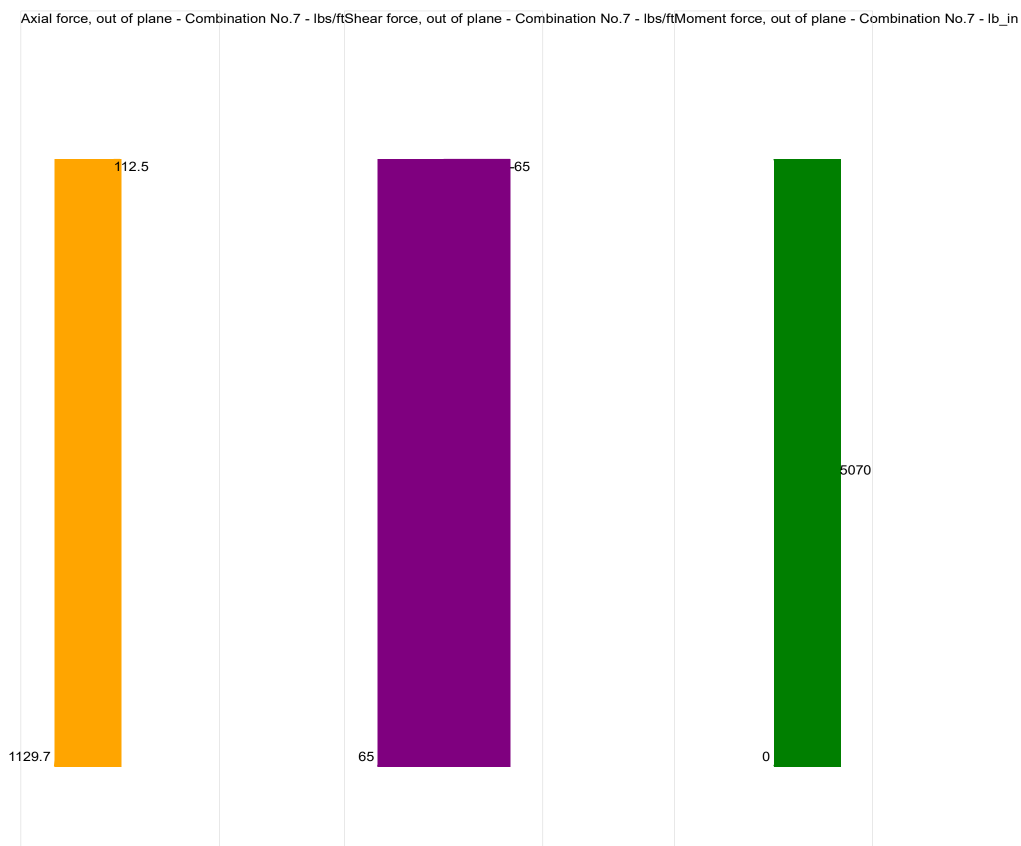
Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b)$ = **358.4** in^4/ft

Section modulus $S = I / c$ = **94** in^3/ft

Radius of gyration $r = \sqrt{[I / A]}$ = **2.608** in

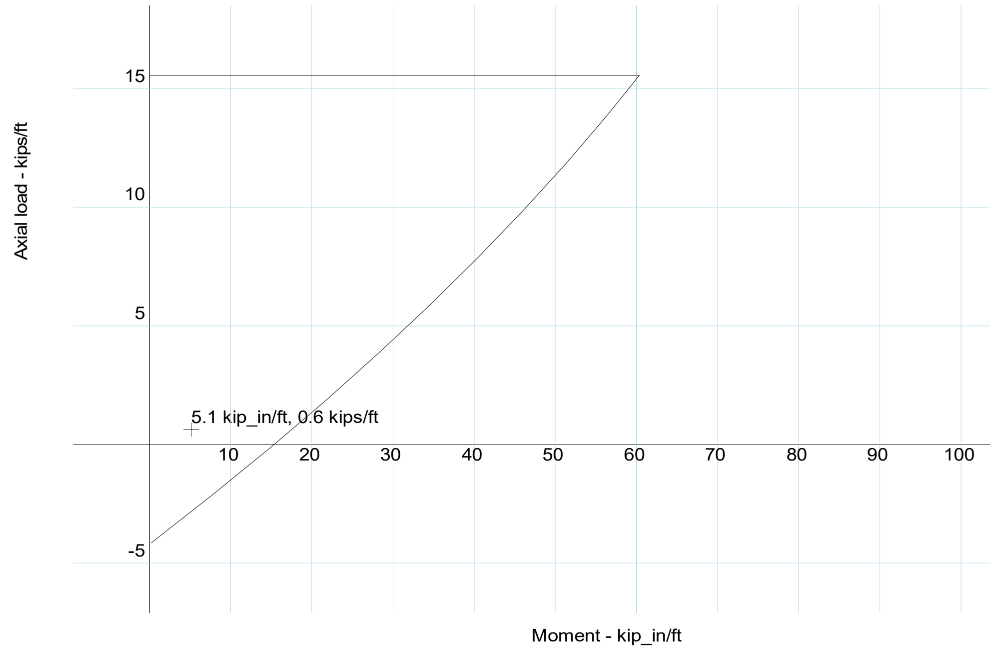
Effective height factor K = **1**

Consider wall at maximum moment location under load combination no.7



Maximum moment location 13 ft

Axial load at mid-height of panel	P = **621** lb/ft
Slenderness ratio	$(K \times h) / r$ = **119.645** > 99
Nominal axial strength	$P_n = 0.8 \times (0.8 \times f'_m \times (A - A_s) + A_s \times 0\ \text{ksi}) \times (70 \times r / (K \times h))^2$ = **17294** lb/ft
Strength reduction factor	ϕ = **0.9**
Design axial strength	$\phi \times P_n$ = **15565** lb/ft
	$P / (\phi \times P_n)$ = **0.040**
	PASS - Nominal axial strength exceeds axial load
Factored axial stress	P / t = **7** psi
Factored axial stress limit	$0.05 \times f'_m$ = **75** psi
	PASS - Allowable stress under out of plane loads exceeds factored axial stress
Nominal cracking moment strength	$M_{cr} = S \times f_{r\_norm}$ = **7489** lb_in/ft
Modular ratio	$n = E_s / E_m$ = **21.481**
Distance to neutral axis	$c_{cr} = (A_s \times f_y + P) / (0.64 \times f'_m)$ = **0.453** in
Moment of inertia of cracked section	$I_{cr} = n \times (A_s + P \times t / (f_y \times 2 \times d)) \times (d - c_{cr})^2 + c_{cr}^3 / 3$ = **22.1** in^4/ft
By iteration	$M_{u0} = M$ = **5070** lb_in/ft
	$\delta_{u0} = 5 \times M_{u0} \times h^2 / (48 \times E_m \times I)$ = **0.106** in
	$M_{u1} = M_{u0} + P \times \delta_{u0}$ = **5136** lb_in/ft
	$\delta_{u1} = 5 \times M_{u1} \times h^2 / (48 \times E_m \times I)$ = **0.108** in
Bending moment at mid-height of panel	$M = M_{u0} + P \times \delta_{u1}$ = **5137** lb_in/ft
Depth of reinforcement	d = **3.867** in
Strength reduction factor	ϕ = **0.9**
Tensile strain in reinforcement	$\varepsilon_s = f_y / E_s$ = **0.0021**
Maximum usable compressive strain of masonry	ε_{mu} = **0.0025**
Fiber of max.compressive strain to neutral axis	$c_{bal} = \varepsilon_{mu} \times d / (\varepsilon_{mu} + \varepsilon_s)$ = **2.116** in
Tensile force at balance point	$T_{bal} = A_s \times f_y$ = **4602** lb/ft
	β_1 = **0.8**
Compressive force at balance point	$C_{bal} = 0.8 \times f'_m \times \beta_1 \times c_{bal}$ = **24375** lb/ft
Design axial force at balance point	$P_{bal} = \phi \times (C_{bal} - T_{bal})$ = **17796** lb/ft
Design moment at balance point	$M_{bal} = \phi \times [T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - \beta_1 \times c_{bal} / 2)]$ = **65296** lb_in/ft
Maximum design moment from integration diagram	M_c = **17473** lb_in/ft
	M / M_c = **0.294**
	PASS - Combination of applied axial load and flexure is acceptable
Maximum area of tensile reinforcement (3.3.3.5)	$A_{s\_max} = 0.64 \times f'_m \times [\varepsilon_{mu} / (\varepsilon_{mu} + 1.5 \times \varepsilon_s)] \times d / f_y$ = **0.331** in^2/ft
	PASS - Area of reinforcement provided is less than maximum allowable



Strength interaction diagram

Consider wall at top under load combination no.7

Shear force	V = **65** lb/ft
Compressive force	N_u = **112** lb/ft
Net shear area	$A_{nv} = d \times l_b / ((N_{web} + 1) \times s_{grout})$ = **7.553** in^2/ft
Nominal shear strength	$V_n = \min([4 - 1.75 \times \min(M / (V \times d), 1)] \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})} + 0.25 \times N_u, 6 \times A_{nv} \times \sqrt{(f'_m \times 1 \text{ psi})})$ = **1198** lb/ft
Strength reduction factor	ϕ_v = **0.8**
Design shear strength	$\phi_v \times V_n$ = **959** lb/ft
	$V / (\phi_v \times V_n)$ = **0.068**

PASS - Design shear strength exceeds applied shear strength

MASONRY LINTEL DESIGN TO TMS/MSJC 2013

Using the allowable stress design method

Tedds calculation version 1.2.01

Masonry details

Masonry type — Concrete

Density of masonry unit — γ = 135 lb/ft^3

Pattern bond — Running

Mortar type — PCL Type M

Compressive strength of masonry unit — f'_{cu} = 1900.0 psi

Net compressive strength of masonry (Table 2) — f'_m = 1900.0 psi

Modulus of elasticity (4.2.2) — $E_m = 900 \times f'_m$ = **1710000** psi

Allowable flexural tensile stress (8.2.4.2) — F_t = 106.0 psi

Reinforcement details

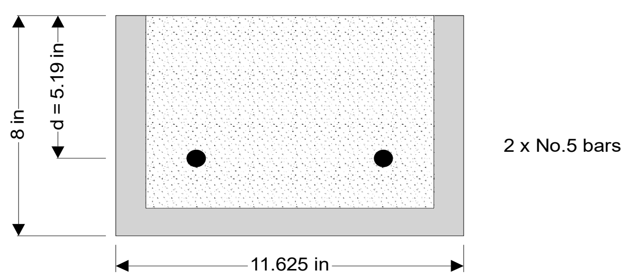
Allowable tensile stress — F_s = 32000 psi

Modulus of elasticity of steel — E_s = 29000000 psi

Cover to reinforcement

Bottom cover to reinforcement — c_{nom_b} = 1.5 in

Side cover to reinforcement — c_{nom_s} = 1.5 in



Section properties

Modular ratio — $n = E_s / E_m$ = **16.96**

Section width — b = **11.625** in

Section depth — h = **8** in

Net shear area — $A_{nv} = b \times h$ = **93** in^2

Section modulus — $S = b \times h^2 / 6$ = **124** in^3

Depth to tension reinforcement — d = **5.19** in

Flexure design (Chapter 8)

Tension reinforcement — 2 x No. 5 bars

Area of tension reinforcement — $A_s = N_{bot} \times BarArea_{bot}$ = **0.62** in^2

Reinforcement ratio — $\rho_{ratio} = A_s / (b \times d)$ = **0.01028**

Neutral axis factor — $k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n$ = **0.441**

Lever arm factor — $j = 1 - k / 3$ = **0.853**

Cracking moment — $M_{cr} = 2.5 \times F_t \times S$ = **2.7** kip_ft

Design bending moment — M = **1.00** kip_ft

Tensile stress in reinforcement — $f_s = M / (A_s \times j \times d)$ = **4372** psi

Allowable tensile stress in reinf. (8.3.3.1) — F_s = **32000** psi

Reinforcement stress ratio — f_s / F_s = **0.137**

PASS - Allowable tensile stress exceeds tensile stress due to flexure

Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2) =$ **203.6** psi
Allowable stress in masonry (8.3.4.2.2)	$F_b = 0.45 \times f'_m =$ **855.0** psi
Masonry stress ratio	$f_b / F_b =$ **0.238**

PASS - Allowable compressive stress exceeds compressive stress due to flexure

Shear design (Chapter 8)

Design shear force	V = **0.55** kips
Depth of shear area	$d_v =$ **8.00** in
Moment shear relationship, M/Vd	Assume $M\_Vd_{ratio} = 1$
Shear stress (8-24)	$f_v = V / A_{nv} =$ **5.9** psi
Allowable masonry shear stress (8-29)	$F_v = 1/2 \times (4.0 - 1.75 \times M\_Vd_{ratio}) \times \sqrt{(f'_m \times 1psi)}$
	$F_v =$ **49.0** psi
Masonry shear stress ratio	$f_v / F_v =$ **0.121**

PASS - Allowable shear stress exceeds shear stress in masonry

MASONRY LINTEL DESIGN TO TMS/MSJC 2013

Using the allowable stress design method

Tedds calculation version 1.2.01

Masonry details

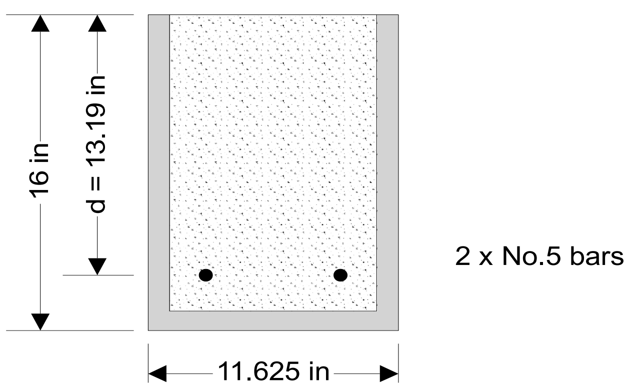
Masonry type	Concrete
Density of masonry unit	γ = 135 lb/ft^3
Pattern bond	Running
Mortar type	PCL Type M
Compressive strength of masonry unit	f'_{cu} = 1900.0 psi
Net compressive strength of masonry (Table 2)	f'_m = 1900.0 psi
Modulus of elasticity (4.2.2)	$E_m = 900 \times f'_m$ = **1710000** psi
Allowable flexural tensile stress (8.2.4.2)	F_t = 106.0 psi

Reinforcement details

Allowable tensile stress	F_s = 32000 psi
Modulus of elasticity of steel	E_s = 29000000 psi

Cover to reinforcement

Bottom cover to reinforcement	c_{nom_b} = 1.5 in
Side cover to reinforcement	c_{nom_s} = 1.5 in



Section properties

Modular ratio	$n = E_s / E_m$ = **16.96**
Section width	b = **11.625** in
Section depth	h = **16** in
Net shear area	$A_{nv} = b \times h$ = **186** in^2
Section modulus	$S = b \times h^2 / 6$ = **496** in^3
Depth to tension reinforcement	d = **13.19** in

Flexure design (Chapter 8)

Tension reinforcement	2 x No. 5 bars
Area of tension reinforcement	$A_s = N_{bot} \times BarArea_{bot}$ = **0.62** in^2
Reinforcement ratio	$\rho_{ratio} = A_s / (b \times d)$ = **0.00404**
Neutral axis factor	$k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n$ = **0.308**
Lever arm factor	$j = 1 - k / 3$ = **0.897**
Cracking moment	$M_{cr} = 2.5 \times F_t \times S$ = **11.0** kip_ft

Design bending moment	M = **15.80** kip_ft
Tensile stress in reinforcement	f_s = M / (A_s × j × d) = **25838** psi
Allowable tensile stress in reinf. (8.3.3.1)	F_s = **32000** psi
Reinforcement stress ratio	f_s / F_s = **0.807**

PASS - Allowable tensile stress exceeds tensile stress due to flexure

Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2)$ = **678.3** psi
Allowable stress in masonry (8.3.4.2.2)	$F_b = 0.45 \times f'_m$ = **855.0** psi
Masonry stress ratio	f_b / F_b = **0.793**

PASS - Allowable compressive stress exceeds compressive stress due to flexure

Shear design (Chapter 8)

Design shear force	V = **4.80** kips
Depth of shear area	d_v = **16.00** in
Moment shear relationship, M/Vd	Assume M_Vd$_{ratio}$ = 1
Shear stress (8-24)	$f_v = V / A_{nv}$ = **25.8** psi
Allowable masonry shear stress (8-29)	$F_v = 1/2 \times (4.0 - 1.75 \times M\_Vd_{ratio}) \times \sqrt{(f'_m \times 1psi)}$
	F_v = **49.0** psi
Masonry shear stress ratio	f_v / F_v = **0.526**

PASS - Allowable shear stress exceeds shear stress in masonry

MASONRY LINTEL DESIGN TO TMS/MSJC 2013

Using the allowable stress design method

Tedds calculation version 1.2.01

Masonry details

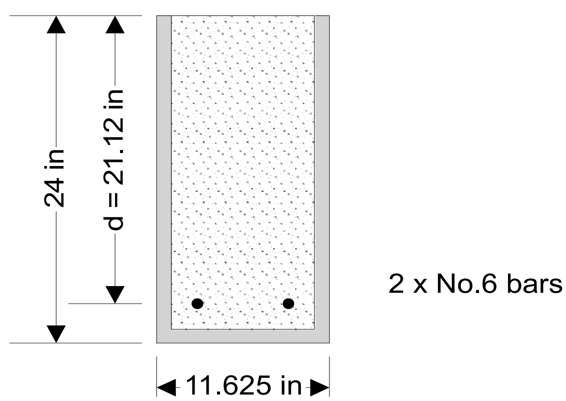
Masonry type	Concrete
Density of masonry unit	γ = 135 lb/ft^3
Pattern bond	Running
Mortar type	PCL Type M
Compressive strength of masonry unit	f'_{cu} = 1900.0 psi
Net compressive strength of masonry (Table 2)	f'_m = 1900.0 psi
Modulus of elasticity (4.2.2)	$E_m = 900 \times f'_m$ = **1710000** psi
Allowable flexural tensile stress (8.2.4.2)	F_t = 106.0 psi

Reinforcement details

Allowable tensile stress	F_s = 32000 psi
Modulus of elasticity of steel	E_s = 29000000 psi

Cover to reinforcement

Bottom cover to reinforcement	c_{nom_b} = 1.5 in
Side cover to reinforcement	c_{nom_s} = 1.5 in



Section properties

Modular ratio	$n = E_s / E_m$ = **16.96**
Section width	b = **11.625** in
Section depth	h = **24** in
Net shear area	$A_{nv} = b \times h$ = **279** in^2
Section modulus	$S = b \times h^2 / 6$ = **1116** in^3
Depth to tension reinforcement	d = **21.12** in

Flexure design (Chapter 8)

Tension reinforcement	2 x No. 6 bars
Area of tension reinforcement	$A_s = N_{bot} \times BarArea_{bot}$ = **0.88** in^2
Reinforcement ratio	$\rho_{ratio} = A_s / (b \times d)$ = **0.00358**
Neutral axis factor	$k = \sqrt{(2 \times \rho_{ratio} \times n + (\rho_{ratio} \times n)^2)} - \rho_{ratio} \times n$ = **0.293**
Lever arm factor	$j = 1 - k / 3$ = **0.902**
Cracking moment	$M_{cr} = 2.5 \times F_t \times S$ = **24.6** kip_ft

Design bending moment	M = **28.00** kip_ft
Tensile stress in reinforcement	$f_s = M / (A_s \times j \times d) =$ **20036** psi
Allowable tensile stress in reinf. (8.3.3.1)	$F_s =$ **32000** psi
Reinforcement stress ratio	$f_s / F_s =$ **0.626**

PASS - Allowable tensile stress exceeds tensile stress due to flexure

Compressive stress in masonry	$f_b = 2 \times M / (j \times k \times b \times d^2) =$ **490.0** psi
Allowable stress in masonry (8.3.4.2.2)	$F_b = 0.45 \times f'_m =$ **855.0** psi
Masonry stress ratio	$f_b / F_b =$ **0.573**

PASS - Allowable compressive stress exceeds compressive stress due to flexure

Shear design (Chapter 8)

Design shear force	V = **5.00** kips
Depth of shear area	$d_v =$ **24.00** in
Moment shear relationship, M/Vd	Assume $M\_Vd_{ratio} = 1$
Shear stress (8-24)	$f_v = V / A_{nv} =$ **17.9** psi
Allowable masonry shear stress (8-29)	$F_v = 1/2 \times (4.0 - 1.75 \times M\_Vd_{ratio}) \times \sqrt{(f'_m \times 1psi)}$
	$F_v =$ **49.0** psi
Masonry shear stress ratio	$f_v / F_v =$ **0.365**

PASS - Allowable shear stress exceeds shear stress in masonry

Summary

The lateral stability system of the project referenced above consists of metal deck diaphragms and CMU shear 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 wall system. Lateral resisting members directly transfer all lateral forces into the foundation system.

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.

Wind Load Distribution - Single Diaphragm Design, Envelope Method

General Building Info :

	Length	Wall Height to Roof	Parapet Height	Total Wall Height
Wall 1 =	220.00 ft	25.00 ft	3.00 ft	28.00 ft
Wall 2 =	58.00 ft	25.00 ft	3.00 ft	28.00 ft
Wall 3 =	220.00 ft	25.00 ft	3.00 ft	28.00 ft
Wall 4 =	58.00 ft	25.00 ft	3.00 ft	28.00 ft

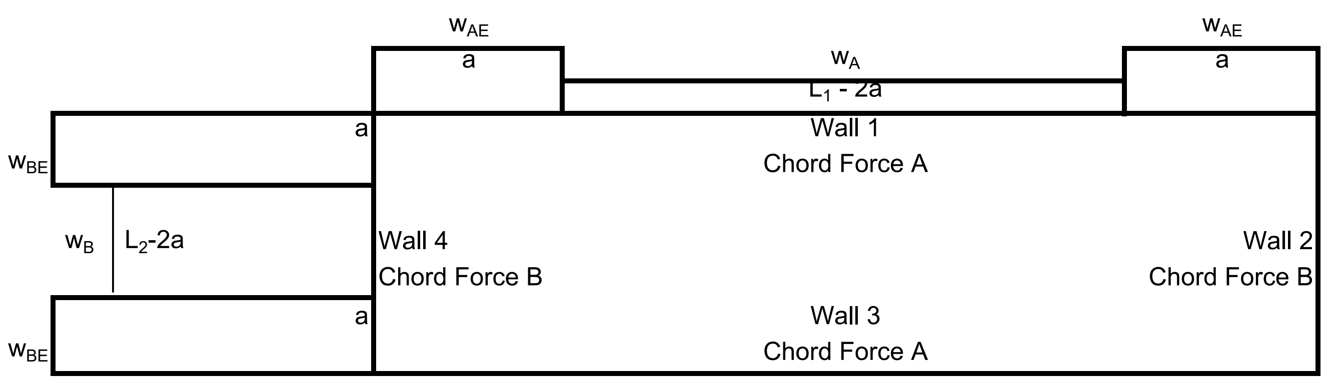
General Wind Loading Info :

ASD Factor =	0.6	(1 for ACSE 7-05, 0.6 for ASCE 7-10)
a =	7.25 ft	(Length of End Zone per ACSE)
Zone 1 Pressure =	5.60 psf	(Refer to Tedd's Calculations for Pressures)
Zone 1E Pressure =	10.90 psf	
Zone 4 Pressure =	11.90 psf	
Zone 4E Pressure =	15.40 psf	
WW Parapet Pressure =	39.89 psf	
LW Parapet Pressure =	26.59 psf	

Wind Loading @ Roof:

L_1 - 2a =	205.50 ft	
Distributed Force, w_A =	418.19 plf	(Pressures are Based on Entered Wind Pressures)
Distributed Force, w_{AE} =	528.19 plf	

L_2 - 2a =	43.50 ft	
Distributed Force, w_B =	418.19 plf	(Pressures are Based on Entered Wind Pressures)
Distributed Force, w_{BE} =	528.19 plf	



Shear Wall Loads:

Wall 1 =	7.8 k ASD	x Factor =	12.9 k LRFD	(Distributed Load x Effective Trib)
Wall 2 =	28.1 k ASD	x Factor =	46.8 k LRFD	(Distributed Load x Effective Trib)
Wall 3 =	7.8 k ASD	x Factor =	12.9 k LRFD	(Distributed Load x Effective Trib)
Wall 4 =	28.1 k ASD	x Factor =	46.8 k LRFD	(Distributed Load x Effective Trib)

56.15808	0.255264	0.246461793

STRUCTURAL ENGINEERS

Project Balderston Auto	Project No. 20-467		
Calc. By RJS	Checked By	Date 2/20/2021	

Diaphragm Loads:

Wall 1 =	35.3 plf ASD	x Factor =	58.8 plf LRFD	(Wall 1 Load*1000/Wall 1 Length)
Wall 2 =	484.1 plf ASD	x Factor =	806.9 plf LRFD	(Wall 2 Load*1000/Wall 2 Length)
Wall 3 =	35.3 plf ASD	x Factor =	58.8 plf LRFD	(Wall 3 Load*1000/Wall 3 Length)
Wall 4 =	484.1 plf ASD	x Factor =	806.9 plf LRFD	(Wall 4 Load*1000/Wall 4 Length)

Deck Design:

Deck =	22 GA Type B Metal Roof deck	(Per Vulcraft Catalog)
Weld Pattern =	36/7	(Per Vulcraft Catalog)
Sidelap Fasteners =	(2) #10 TEK Screws	(Per Vulcraft Catalog)
Allowable Load =	328.0 plf ASD	(Per Vulcraft Catalog)
Max Diaphragm Load =	484.1 plf ASD	
	N.G.	

Chord Loads:

Chord Force A =	26.2 k ASD	x Factor =	43.7 k LRFD	(M_{wind} against A / Wall B Length)
Chord Force B =	0.5 k ASD	x Factor =	0.8 k LRFD	(M_{wind} against B / Wall A Length)

Chord Design:

Chord A Design:		
Angle =	L3x3x1/2	
Angle Area =	2.8 in^2	(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	89.4 k LRFD	(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	39.7 k LRFD	(AISC Manual Table 4-11)
	N.G.	
Chord B Design:		
Angle =	L3x3x1/4	
Angle Area =	2.8 in^2	(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	89.4 k LRFD	(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	32.6 k LRFD	(AISC Manual Table 4-11)
	OK	

BSE STRUCTURAL ENGINEERS

Project Balderston Auto Project No. 20-467

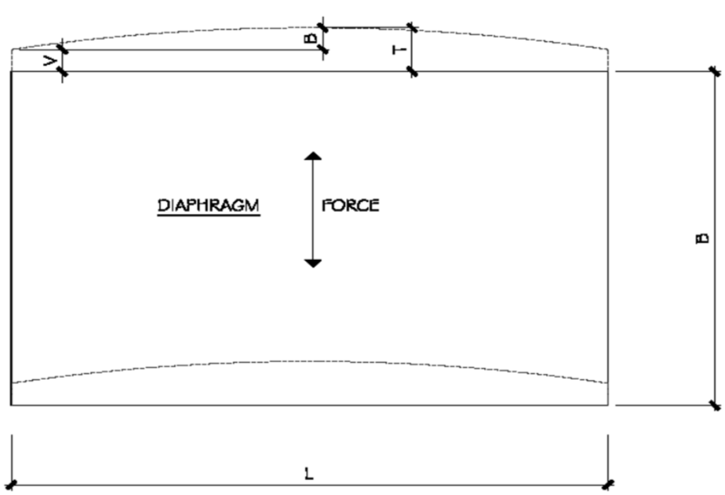
Calc. By RJS Checked By Date 2/20/2021

Diaphragm Deflection Calculations : Simply Supported

Span	Max Joist Spacing (ft)	L (ft)	B (ft)	E (ksi)	Chord Area A (in^2)
Wall 2 to 4	6	220.00	58.00	29000	2.76
Wall 1 to 3	6	58.00	220.00	29000	2.76

Values From Vulcraft Deck Catalog			Loading	Deflections		
K_2	DB	K_1	q(lb/ft)	ΔB (in)	ΔV (in)	ΔT (in)
870	129	0.204	250.914	0.68	0.418225305	1.10
870	129	0.204	250.914	0.00	0.007663484	0.01

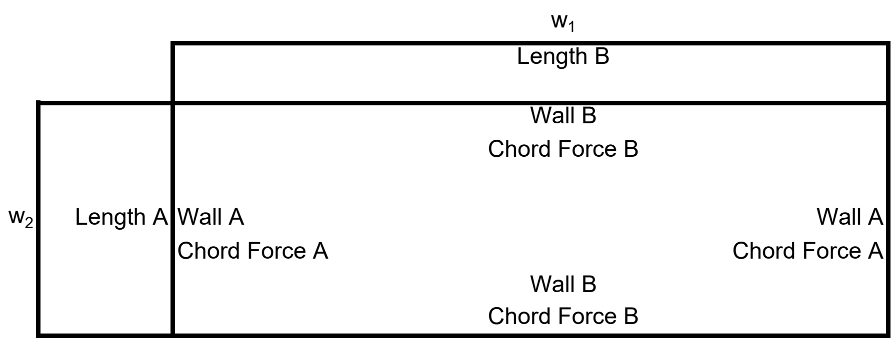
ΔB=	Deflection due to moment in inches	$5qL^4/384EI$
ΔV=	Deflection due to shear in inches	$qL^2/8BG'$
ΔT=	Total deflection in inches	$\Delta B+\Delta V$
q(lb/ft)=	distributed load on diaphragm	
K_1=	Factor from steel deck catalog	
DB=	Factor from steel deck catalog	
K_2 =	Factor from steel deck catalog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



Seismic Load Distribution - Single Diaphragm Design

General Info :

Seismic Response Coeff. C_s =	0.0475	(from Tedds Seismic Load Calculations)
Roof Dead Load =	15.00 psf	
Wall Length A =	183.00 ft	
Wall Weight A =	12.00 psf	
Wall Height A =	16.67 ft	(Roof to FFE)
Parapet Height A =	1.00 ft	(Top of Parapet to Roof)
Distributed Force, w_1 =	148.63 plf	((Length A*Dead+(0.5*Height B+Parapet)*Weight B) * C_S)
Wall Length B =	225.00 ft	
Wall Weight B =	12.00 psf	
Wall Height B =	16.00 ft	(Roof to FFE)
Parapet Height B =	8.00 ft	(Top of Parapet to Roof)
Distributed Force, w_2 =	170.95 plf	((Length B*Dead+(0.5*Height A+Parapet)*Weight A) * C_S)



Shear Wall Loads from Diaphragm:

Wall A =	11.7 k ASD	/0.7 =	16.7 k LRFD	((0.7*w_1*Length B*0.5)/1000)
Wall B =	10.9 k ASD	/0.7 =	15.6 k LRFD	((0.7*w_2*Length A*0.5)/1000)

Total Shear Wall Loads (Includes Wall Weight):

Wall A =	13.0 k ASD	/0.7 =	18.6 k LRFD	(Dia. A+Length A*Weight A*Height A*C_S)
Wall B =	13.1 k ASD	/0.7 =	18.7 k LRFD	(Dia. B+Length B*Weight B*Height B*C_S)

Diaphragm Loads:

Wall A =	64.0 plf ASD	/0.7 =	91.4 plf LRFD	(Wall A Load*1000/Wall A Length)
Wall B =	48.7 plf ASD	/0.7 =	69.5 plf LRFD	(Wall B Load*1000/Wall B Length)

Deck Design:

Deck =	22 GA Type B Metal Roof deck	(Per Vulcraft Catalog)
Weld Pattern =	36/7	(Per Vulcraft Catalog)
Sidelap Fasteners =	(2) #10 TEK Screws	(Per Vulcraft Catalog)
Allowable Load =	328.0 plf ASD	(Per Vulcraft Catalog)
Max Diaphragm Load =	64.0 plf ASD	
	OK	

Chord Loads:

Chord Force A =	2.2 k ASD	/0.7 =	3.2 k LRFD (M_{wind} against A / Wall B Length)
Chord Force B =	3.6 k ASD	/0.7 =	5.1 k LRFD (M_{wind} against B / Wall A Length)

Chord Design:

Chord A Design:			
Angle =	L3x3x1/4		
Angle Area =	1.4 in^2		(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	46.7 k LRFD		(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	20.9 k LRFD	k	(AISC Manual Table 4-11)
	OK		
Chord B Design:			
Angle =	L3x3x1/4		
Angle Area =	1.4 in^2		(AISC Manual Table 1-7)
Tensile Capacity, ΦT =	46.7 k LRFD		(AISC 360 Eq. D2-1ΦT = 0.9*Area *36 ksi)
Comp. Capacity, ΦP =	32.6 k LRFD	k	(AISC Manual Table 4-11)
	OK		

MASONRY WALL PANEL DESIGN TO TMS 402/602-16

Using the allowable stress design method

Tedds calculation version 2.2.04

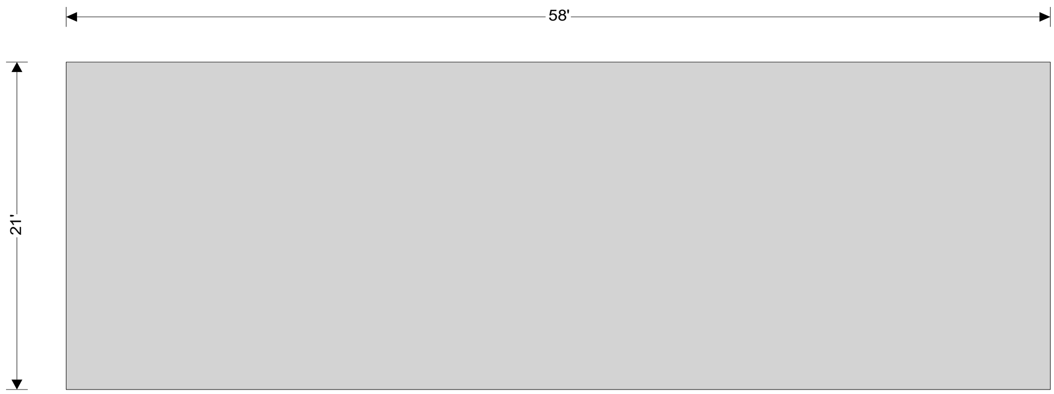
Masonry wall panel details

Solid masonry wall without parapet - Reinforced single-wythe wall, the wall is pinned at the top and fixed at the bottom for out of plane loads

The wall is fixed at the bottom and free at the top for in plane loads

Panel length L = **58** ft

Panel height h = **21** ft



Seismic properties

Seismic design category A

Seismic importance factor (ASCE7 Table 1.5-2) I_e = **1**

Design spectral response acceleration parameter, short periods (ASCE7 11.4.4)

S_{DS} = **1**

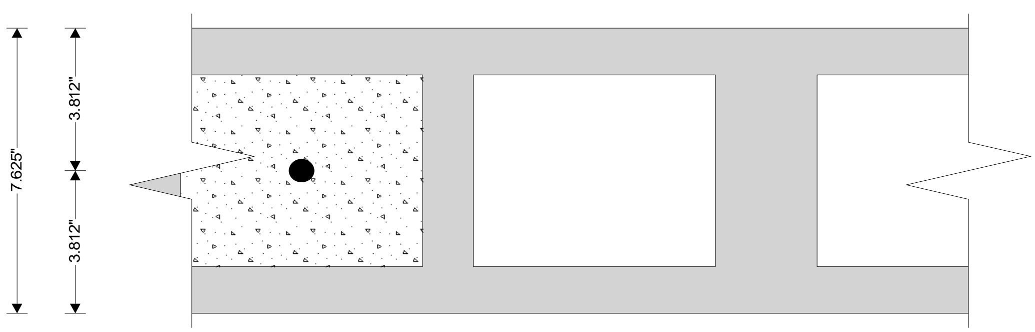
Seismic wall classification Nonparticipating

No prescriptive minimum seismic reinforcement

Redundancy factor, on out-of-plane load ρ_E = **1.0**

Construction details

Wall thickness t = **7.625** in



Masonry details

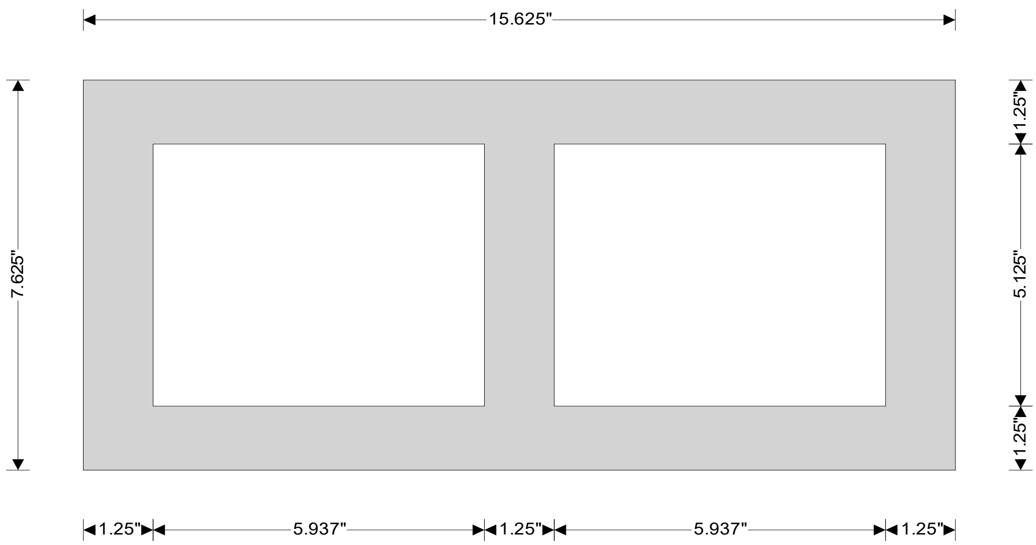
Hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class M mortar

Compressive strength of unit f'_{cu} = **2800** psi

Density of masonry units γ_{block} = **115** lb/ft^3

Height of masonry units h_b = **7.625** in

Length of masonry units	l_b = **15.625** in
Number of internal webs	N_{web} = **1**
Number of end webs	N_{end} = **2**
Internal web thickness	t_{bw} = **1.25** in
Face shell thickness	t_{bf} = **1.25** in
End web thickness	t_{be} = **1.25** in
Area of block	$A_{block} = [t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **44.76** in^2/ft
Area of grout	$A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b$ = **7.95** in^2/ft
Density of grout	γ_{grout} = **140** lb/ft^3
Self weight of wall	$w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout}$ = **43.47** psf



From TMS 602-16 Table 2 - Compressive strength of masonry

Net compressive strength of masonry	f'_m = **2250** psi
Modulus of elasticity for masonry	$E_m = 900 \times f'_m$ = **2024584** psi
Shear modulus of masonry	$G_v = 0.4 \times E_m$ = **809834** psi

From TMS 402 -16 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry

Allowable flexural tensile stress normal to bed	F_{t_norm} = **38** psi
Allowable flexural tensile stress parallel to bed	F_{t_para} = **66** psi

Reinforcement details

Yield strength of reinforcement	f_y = **60000** psi
Allowable tensile stress in reinforcement	F_s = **32000** psi
Modulus of elasticity for reinforcement	E_s = **29000000** psi
Vertical reinforcement provided	No.5 bars at 48 in centers
Area of vertical reinforcement	$A_s = \pi \times Dia^2 / (4 \times s)$ = **0.08** in^2/ft

Lateral out-of-plane loads

Wind load on panel	W = **5** psf
Wind load on parapet	W_p = **18** psf
Seismic load factor (ASCE7 12.11.1)	$F_p = 0.4 \times S_{DS} \times I_e$ = **0.4**
Seismic load from wall	$E_{wall} = \max(F_p, 0.1) \times w_{SW}$ = **17.4** psf
Additional seimic load	E_{add} = **0** psf
Seismic lateral load on panel	$E = E_{wall} + E_{add}$ = **17.4** psf

Lateral in-plane loads

Vertical loading details

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

From ASCE 7-16 cl.2.4 - Combining nominal loads using allowable stress design (Utilization)

Load combination no.1 DL (0.059)

Properties of masonry section

Cross-sectional area $A = [t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b =$ **52.7** in^2/ft

Properties for walls loaded out-of-plane:

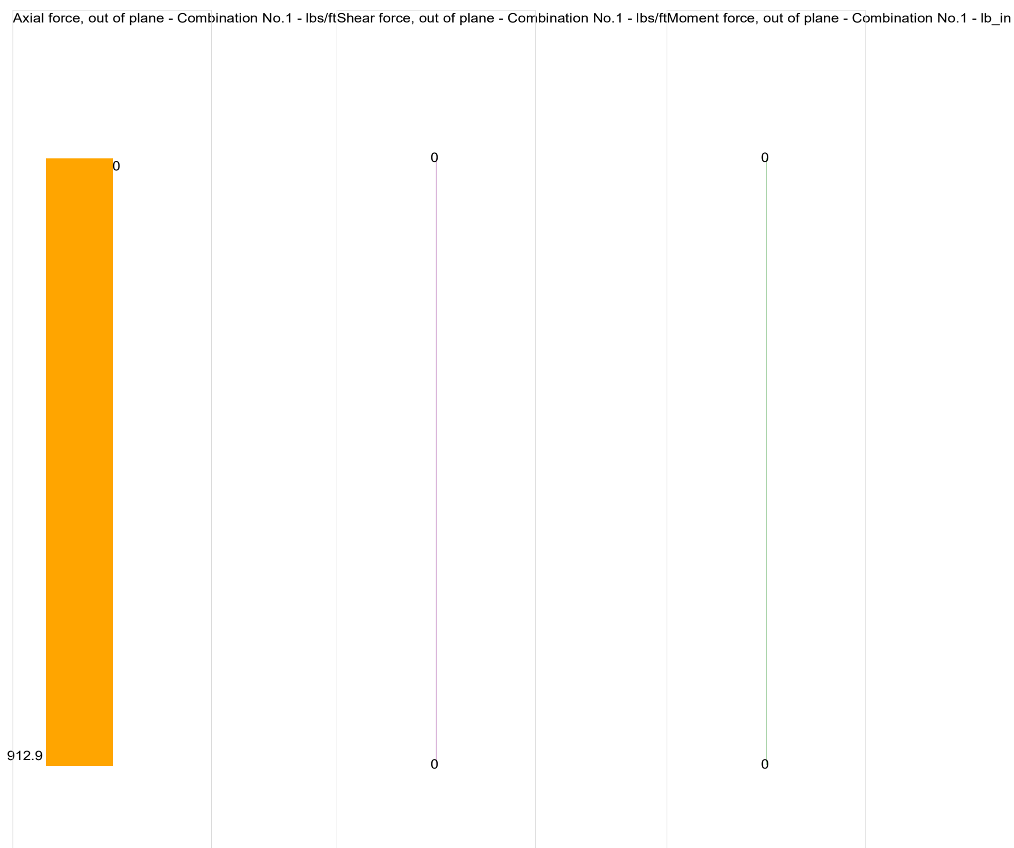
Moment of inertia $I = t^3 / 12 - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) =$ **358.4** in^4/ft

Section modulus $S = I / c =$ **94** in^3/ft

Radius of gyration $r = \sqrt{[I / A]} =$ **2.608** in

Effective height factor $K =$ **1**

Consider wall at bottom under load combination no.1



Axial load at bottom of panel $P =$ **913** lb/ft

Compressive stress due to axial load $f_a = P / A =$ **17.3** psi

Slenderness ratio $(K \times h) / r =$ **96.636** < 99

Allowable compressive stress due to axial load $F_a = (1 / 4) \times f'_m \times [1 - ((K \times h) / (140 \times r))^2] =$ **294.4** psi

$f_a / F_a =$ **0.059**

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

Allowable compressive force $P_a = 0.25 \times f'_m \times (A - A_s) \times [1 - ((K \times h) / (140 \times r))^2] =$ **15496** lb/ft

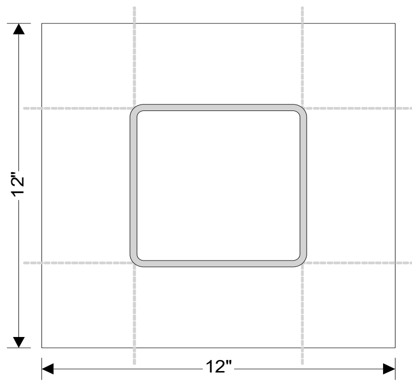
$P / P_a =$ **0.059**

PASS - Allowable compressive force exceeds axial load

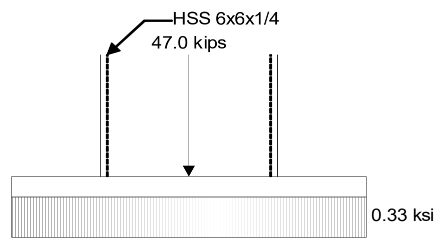
COLUMN BASE PLATE DESIGN

In accordance with AISC 360-05

Tedds calculation version 2.1.01



Plan on baseplate



Elevation on baseplate

Design forces and moments

Axial force	P_u = **47.0** 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 = **34.000** in
Dist center of baseplate to left edge foundation	x_{ce1} = **48.000** in
Dist center of baseplate to right edge foundation	x_{ce2} = **48.000** in
Dist center of baseplate to bot edge foundation	y_{ce1} = **48.000** in
Dist center of baseplate to top edge foundation	y_{ce2} = **48.000** in
Minimum tensile strength, base plate	F_y = **36** ksi
Minimum tensile strength, column	F_{yCol} = **50** ksi
Compressive strength of concrete	f'_c = **4** ksi

Strength reduction factors

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

Plate cantilever dimensions

Area of base plate	$A_1 = B \times N =$ **144.000** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min}) \times (B + 2 \times l_{min}) =$ **9216.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2) =$ **979.2** 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	$t_{p,req} = l \times \sqrt{((2 \times P_u) / (\phi_b \times F_y \times B \times N))} =$ **0.447** 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	$P_p =$ **979.20** kips
Factored bearing strength	$\phi_c P_p =$ **587.52** kips

PASS - Allowable bearing stress exceeds applied bearing stress

Flange weld

Flange weld leg length	$t_{wf} =$ **0.3126** in
Tension capacity of flange	$P_{tf} = b_f \times t \times F_{yCol} =$ **69.9** kips
Force in tension flange	$F_{tf} = M_u / (d - t) - P_u \times (b_f \times t) / A_{col} =$ **-12.5** 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	$\phi F_w = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90\text{deg}))^{1.5}) =$ **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_w \times t_{wf} / \sqrt{(2)} =$ **10.4** kips/in

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

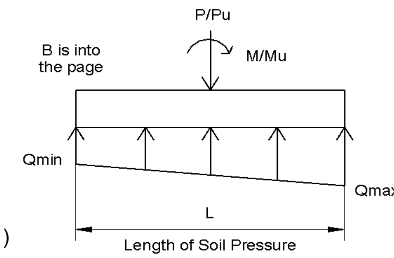
Summary

The gravity structure system of the project referenced above consists primarily of steel joists and girders, load-bearing CMU walls and steel columns. The load bearing walls are supported at grade by continuous shallow foundations. Interior steel framing is supported by steel columns. All columns are supported by shallow spread foundations. The locations of all footings are indicated on the structural foundation plans.

The following section of calculations covers the complete design of the foundation system for project referenced above, including the design of retaining walls located on the site. Refer to the “Loads” section of these calculations for the determination of all dead, live, roof live, and snow loads. Refer to the “Roof Framing” and “Tilt Panel” section of these calculations for the design of the members being supported by the continuous and spread footings.

Footing Designation: F1

Footing Location: Interior w/ solar



General Information:

Footing Length, L =	5	ft			
Footing Width, B =	5	ft			
Footing Depth, H =	12	in			
Location =	Interior	in			
Steel Depth, d =	8.0625	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	5	in			
Slab Depth Above Footing =	8	in			
Area of Footing =	25	ft^2			(B*L)
Soil Bearing Pressure =	2.5	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 =	9.7	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	5.3125	k	Live =	1.6	
Applied Live Load =	10.3	k	Uplift=	1.6	
ASD Total Load, P =	25.3125	k			
LRFD Total Load, Pu =	34.495	k			
ASD Uplift Load =	6.7	k			
LRFD Uplift Load =	10.72	k			

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.833	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.013	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.013	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 =	0.833	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	5.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.380	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.380	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	1.380	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.706	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 =	11.77	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	39.74	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 =	19.06	in	(Critical Section B + d)
b2 =	19.06	in	(Column Height L + d)
b0 =	76.25	in	(2*b1 + 2*b2)
Vu2 =	31.01	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	151.52	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	157.32	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	101.02	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

ΦPn =	649.74	k	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Plate Area*2))
Adequate in Bearing?	**YES**		

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	6.7	k	(From Above)
Required Dead Load =	11.17	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	15.0125	k	
Additional Slab Used =	0	ft	(Length of Additional Slab in Each Direction)
Wall Weight Over Footing =	0	klf	
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 Used =	0	ft	
Total Dead Load =	15.0125	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 =	0.62	k-ft / ft	(Mu = (Pu/A)*0.5*Crit. L^2)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.010	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0002		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.022	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	4		
Bar Spacing =	12	in	
As Provided =	0.20	in^2/ft	= 6 Bars in B Direction
			= 6 Bars in L Direction

Bottom Steel:

Mu =	2.01	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 =	0.034	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0006		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0008		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0008		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.074	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	12	in	
As Provided =	0.31	in^2/ft	= 6 Bars in B Direction
			= 6 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.2592	in^2/ft	(T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.20	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.51	in^2/ft	
T&S Steel Provided?	**YES**		

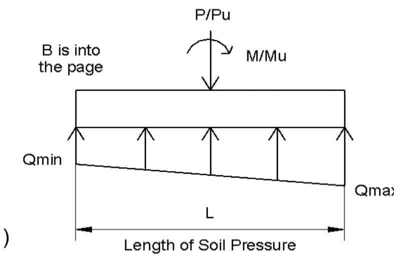
Final Footing Design:

Footing Width, B =	**5**	ft
Footing Length L =	**5**	ft
Footing Depth, H =	**12**	in

Top Steel =	**#4 bars**	**@12 inches O.C.**
Bottom Steel =	**#5 bars**	**@12 inches O.C.**

Footing Designation F2

Footing Location: Interior at Mezzanine



General Information:

Footing Length, L =	8	ft			
Footing Width, B =	8	ft			
Footing Depth, H =	12	in			
Location =	Interior	in			
Steel Depth, d =	8.0625	in			(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	4	in			
Slab Depth Above Footing =	8	in			
Area of Footing =	64	ft^2			(B*L)
Soil Bearing Pressure =	2.5	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 =	45	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	12.8	k	Live =	1.6	
Applied Live Load =	51	k	Uplift=	1.6	
ASD Total Load, P =	108.8	k			
LRFD Total Load, Pu =	150.96	k			
ASD Uplift Load =	0	k			
LRFD Uplift Load =	0	k			

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	1.333	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	8.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	1.700	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.700	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 =	1.333	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	8.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	2.359	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	2.359	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	2.359	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	3.289	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 =	62.06	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	63.59	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 =	17.06	in	(Critical Section B + d)
b2 =	17.06	in	(Column Height L + d)
b0 =	68.25	in	(2*b1 + 2*b2)
Vu2 =	87.71	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	135.63	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	152.02	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	90.42	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

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

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	0	k	(From Above)
Required Dead Load =	0.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	57.8	k	
Additional Slab Used =	4	ft	(Length of Additional Slab in Each Direction)
Wall Weight Over Footing =	0	klf	
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 Used =	0	ft	
Total Dead Load =	67.4	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 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	4		
Bar Spacing =	12	in	
As Provided =	0.20	in^2/ft	= 9 Bars in B Direction
			= 9 Bars in L Direction

Bottom Steel:

Mu =	12.76	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 =	0.218	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0038		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0051		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0038		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.368	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	12	in	
As Provided =	0.31	in^2/ft	= 9 Bars in B Direction
			= 9 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.2592	in^2/ft	(T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.20	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.51	in^2/ft	
T&S Steel Provided?	**YES**		

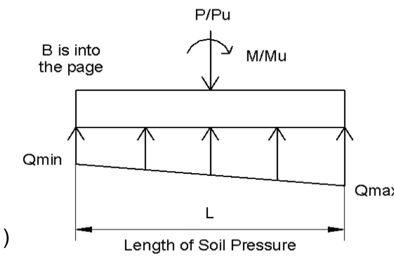
Final Footing Design:

Footing Width, B =	**8**	ft
Footing Length L =	**8**	ft
Footing Depth, H =	**12**	in

Top Steel =	**#4 bars**	**@12 inches O.C.**
Bottom Steel =	**#5 bars**	**@12 inches O.C.**

Footing Designation F3

Footing Location: Interior at Mezzanine



General Information:

Footing Length, L =	4	ft		
Footing Width, B =	4	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 =	5	in		
Slab Depth Above Footing =	8	in		
Area of Footing =	16	ft^2		(B*L)
Soil Bearing Pressure =	2.5	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 =	9.7	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	7.8	k	Live =	1.6	
Applied Live Load =	10.3	k	Uplift=	1.6	
ASD Total Load, P =	27.8	k			
LRFD Total Load, Pu =	37.48	k			
ASD Uplift Load =	0	k			
LRFD Uplift Load =	0	k			
Moments:			LRFD Factors:		
Dead Load Moment =	0	k-ft	Dead =	1.2	(ASCE 7 Combo)
Live Load Moment =	0	k-ft	Wind =	1.6	
ASD Total Moment, M =	0	k-ft			
LRFD Total Moment, Mu =	0	k-ft			

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	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 =	1.738	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.738	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(ASD M / Pu)
Kern =	0.667	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	2.343	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	2.343	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	2.343	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	0.372	ft	"Greater Than", Qmax = (4*Pu) / (3*B*(L - 2*e))
			(Qcritical = pressure @ critical section of footing)
			(Critical Length = L/2 - Critical Section/2-d/2)

One-Way Shear Check:

Vu1 =	3.49	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	118.55	k	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		

Two-Way Shear Check:

b1 =	39.06	in	(Critical Section B + d)
b2 =	39.06	in	(Column Height L + d)
b0 =	156.25	in	(2*b1 + 2*b2)
Vu2 =	12.66	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	1157.76	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	1870.94	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	771.84	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

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

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	0	k	(From Above)
Required Dead Load =	0.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	17.5	k	
Additional Slab Used =	4	ft	(Length of Additional Slab in Each Direction)
Wall Weight Over Footing =	0	klf	
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 Used =	0	ft	
Total Dead Load =	25.5	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 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	10	in	
As Provided =	0.37	in^2/ft	= 6 Bars in B Direction
			= 6 Bars in L Direction

Bottom Steel:

Mu =	0.16	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 =	0.000	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.002	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	5		
Bar Spacing =	10	in	
As Provided =	0.37	in^2/ft	= 6 Bars in B Direction
			= 6 Bars in L Direction

Temperature & Shrinkage Steel:

Minimum Steel =	0.7344	in^2/ft	(T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.37	in^2/ft	
As Provided Bott =	0.37	in^2/ft	
As Provided Total =	0.74	in^2/ft	
T&S Steel Provided?	**YES**		

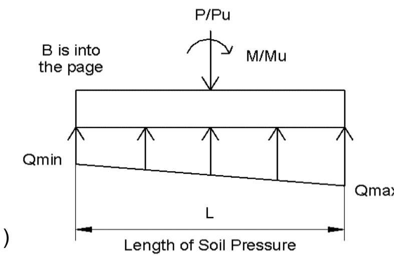
Final Footing Design:

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

Top Steel =	**#5 bars**	**@10 inches O.C.**
Bottom Steel =	**#5 bars**	**@10 inches O.C.**

Footing Designation F4

Footing Location:



General Information:

Footing Length, L =	4	ft	
Footing Width, B =	4	ft	
Footing Depth, H =	12	in	
Location =	Interior	in	
Steel Depth, d =	8.0625	in	(H - 3 in - 1.5*Bar Dia.)
Typical Slab Depth =	5	in	
Slab Depth Above Footing =	8	in	
Area of Footing =	16	ft^2	(B*L)
Soil Bearing Pressure =	2.5	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 =	9.7	k	Dead =	1.2	(ASCE 7 Combo)
Slab + Wall +Footing Weight =	3.4	k	Live =	1.6	
Applied Live Load =	10.3	k	Uplift=	1.6	
ASD Total Load, P =	23.4	k			
LRFD Total Load, Pu =	32.2	k			
ASD Uplift Load =	0	k			
LRFD Uplift Load =	0	k			

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

ASD Soil Pressures:

e =	0.000	ft	(ASD M / P)
Kern =	0.667	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 =	1.463	ksf	("Less Than",Qmin = (P/L*B) - (6*M / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	1.463	ksf	("Less Than",Qmax = (P/L*B) + (6*M / B*L^2),
Is Qmax<SBC?	**YES**		"Equal To", Qmax = (2*P) / (L*B)
			"Greater Than", Qmax = (4*P) / (3*B*(L - 2*e))

LRFD Soil Pressures:

e =	0.000	ft	(ASD M / Pu)
Kern =	0.667	ft	(L / 6)
e > = < Kern ?	Less Than		
Length of Pressure =	4.000	ft	("Greater Than", Length = 3*(L/2 -e) ; Otherwise = L)
Minimum Pressure, Qmin =	2.013	ksf	("Less Than",Qmin = (Pu/L*B) - (6*Mu / B*L^2), Otherwise = 0)
Maximum Pressure, Qmax =	2.013	ksf	("Less Than",Qmax = (Pu/L*B) + (6*Mu / B*L^2),
Qcritical =	2.013	ksf	"Equal To", Qmax = (2*Pu) / (L*B)
Critical Length =	1.289	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 =	10.38	k	(Qcrit*Crit.L + (Qmax-Qcrit)*Crit L*0.5)
ΦVn =	31.80	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 =	17.06	in	(Critical Section B + d)
b2 =	17.06	in	(Column Height L + d)
b0 =	68.25	in	(2*b1 + 2*b2)
Vu2 =	28.13	k	(Vu2 = (Qmax+Qmin)/2 * (Ftg Area - b1*b2))
α =	40		(ACI 318-08 Section 11.11.2.1)
β =	1		(ACI 318-08 Section 11.11.2.1, Larger Ftg Dim / Smaller Ftg Dim)
ΦVn =	135.63	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+4/Beta))
ΦVn =	152.02	k	(ACI 318-08 Eq 11-32, ΦVn = 0.75*sqrt(f'c)*bo*d*(2+Alpha*d/bo))
ΦVn =	90.42	k	(ACI 318-08 Eq 11-33, ΦVn = 0.75*4*sqrt(f'c)*bo*d)
Adequate in Two-Way Shear?	**YES**		

Column Bearing Check:

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

Uplift Check:

ASD Combo for Uplift =	0.6D + Uplift		(ASCE 7)
Uplift Force =	0	k	(From Above)
Required Dead Load =	0.00	k	(Uplift / 0.6)
Applied Dead Load + Slab + Ftg =	13.1	k	
Additional Slab Used =	4	ft	(Length of Additional Slab in Each Direction)
Wall Weight Over Footing =	0	klf	
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 Used =	0	ft	
Total Dead Load =	21.1	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 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0000		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0000		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.000	in^2/ft	(As = Governing ρ*12 inches*d)
Bar # =	4		
Bar Spacing =	12	in	
As Provided =	0.20	in^2/ft	= 5 Bars in B Direction
			= 5 Bars in L Direction

Bottom Steel:

Mu =	1.67	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 =	0.029	ksi	(Ru = Mu/(0.9*12 inches*d^2))
ρ Req'd =	0.0005		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0006		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0006		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.062	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 =	0.2592	in^2/ft	(T&S Steel = 0.0018*12 inches*H)
As Provided Top =	0.20	in^2/ft	
As Provided Bott =	0.31	in^2/ft	
As Provided Total =	0.51	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

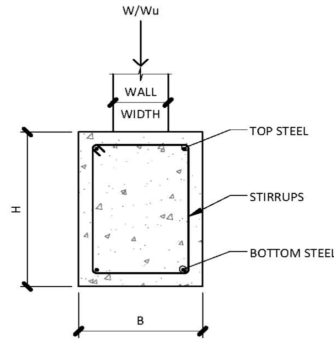
Footing Width, B =	**4**	ft
Footing Length L =	**4**	ft
Footing Depth, H =	**12**	in

Top Steel =	**#4 bars**	**@12 inches O.C.**
Bottom Steel =	**#5 bars**	**@12 inches O.C.**

Project	Balderston Auto	Project No.	20-467
Calc. By	RJS	Checked By	JH
Date	03/01/21		

Grade Beam Design

Grade Beam Location: Thickened Slab NLB 8" CMU



General Information:

Footing Width, B =	18	in	
Footing Depth, H =	12	in	
Steel Depth, d =	8.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	8.625	in	
Soil Bearing Pressure =	2.5	ksf	
Allowable or Effective SBC?	Allowable		
Concrete Strength =	3	ksi	

Loading:

Vertical Loads:			LRFD Factors: (ASCE 7 Combo)
Applied Dead Load =	0.9	klf	
Wall Weight =	56	psf	Dead = 1.2
Wall Height =	22	ft	Live = 1.6
Total Wall Weight =	1.232	klf	
Footing Weight =	0.225	klf	
Applied Live Load =	0.10	klf	
ASD Total Load, W =	2.457	klf	
LRFD Total Load, Wu =	2.9884	klf	

ASD Soil Pressures:

Required Footing Width =	0.9828	ft	(Footing Width = W / Soil Bearing Pressure)
Chosen Footing Width =	1.5	ft	
Assumed Footing Span =	6	ft	
Is Footing Width Adequate?	**YES**		

One-Way Shear Check:

Vu1 =	8.97	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	11.92	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	YES		(ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otherwise "Yes")
	Calculate Stirrups		(Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	28.591875	klf	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall Width*1'*2))
Adequate in Bearing?	**YES**		

Bottom Steel:

Mu =	13.45	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.153	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0026		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0035		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0027		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.397	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 =	2	bars	
As Provided =	0.62	in^2	

Temperature & Shrinkage Steel:

Minimum Steel =	0.3888	in^2/ft	(T&S Steel = 0.0018*B*H)
As Provided Top =	0.62	in^2/ft	
As Provided Bott =	0.93	in^2/ft	
As Provided Total =	1.55	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

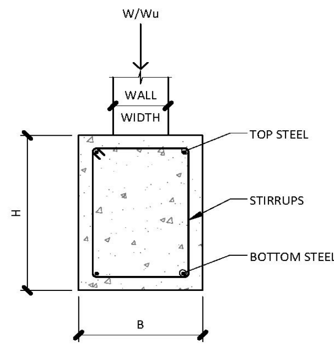
Footing Width, B =	**18**	in
Footing Depth, H =	**12**	in

Top Steel =	**(5) #5 bars**
Bottom Steel =	**(3) #5 bars**
Stirrups =	**Calculate Stirrups**

Footing Design

Grade Beam Design

Grade Beam Location: NLB w/ 12" CMU



General Information:

Footing Width, B =	30	in	
Footing Depth, H =	34	in	
Steel Depth, d =	30.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	11.625	in	
Soil Bearing Pressure =	2.5	ksf	
Allowable or Effective SBC?	Allowable		
Concrete Strength =	3	ksi	

Loading:

Vertical Loads:

LRFD Factors: (ASCE 7 Combo)

Applied Dead Load =	0.05	klf		
Wall Weight =	83	psf	Dead =	1.2
Wall Height =	27	ft	Live =	1.6
Total Wall Weight =	2.241	klf		
Footing Weight =	1.0625	klf		
Applied Live Load =	0.13	klf		
ASD Total Load, W =	3.48575	klf		
LRFD Total Load, Wu =	4.2358	klf		

ASD Soil Pressures:

Required Footing Width =	1.3943	ft	(Footing Width = W / Soil Bearing Pressure)
Chosen Footing Width =	2.5	ft	
Assumed Footing Span =	6	ft	
Is Footing Width Adequate?	**YES**		

One-Way Shear Check:

Vu1 =	12.71	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	74.10	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otherwise "Yes")
	Use #3 Stirrups at 18 in. O.C.		(Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	38.536875	klf	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall Width*1'*2))
Adequate in Bearing?	**YES**		

Bottom Steel:

Mu =	19.06	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.009	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0002		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.188	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

Footing Design

Temperature & Shrinkage Steel:

Minimum Steel =	1.836	in^2/ft	(T&S Steel = 0.0018*B*H)
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:

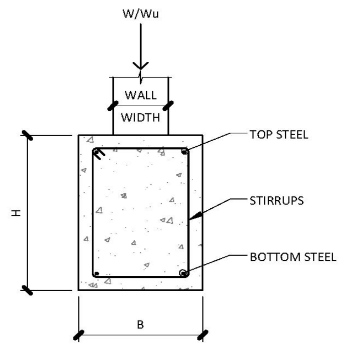
Footing Width, B =	**30**	in
Footing Depth, H =	**34**	in

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

Footing Design

Grade Beam Design

Grade Beam Location: Mezzanine



General Information:

Footing Width, B =	42	in	
Footing Depth, H =	34	in	
Steel Depth, d =	30.0625	in	(H - 3 in - 1.5*Bar Dia.)
Wall Width =	11.625	in	
Soil Bearing Pressure =	2.5	ksf	
Allowable or Effective SBC?	Allowable		
Concrete Strength =	3	ksi	

Loading:

Vertical Loads:			LRFD Factors: (ASCE 7 Combo)
Applied Dead Load =	1.4	klf	
Wall Weight =	83	psf	Dead = 1.2
Wall Height =	24	ft	Live = 1.6
Total Wall Weight =	1.992	klf	
Footing Weight =	1.4875	klf	
Applied Live Load =	1.50	klf	
ASD Total Load, W =	6.3795	klf	
LRFD Total Load, Wu =	8.2554	klf	

ASD Soil Pressures:

Required Footing Width =	2.5518	ft	(Footing Width = W / Soil Bearing Pressure)
Chosen Footing Width =	3.5	ft	
Assumed Footing Span =	6	ft	
Is Footing Width Adequate?	**YES**		

One-Way Shear Check:

Vu1 =	24.77	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	103.74	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otherwise "Yes")
	Use #3 Stirrups at 18 in. O.C.		(Provide minimum stirrups to support steel)

Wall Bearing Check:

ΦPn =	38.536875	klf	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall Width*1'*2))
Adequate in Bearing?	**YES**		

Bottom Steel:

Mu =	37.15	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.013	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0003		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0003		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.366	in^2	(As = Governing ρ*B*d)
Bar # =	5		
Number of Bars =	5	bars	
As Provided =	1.55	in^2	
Is Steel Adequate ?	**YES**		

Top Steel:

Bar # =	5	
Number of Bars =	5	bars
As Provided =	1.55	in^2

Footing Design

Temperature & Shrinkage Steel:

Minimum Steel =	2.5704	in^2/ft	(T&S Steel = 0.0018*B*H)
As Provided Top =	1.55	in^2/ft	
As Provided Bott =	1.55	in^2/ft	
As Provided Total =	3.10	in^2/ft	
T&S Steel Provided?	**YES**		

Final Footing Design:

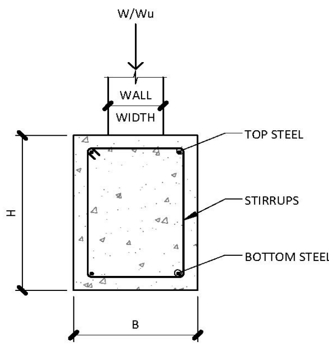
Footing Width, B =	**42**	in
Footing Depth, H =	**34**	in

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

Footing Design

Grade Beam Design

Grade Beam Location: 12" CMU



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 =	11.625	in	
Soil Bearing Pressure =	2.5	ksf	
Allowable or Effective SBC?	Allowable		
Concrete Strength =	3	ksi	

Loading:

Vertical Loads:

LRFD Factors: (ASCE 7 Combo)

Applied Dead Load =	0.24	klf		
Wall Weight =	83	psf	Dead =	1.2
Wall Height =	27	ft	Live =	1.6
Total Wall Weight =	2.241	klf		
Footing Weight =	0.85	klf		
Applied Live Load =	0.40	klf		
ASD Total Load, W =	3.731	klf		
LRFD Total Load, Wu =	4.6372	klf		

ASD Soil Pressures:

Required Footing Width =	1.4924	ft	(Footing Width = W / Soil Bearing Pressure)
Chosen Footing Width =	2	ft	
Assumed Footing Span =	6	ft	
Is Footing Width Adequate?	**YES**		

One-Way Shear Check:

Vu1 =	13.91	klf	(Wu*Assumed Footing Span / 2)
ΦVn =	59.28	klf	(ACI 318-08 Equation 11-5, ΦVn = 0.75*2*sqrt(f'c)*B*d / 1000)
Adequate in One-Way Shear?	**YES**		
Are Stirrups Req'd?	NO		(ACI 318-08 Section 11.4.6.1 If ΦVn/2 >Vu1 "No", Otherwise "Yes")

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

Wall Bearing Check:

ΦPn =	38.536875	klf	(ACI 318-08 Section 10.14.1 ΦPn = 0.65*0.85*f'c*Wall Width*1'*2))
Adequate in Bearing?	**YES**		

Bottom Steel:

Mu =	20.87	k-ft	(Wu*Assumed Footing Span^2 / 8)
m =	23.529		(m = fy/(0.85*f'c))
Ru =	0.013	ksi	(Ru = Mu/(0.9*B*d^2))
ρ Req'd =	0.0002		(ρ = (1/m)*(1-sqrt(1-2*Ru*m/fy)))
ρ Min. =	0.0027		(ACI 318-08 Equation 10-3, Smaller of: 3*sqrt(f'c)/fy & 200/fy)
4/3*Mu ρ Req'd =	0.0003		(ρ = (1/m)*(1-sqrt(1-2*1.33*Ru*m/fy)))
Governing ρ =	0.0003		(If ρ Req'd < 4/3*Mu ρ Req'd < ρ Min, Use 4/3*Mu ρ Req'd
A's Required =	0.206	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

Footing Design

Temperature & Shrinkage Steel:

Minimum Steel =	1.4688	in^2/ft	(T&S Steel = 0.0018*B*H)
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 18 in. O.C.**