Creech DRP Phase 2

Aircraft Maintenance Facility (AMF)

Structural Calculations

Michael Baker INTERNATIONAL

Structural Calculation Index

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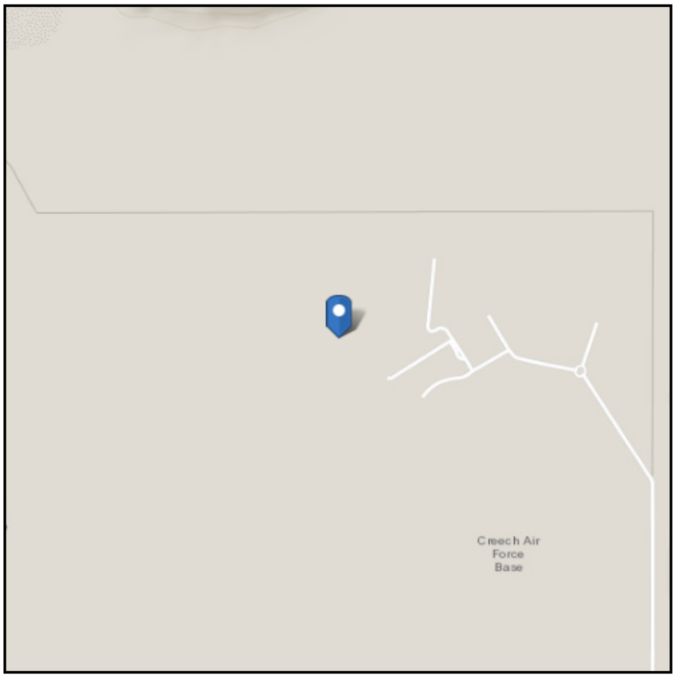
Criteria and Model Information

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-22	**Latitude:**	36.592935
Risk Category:	III	**Longitude:**	-115.66473
Soil Class:	D - Stiff Soil	**Elevation:**	3096.745107028911 ft (NAVD 88)



Wind

Results:

Wind Speed	105 Vmph
10-year MRI	69 Vmph
25-year MRI	75 Vmph
50-year MRI	80 Vmph
100-year MRI	84 Vmph
300-year MRI	92 Vmph
700-year MRI	98 Vmph
1,700-year MRI	105 Vmph
3,000-year MRI	108 Vmph
10,000-year MRI	117 Vmph
100,000-year MRI	135 Vmph
1,000,000-year MRI	153 Vmph

Data Source:	ASCE/SEI 7-22, Fig. 26.5-1C and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Wed Jun 18 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-22 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years). Values for 10-year MRI, 25-year MRI, 50-year MRI and 100-year MRI are Service Level wind speeds, all other wind speeds are Ultimate wind speeds.

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-22 Section 26.2.

Seismic

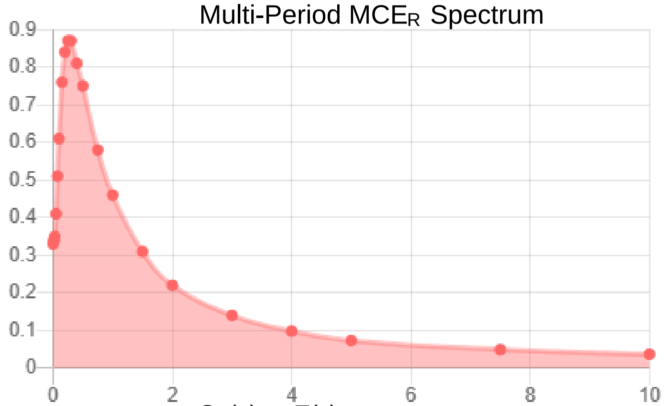
Site Soil Class: D - Stiff Soil

Results:

PGA_M :	0.31	T_L :	6
S_{MS} :	0.79	S_S :	0.58
S_{M1} :	0.46	S_1 :	0.17
S_{DS} :	0.52	V_{S30} :	260
S_{D1} :	0.3		

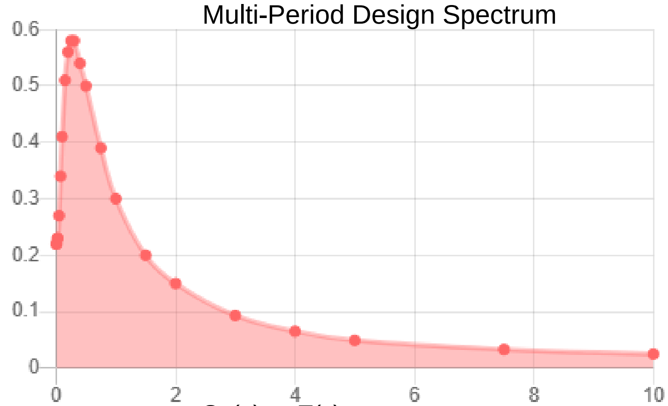
Seismic Design Category: D

Multi-Period MCE_R Spectrum



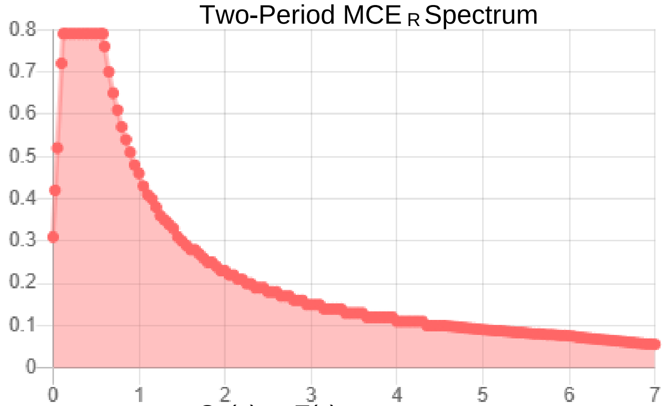
S_a(g) vs T(s)

Multi-Period Design Spectrum



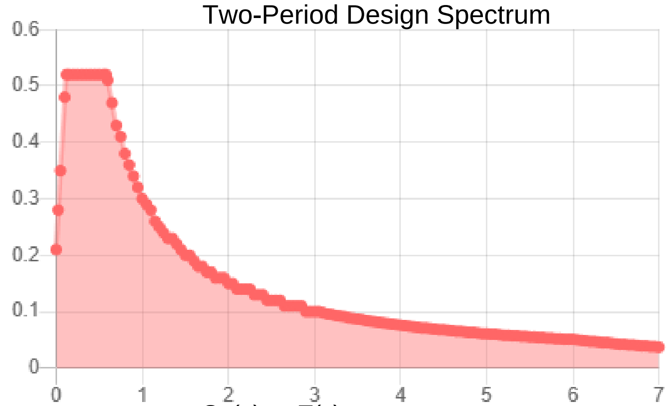
S_a(g) vs T(s)

Two-Period MCE_R Spectrum



S_a(g) vs T(s)

Two-Period Design Spectrum



S_a(g) vs T(s)

MCE_R Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Wed Jun 18 2025**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

Snow

Results:

Ground Snow Load, p_g :	21 lb/ft^2
20-year MRI Value:	3.88 lb/ft^2
Winter Wind Parameter:	0.35
Mapped Elevation:	3109.2 ft
Data Source:	ASCE/SEI 7-22, Figures 7.6-1 and 7.6-2 A-D
Date Accessed:	Wed Jun 18 2025

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

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

Ground Snow Loads for IRC only, $p_{g(asd)}$:	14.7 lb/ft^2

Code Search

Code: International Building Code 2024

Occupancy:

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category =	III
Wind Factor =	1.00
Snow Factor =	1.00
Seismic Importance factor =	1.25

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.25 / 12	1.2 deg
Building length	121.2 ft	
Least width	78.0 ft	
Mean Roof Ht (h)	71.0 ft	
Parapet ht above grd	74.5 ft	
Minimum parapet ht	1.5 ft	
hb for Elevated bldg	0.0 ft	

Live Loads:

Roof

0 to 200 sf:	20 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12 psf

0 psf

Floor:

Typical Floor	50 psf
Partitions	15 psf
Corridors above first floor	80 psf
Lobbies & first floor corridors	100 psf
Stairs and exit ways	100 psf
	0 psf
	0 psf

Roof Design Loads

Items	Description	Multiple	psf (max)	psf (min)
			0.0	0.0
Decking	Metal Roof deck, 1.5, 22 ga.		1.7	1.2
Framing	Steel roof joists & girders		3.0	2.0
Insulation	Rigid insulation, per 1"	x 5.0	7.5	3.5
Ceiling	1/2" gypsum board		2.2	2.0
Mech & Elec	Mech. & Elec.	x 3.0	6.0	0.0
Misc.	Misc.	x 4.0	2.0	0.0
			0.0	0.0
		Actual Dead Load	22.4	8.7
		Use this DL instead	25.0	9.0
		Live Load	20.0	0.0
		Snow Load	21.0	0.0
		Ultimate Wind (zone 2 - 100 sf)	16.0	-54.7
ASD Loading		D + Lr	45.0	-
		D + 0.75(0.6W + Lr)	47.2	-
		0.6*D + 0.6*W	-	-27.4
LRFD Loading		1.2D + 1.0S + 0.5W	70.0	-
		1.2D + 1.0W + 0.5Lr	56.0	-
		0.9D + 1.0W	-	-46.6

Roof Live Load Reduction Roof angle 0.25 / 12 1.2 deg

0 to 200 sf:	20.0 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12.0 psf

	300 sf	18.0 psf
	400 sf	16.0 psf
	500 sf	14.0 psf
User Input:	450 sf	15.0 psf

Floor Design Loads

Items	Description	Multiple	psf (max)	psf (min)
Flooring	Carpet & pad		1.0	1.0
Topping	Concrete regular per 1"	x 3.5	43.8	42.0
Decking	Metal Floor deck - 2", 20ga		2.0	1.5
Framing	Steel floor bms/joists & girders		8.0	5.0
			0.0	0.0
Ceiling	Suspended acoustical tile		1.8	1.0
Sprinklers	Sprinklers		2.0	0.0
Mech & Elec	Mech. & Elec.		2.0	0.0
Misc.	Misc.		0.5	0.0
		Actual Dead Load	61.1	50.5
		Use this DL instead	80.0	65.0
		Partitions	15.0	0.0
		Live Load	50.0	0.0
		Total Live Load	65.0	0.0
		Total Load	145.0	50.5

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JOB TITLE Creech 2 - Admin Criteria

JOB NO. **SHEET NO.**
CALCULATED BY ACV **DATE** 10/7/25
CHECKED BY **DATE**

Wall Design Load #1

Items	Description	Multiple	psf (max)	psf (min)
			0.0	0.0
Sheathing	5/8" gypsum		2.8	2.5
Framing	CMU wall		83.0	81.0
veneer	4" Clay Brick		40.0	38.0
			0.0	0.0
Insulation	Rigid insulation, per 1"	x 1.00	1.5	0.7
Mech & Elec	Mech. & Elec.		1.0	0.0
Misc.	Misc.		0.5	0.0
		Actual Dead Load	128.8	122.2
		Use this DL instead	130.0	40.0

CMU walls

Size = 8" CMU
Unit Density = Normal weight (135 pcf)
Grout Spacing = Full Grout
CMU wall weight = 83 psf

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Wind Loads : ASCE 7- 22

Ultimate Wind Speed	105 mph
Nominal Wind Speed	81.3 mph
Risk Category	III
Exposure Category	C
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Bldg Directionality (Kd)	0.85
Kh MWFRS<=60	1.012
Kh all other	1.012
Type of roof	Gable

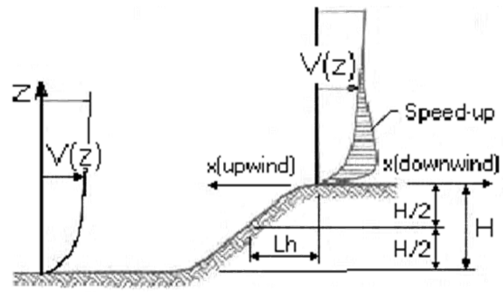
Topographic Factor (Kzt)

Topography	Flat
Hill Height (H)	80.0 ft
Half Hill Length (Lh)	100.0 ft
Actual H/Lh =	0.80
Use H/Lh =	0.50
Modified Lh =	160.0 ft
From top of crest: x =	50.0 ft
Bldg up/down wind?	downwind

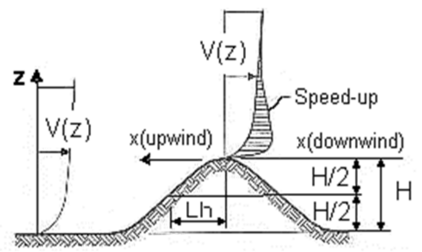
H/Lh= 0.50	K_1 = 0.000
x/Lh = 0.31	K_2 = 0.792
z/Lh = 0.22	K_3 = 1.000

At Mean Roof Ht:

$Kzt = (1+K_1K_2K_3)^2 = 1.00$



ESCARPMENT



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	35.0 ft
B =	78.0 ft
/z (0.6h) =	21.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 0.45 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

ē =	0.20
ℓ =	500 ft
z_{min} =	15 ft
c =	0.20
g_Q, g_v =	3.4
L_z =	456.8 ft
Q =	0.89
I_z =	0.22
G =	**0.87** use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.7 Hz				
Damping ratio (β) =	0.01				
/b =	0.660				
/α =	0.156				
Vz =	94.7				
N_1 =	3.38				
R_n =	0.065				
R_h =	0.520	η =	1.190	h =	35.0 ft
R_B =	0.306	η =	2.652		
R_L =	0.070	η =	13.795		
g_R =	4.104				
R =	0.763				
Gf =	1.069				

Ground Elevation Factor (Ke)

Grd level above sea level =	0 ft	Ke =	1.0000
Constant =	0.00256		
0.00256Ke =	0.00256		

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Wind Loads - MWFRS all h (Except for Open Buildings)

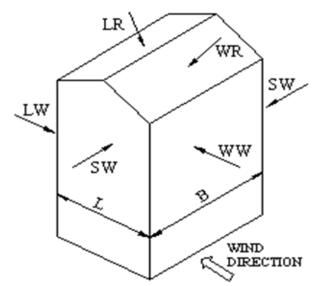
Base pressure (qh) =	28.6 psf	Kh =	1.012	GCpi =	+/-0.18
(Kd qh) =	**24.3 psf**	Bldg dim parallel to ridge =	121.2 ft	G =	0.85
Roof Angle (θ) =	1.2 deg	Bldg dim normal to ridge =	78.0 ft	qi = qh	
Roof tributary area:		h =	35.0 ft		
Wind normal to ridge =(h/2)*L:	2121 sf	ridge ht =	35.8 ft		
Wind parallel to ridge =(h/2)*L:	1365 sf				

Ultimate Wind Surface Pressures (psf)

	Wind Normal to Ridge				**Wind Parallel to Ridge**				
	L/B = 0.64		h/L = 0.45		L/B = 1.55			h/L = 0.29	
Surface	Cp	q_hGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Dist.*	Cp	q_hGC_p	w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Windward Wall (WW)	0.80	16.5	see table below			0.80	16.5	see table below	
Leeward Wall (LW)	-0.50	-10.3	-14.7	-5.9		-0.39	-8.0	-12.4	-3.7
Side Wall (SW)	-0.70	-14.4	-18.8	-10.1		-0.70	-14.4	-18.8	-10.1
Leeward Roof (LR)		**			Included in windward roof				
Neg Windward Roof: 0 to h/2*	-0.90	-18.6	-22.9	-14.2	0 to h/2*	-0.90	-18.6	-22.9	-14.2
h/2 to h*	-0.90	-18.6	-22.9	-14.2	h/2 to h*	-0.90	-18.6	-22.9	-14.2
h to 2h*	-0.50	-10.3	-14.7	-5.9	h to 2h*	-0.50	-10.3	-14.7	-5.9
> 2h*	-0.30	-6.2	-10.6	-1.8	> 2h*	-0.30	-6.2	-10.6	-1.8
Pos/min windward roof press.	-0.18	-3.7	-8.1	0.7	Min press.	-0.18	-3.7	-8.1	0.7

*Horizontal distance from windward edge
**Roof angle < 10 degrees. Therefore, leeward roof is included in windward roof pressure zones.

Windward roof overhangs : 16.5 psf (upward : add to qhGCp windward roof pressure)



WIND NORMAL TO RIDGE

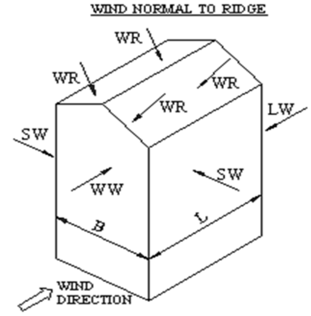
Parapet

z	Kz	Kzt	Kdqp (psf)
39.3 ft	1.036	1.00	24.9

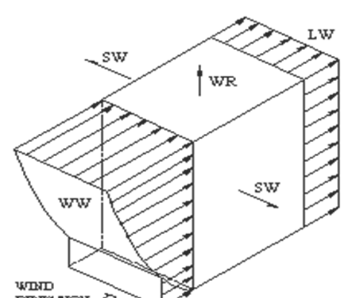
Windward parapet: 37.3 psf (GCpn = +1.5)
Leeward parapet: -24.9 psf (GCpn = -1.0)

Windward Wall Pressures at "z" (psf)

				Windward Wall		Combined WW + LW	
z	Kz	Kzt	q_zGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Wind Normal to Ridge	Wind Parallel to Ridge
0 to 15'	0.85	1.00	13.9	9.5	18.3	24.2	21.9
20.0 ft	0.90	1.00	14.7	10.4	19.1	25.0	22.8
25.0 ft	0.94	1.00	15.4	11.0	19.8	25.7	23.4
30.0 ft	0.98	1.00	16.0	11.6	20.4	26.3	24.0
h= 35.0 ft	1.01	1.00	16.5	12.1	20.9	26.8	24.5
ridge = 35.8 ft	1.02	1.00	16.6	12.2	21.0	26.9	24.6



WIND PARALLEL TO RIDGE



TYPICAL WIND LOADING

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Elevated Buildings

Horizontal MWFRS wind pressures on objects below hb

h = 35.0 ft
hb = 0.0 ft
z = 8.8 ft

Elevated Building Geometry limitation 1

Bldg Length = 121.2 ft
Bldg Width = 78.0 ft
Area of elevated building above = 9,454 sf

Area of below elements / Area of Bldg above = 1.2%

Cross setional area of all columns below bldg = 64.0 sf
Area of enclosed areas below bldg = 50.0 sf
Total cross sectional area below bldg = 114.0 sf

Direction 1 L/B = 0.64 Max L/B = 0.500 OK
Direction 2 L/B = 1.55 Max L/B = 0.500 OK
Meets geometry Limitation No 1 for both directions

Elevated Building Geometry limitation 2

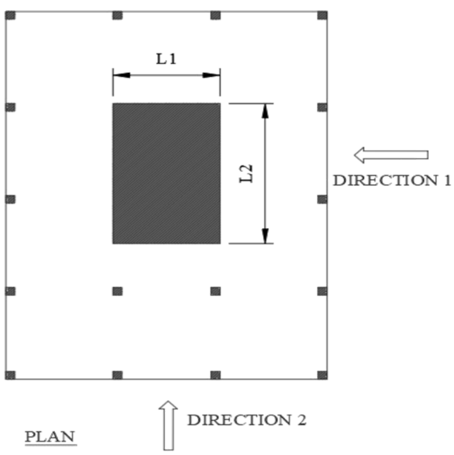
Direction 1
Projected width of all columns facing direction 1 = 32.0 ft
Projected L2 width of enclosed areas below bldg = 40.0 ft
Total projected width below bldg (width) = 72.0 ft

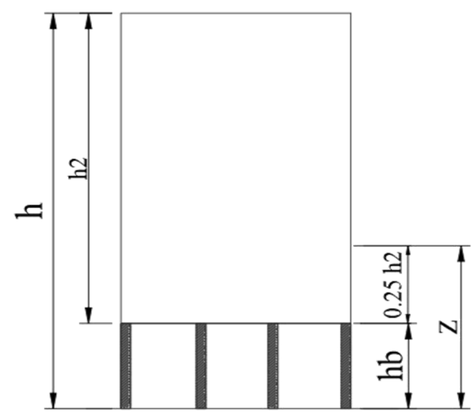
Projected area ratio = 59.4% OK

Direction 2
Projected width of columns direction 2 = 30.0 ft
Projected L1 width of enclosed areas = 42.0 ft
Total projected width below bldg (width) = 72.0 ft

Projected area ratio = 92.3% > 75% NG
Meets geometry Limitation No 2 for direction 1 only

hb = 0, therefore building is not an elevated building





ELEVATION

Combined MWFRS windward and leeward wind pressure on surfaces from 0 to hb (Kd qzGCp) = **0.0** psf *
MWFRS direction 1 force at height hb (width*hb/2) = 0.0 k
MWFRS direction 2 force at height hb (width*hb/2) = 0.0 k *
* Direction 1 meets geometry limitations - design as elevated structure, but design direction 2 as continous to ground

Vertical MWFRS wind pressures on bottom surface of the elevated building

Base pressure (Kd qz) = 0.0 psf

Ultimate Vertical MWFRS Wind Surface Pressures (psf) at horizontal bottom surface of elevated building

	Wind Normal to Ridge				**Wind Parallel to Ridge**				
	L/B = 0.64		hb/L = 0.00			L/B = 1.55		hb/L = 0.00	
	Cp	q_hGC_p	w/+q_iGC_{pi}	w/-q_hGC_{pi}	Dist.*	Cp	q_hGC_p	w/ +q_iGC_{pi}	w/ -q_hGC_{pi}
Downward pressure: 0 to hb/2*	-0.90	0.0	0.0	0.0	0 to hb/2*	-0.90	0.0	0.0	0.0
hb/2 to hb*	-0.90	0.0	0.0	0.0	hb/2 to hb*	-0.90	0.0	0.0	0.0
hb to 2hb*	-0.50	0.0	0.0	0.0	hb to 2hb*	-0.50	0.0	0.0	0.0
> 2hb*	-0.30	0.0	0.0	0.0	> 2hb*	-0.30	0.0	0.0	0.0
Upward or min wind pressure	-0.18	0.0	0.0	0.0	Min press.	-0.18	0.0	0.0	0.0

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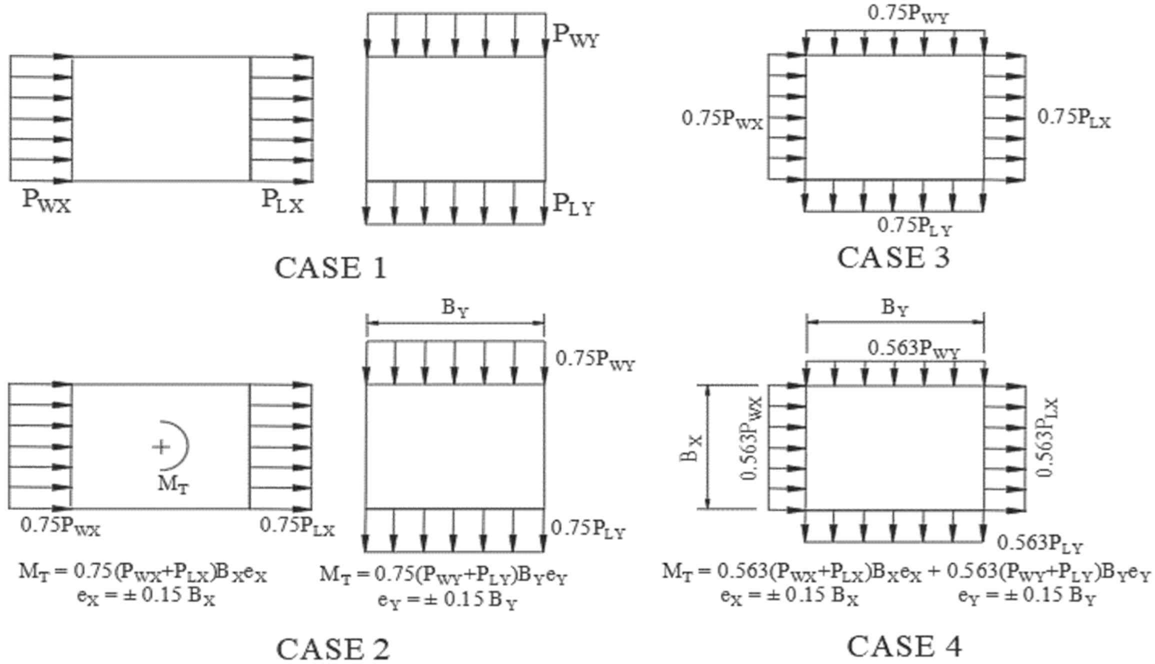
NOTE: ASCE 7 requires the application of full and partial loading of the wind pressures per the 4 cases below.



Wind Forces at Floors

Building dimension (parallel with ridge) = 121.2 ft   e = 18.18 ft

Total Floors above grade = 2   Building dimension (normal to ridge) = 78.0 ft   e = 11.70 ft

T/Fdn (dist below grade) = 2.0 ft   L is the building dimension parallel to the wind direction

Level	Elevation Above Grade (ft)	Height of Centroid to Fdn (ft)	Wind Normal to Ridge: L	B	Area (sf)	Applied Force (k)	Story Shear (k)	Overturning Moment ('k)	Wind Parallel to Ridge: Area	Applied Force (k)	Story Shear (k)	Overturning Moment ('k)	
Equip,etc	66.00	68.00	wind on equip, screenwalls, etc =			2				0			
Parapet	39.30	39.15	78.0	121.2	521.2	32.4			335.4	20.8			
T/Ridge	0.00	0.00			0.0	0.0			0.0	0.0			
Roof	35.00	37.00	78.0	121.2	0.0	0.0	34.4	131.6	0.0	0.0	20.8	44.8	Roof
2	35.00	37.00	78.0	121.2	909.0	24.4	58.8	131.6	585.0	14.4	35.2	44.8	2
1	20.00	22.00	78.0	121.2	2,121.0	53.1	111.9	1,013.1	1,365.0	31.1	66.3	572.7	1
GRD		2.00						3,250.7				1,897.9	GRD
FDN		0.00						3,474.4				2,030.4	FDN

Wind Base Shear = 66.3kips < Seismic Base Shear = 159.9kips (Parallel to Ridge)
Wind Base Shear = 111.9kips < Seismic Base Shear = 179.5 kips (Normal to Ridge)

Seismic Governs.

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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Base pressure (qh) =	28.6 psf	Kh =	1.012	
(Kd qh) =	**24.3 psf**	h =	35.0 ft	0.2h = 7.0 ft
Minimum parapet ht =	1.5 ft	0.6h =	21.0 ft	
Roof Angle (θ) =	1.2 deg	GCpi =	+/-0.18	
Type of roof =	Gable	Kd qi = Kd qh =	24.3 psf	

Roof

	GCp +/- Gcpi				Surface Pressure (psf)				User input	
Area	10 sf	100 sf	500 sf	1000 sf	10 sf	100 sf	500 sf	1000 sf	100 sf	100 sf
Negative Zone 1	-1.88	-1.47	-1.18	-1.18	-45.6	-35.6	-28.6	-28.6	-35.6	-35.6
Negative Zone 1'	-1.08	-1.08	-0.73	-0.58	-26.2	-26.2	-17.7	-16.0	-26.2	-26.2
Negative Zone 2	-2.48	-1.95	-1.58	-1.58	-60.2	-47.3	-38.4	-38.4	-47.3	-47.3
Negative Zone 3	-3.38	-2.32	-1.58	-1.58	-82.0	-56.3	-38.4	-38.4	-56.3	-56.3
Positive All Zones	0.48	0.38	0.38	0.38	16.0	16.0	16.0	16.0	16.0	16.0
Overhang Zone 1&1'	-1.70	-1.60	-1.00	-1.00	-41.3	-38.8	-24.3	-24.3	-38.8	-38.8
Overhang Zone 2	-2.30	-1.59	-1.10	-1.10	-55.8	-38.7	-26.7	-26.7	-38.7	-38.7
Overhang Zone 3	-3.20	-1.96	-1.10	-1.10	-77.7	-47.7	-26.7	-26.7	-47.7	-47.7

Overhang pressures in the table above assume an internal pressure coefficient (Gcpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 4.4 psf)

Parapet

Kd qp = 24.9 psf

		Surface Pressure (psf)						User input
Solid Parapet Pressure		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	50 sf
CASE A:	Zone 2 :	79.5	74.4	67.6	62.4	57.3	50.5	67.6
	Zone 3 :	101.9	92.8	80.7	71.6	62.5	50.5	80.7
CASE B:	Interior zone :	-47.0	-44.6	-41.5	-39.1	-36.7	-33.6	-41.5
	Corner zone :	-53.7	-50.1	-45.4	-41.8	-38.3	-33.6	-45.4

wall a = 7.8 ft

Walls

	GCp +/- GCpi				Surface Pressure at h				User input	
Area	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	100 sf	200 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-28.4	-24.5	-23.4	-21.8	-24.5	-23.4
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-35.0	-27.2	-24.9	-21.8	-27.2	-24.9
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	26.2	22.4	21.2	19.7	22.4	21.2

Note: GCp reduced by 10% due to roof angle <= 10 deg.

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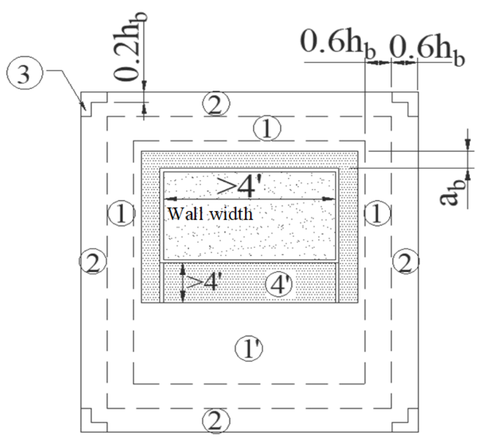
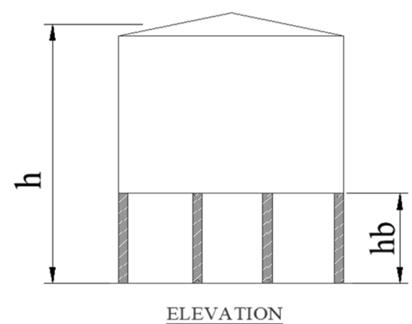
Bottom Horizontal Surface of Elevated Buildings

Base pressure (qh) =	1.01	h =	35.0 ft	0.2hb =	0.00
(Kd qh) =	**24.3 psf**	hb =	0.0 ft	0.6hb =	0.00
Wall width =	5.0 ft			ab =	0.00

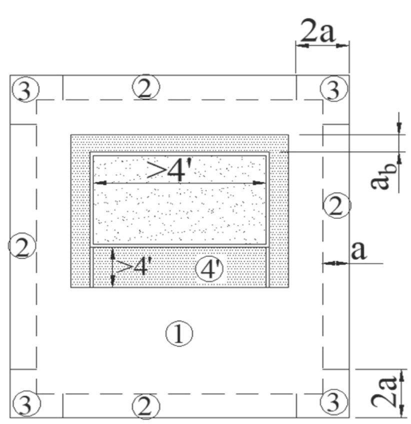
	GCp				Surface Pressure (psf)				**User input**	
Area	10 sf	100 sf	500 sf	1000 sf	10 sf	100 sf	500 sf	1000 sf	100 sf	100 sf
Negative Zone 1	-1.70	-1.29	-1.00	-1.00	-45.6	-35.6	-28.6	-28.6	-35.6	-35.6
Negative Zone 1'	-0.90	-0.90	-0.55	-0.40	-26.2	-26.2	-17.7	-16.0	-26.2	-26.2
Negative Zone 2	-2.30	-1.77	-1.40	-1.40	-60.2	-47.3	-38.4	-38.4	-47.3	-47.3
Negative Zone 3	-3.20	-2.14	-1.40	-1.40	-82.0	-56.3	-38.4	-38.4	-56.3	-56.3
Positive Zones 1-3	0.30	0.20	0.20	0.20	16.0	16.0	16.0	16.0	16.0	16.0
Negative Zone 4'	-0.99	-0.83	-0.72	-0.72	-28.4	-24.5	-21.8	-21.8	-24.5	-24.5
Positive Zone 4'	0.90	0.74	0.63	0.63	26.2	22.4	19.7	19.7	22.4	22.4

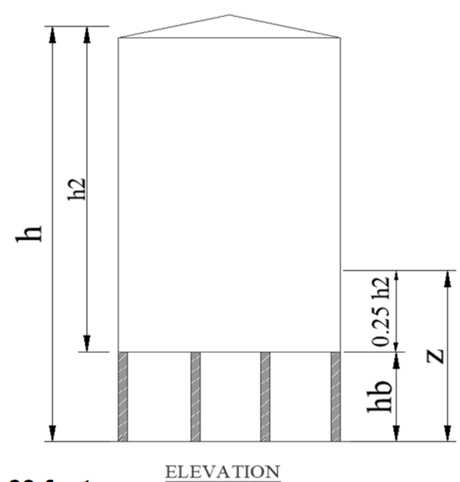
Negative pressures are downward





Building Bottom Plan: h ≤ 60' and alternate design 60'<h<90'





Building Bottom Plan: h> 60 feet

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JOB TITLE	Creech 2 - Admin Criteria		
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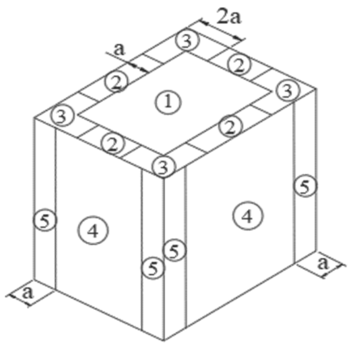
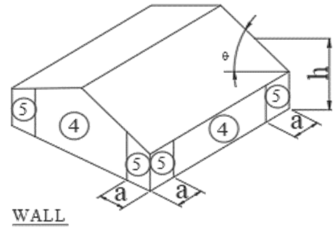
CHECKED BY		**DATE**	

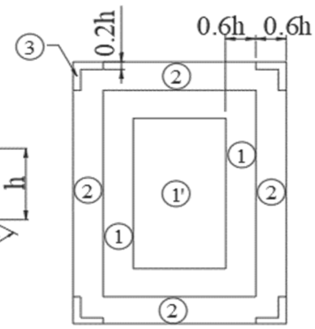
Location of C&C Wind Pressure Zones - ASCE 7-22



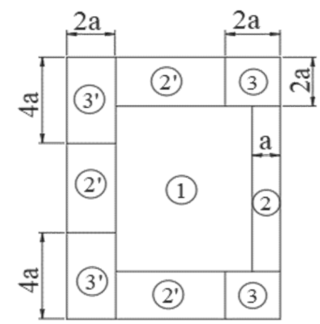
Roofs w/ θ ≤ 10°
and all walls
h > 60'



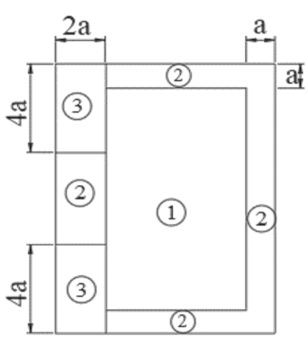
Walls h ≤ 60'
& alt design h<90'



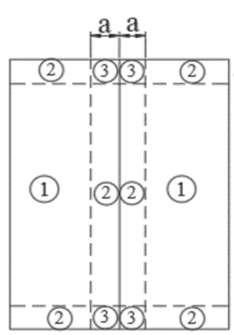
Multispan Gable & Sawtooth ≤ 10°
& Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



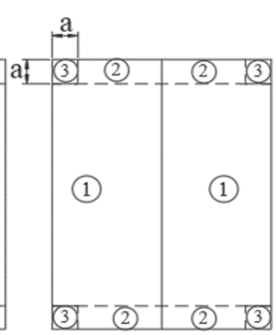
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



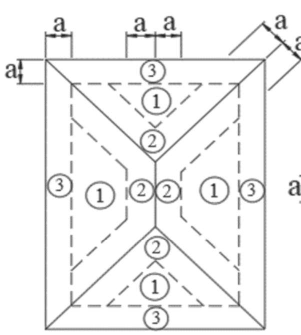
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



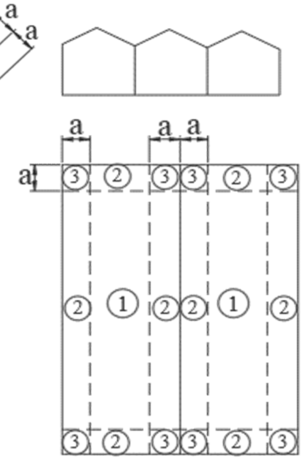
Gable 7° < θ ≤ 27°



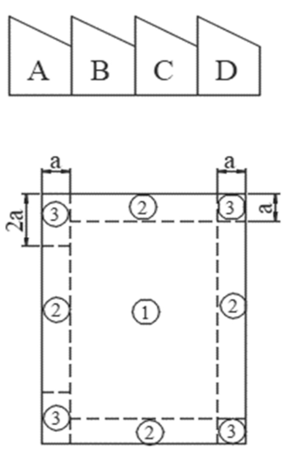
Gable 27° < θ ≤ 45°



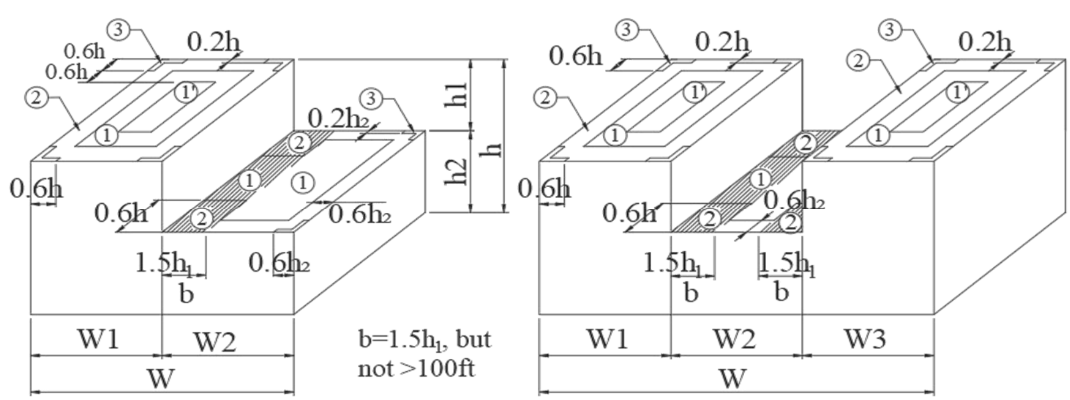
Hip 7° < θ ≤ 45°



Multispan gable 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



Sawtooth 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



Stepped roofs θ ≤ 3°
h ≤ 60' & alt design h<90'

Note: The hatched area indicates where roof positive pressures are equal to the adjacent wall positive pressure.

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Snow Loads : ASCE 7- 22

Ultimate Snow Forces

Roof slope	=	1.2 deg
Horiz. eave to ridge dist (W)	=	39.0 ft
Roof length parallel to ridge (L)	=	121.2 ft

Type of Roof			Hip and gable w/ trussed systems
Ground Snow Load	Pg	=	21.0 psf
Risk Category		=	III
Snow Factor		=	1.0
Roof R value	Rroof	=	30
Thermal Factor	Ct	=	1.188
Exposure Factor	Ce	=	1.00
Pf = 0.7*Ce*Ct*I*Pg		=	17.5 psf
Unobstructed Slippery Surface			no
Sloped-roof Factor	Cs	=	1.00
Balanced Snow Load		**=**	**17.5 psf**
Rain on Snow Surcharge Angle			0.78 deg
Code Maximum Rain Surcharge			8.0 psf
Rain on Snow Surcharge		=	0.0 psf
Ps plus rain surcharge		=	17.5 psf
Minimum Snow Load	Pm	=	21.0 psf
Uniform Roof Design Snow Load		=	**21.0 psf**

Near ground level surface balanced snow load = **21.0 psf**

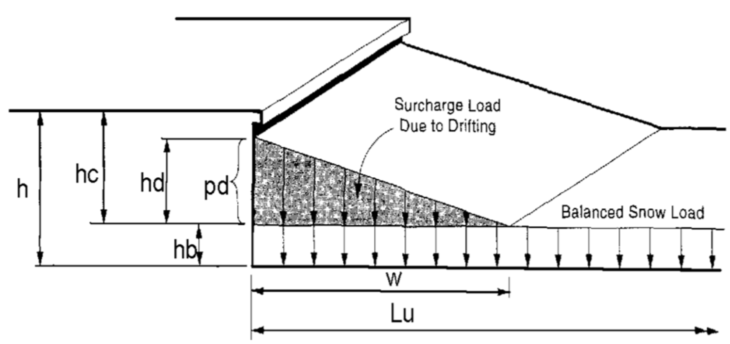
NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs

Unbalanced Snow Loads - for Hip & Gable roofs only

Winter Wind Parameter W2 = 0.35

Required if slope is between 7 on 12 =	30.26 deg	
and 2.38 deg =	2.38 deg	**Unbalanced snow loads are not required**
Windward snow load =	21.0 psf	
Leeward snow load =	21.0 psf	= hdγ / √S + Ps

ft



Note: If bottom of projection is at least 2 feet above hb then snow drift is not required.

Snow Loads - from adjacent building or roof:

ASCE 7- 22 — Ultimate Snow Forces

		Higher Roof	Lower Roof
Roof slope	=	1.2 deg	0.50 / 12 = 2.4 deg
Horiz. eave to ridge dist (W)	=	39.0 ft	78.0 ft
Roof length parallel to ridge (L)	=	121.2 ft	121.0 ft
Projection height (roof step) h	=		5.0 ft
Building separation s	=		0.0 ft

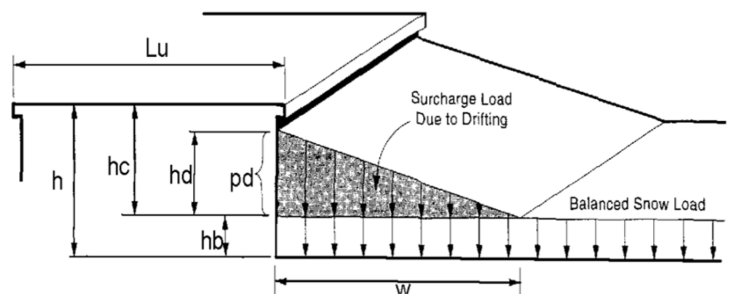
Winter Wind Parameter W2 = 0.35

			Higher Roof	Lower Roof
Type of Roof			Hip and gable w/ trussed systems	Hip and gable w/ trussed systems
Ground Snow Load	Pg	=	21.0 psf	21.0 psf
Risk Category		=	III	III
Snow Factor		=	1	1
Roof R value	Rroof	=	30	30
Thermal Factor	Ct	=	1.188	1.188
Exposure Factor	Ce	=	1.0	1.0
Pf = 0.7*Ce*Ct*I*Pg		=	17.5 psf	17.5 psf
Unobstructed Slippery Surface			yes	yes
Sloped-roof Factor	Cs	=	1.00	1.00
Balanced Snow Load	**Ps**	**=**	**17.5 psf**	**17.5 psf**
Rain on Snow Surcharge Angle			0.78 deg	1.56 deg
Code Maximum Rain Surcharge			8.0 psf	8.0 psf
Rain on Snow Surcharge		=	0.0 psf	0.0 psf
Ps plus rain surcharge		=	17.5 psf	17.5 psf
Minimum Snow Load	Pm	=	21.0 psf	21.0 psf
Uniform Roof Design Snow Load		=	**21.0 psf**	**21.0 psf**
Building Official Minimum		=		

NOTE: Alternate spans of continuous beams and other areas shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code.

Leeward Snow Drifts - from adjacent higher roof

Upper roof length	lu	=	123.0 ft
Snow density	ɣ	=	16.7 pcf
Balanced snow height	hb	=	1.04 ft
	hc	=	3.96 ft
hc/hb >0.2 = 3.8			**Therefore, design for drift**
Adj structure factor		=	1.00
Drift height	hd	=	2.50 ft
Drift width	w	=	9.99 ft
Surcharge load:	pd = γ*hd	=	**41.8 psf**
Balanced Snow load:		=	17.5 psf
			59.2 psf **Leeward drift controls**

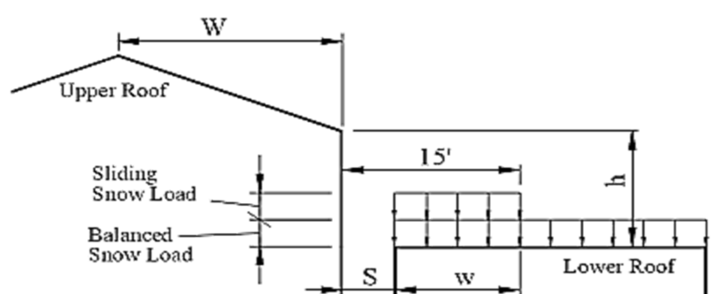


Windward Snow Drifts - from low roof against high roof

Lower roof length	lu	=	78.0 ft
Adj structure factor		=	1.00
Drift height	hd	=	1.60 ft
Drift width	w	=	12.78 ft
Surcharge load:	pd = γ*hd	=	**26.7 psf**
Balanced Snow load:		=	17.5 psf
			44.2 psf

Sliding Snow - onto lower roof

Sliding snow = 0.4 Pf W	=	0.0 plf
Distributed over 15 feet	=	0.0 psf
hd + hb	=	1.04 ft
hd + hb < =h therefore sliding snow	=	**0.0 psf**
Balanced snow load	=	17.5 psf
Uniform snow load	=	17.5 psf



Sliding snow not required since upper roof slope is 1/4 in 12 or less

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Seismic Loads: IBC 2024 — Strength Level Forces

Risk Category : III
Importance Factor (Ie) : 1.25

Site Class : D

Ss (0.2 sec) = 0.72 g
S1 (1.0 sec) = 0.23 g

Site specific ground motion analysis performed:

Sms =	0.790	S_{DS} =	0.527	Design Category =	D
Sm1 =	0.460	S_{D1} =	0.307	Design Category =	D

Seismic Design Category = **D**
Redundancy Coefficient ρ = 1.30
Number of Stories: 2

Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: No
Building System: **Building Frame Systems**
Seismic resisting system: **Special reinforced masonry shear walls**
System Structural Height Limit: **160 ft**
Actual Structural Height (hn) = 71.0 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) =	5.5	To = 0.2(Sd1/Sds) =	0.116
Over-Strength Factor (Ωo) =	2.5	Ts = Sd1/Sds =	0.582
Deflection Amplification Factor (Cd) =	4	Long Period Transition Period (TL) =	6 sec

S_{DS} = 0.527
S_{D1} = 0.307

Seismic Load Effect (E) = Eh +/-Ev = ρ Q_E +/- $0.2S_{DS}$ D = 1.3Qe +/- 0.105D — Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = Ωo Q_E +/- $0.2S_{DS}$ D = 2.5Qe +/- 0.105D — D = dead load

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift Δa = 0.015hsx where hsx is the story height below level x

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis - Method Not Permitted (only applies to Seismic Category A)

Model & Seismic Response Analysis - Permitted (see code for procedure)

Equivalent Lateral-Force (ELF) Analysis - Permitted

Building period coef. (C_T) = 0.020 — Cu = 1.40
Approx fundamental period (Ta) = $C_T h_n^x$ = 0.489 sec x= 0.75 — Tmax = CuTa = 0.685 sec
User calculated fundamental period = — T = 0.489 sec

Method 2:

Seismic response coef. (Cs) =	SdsI/R =	0.120
need not exceed Cs =	Sd1 I /RT =	0.142
but not less than Cs =	0.044Sds*I =	0.029
USE Cs =		0.120

Design Base Shear V = 0.120W

Method 1:

Enter Sa =		
Seismic response coef. (Cs) =	SaI/R =	0.000
but not less than Cs =	0.044Sds*I =	0.029
Cs =		Method not applicable

SEISMIC FORCES AT FLOORS - ELF Procedure

Total Stories =	2	Floor Dead Load =	80.0 psf	Roof Snow Load =	0.0 psf
Building length L =	121.2 ft	Floor LL to include =	0.0 psf	Roof Equip wt =	0.0 kips
Building width W =	78.0 ft	Floor Equip wt =	0.0 kips	Parapet weight =	0.0 psf
hn =	71.0 ft	Partition weight =	10.0 psf	Parapet height =	0.0 ft
k =	1.000	Ext Wall Weight =	72.0 psf		
V =	0.120W	Roof Dead Load =	20.0 psf		

Bottom Floor (level 1) is a slab on grade

Diaphragm shall be designed for level force Fx,
but not less than Fpx = (Σ Fi / Σ wi) wpx, but :
Fpx min = $0.2S_{DS}$ Ie wpx = 0.132 wpx
Fpx max = $0.4S_{DS}$ Ie wpx = 0.263 wpx

Seismic Forces (Including all exterior walls)

Level (x)	EL above Seismic Base hx (ft)	Level Weight Wx (kips)	$W_x h_x^k$ (ft-kips)	Cvx = $\frac{W_x h_x^k}{\Sigma W_i h_i^k}$	V = 214.9k Base Shear Distribution Fx=CvxV	Σ Fx (k)	Story M	Diaphragm Force Fpx Σ Wi (k)	Fpx	Design Fpx
Roof	35.00	442	15,486	0.398	85.47	85.5	0	442	85.5	85.5
2	17.33	1,353	23,448	0.602	129.42	214.9	1,510	1,795	161.9	178.1
1	0.00	0	0	0.000	0.00	0.0	0	0	0.0	0.0
Base		1,795		1.000		214.9	3,725			
							5,235 = Base M			

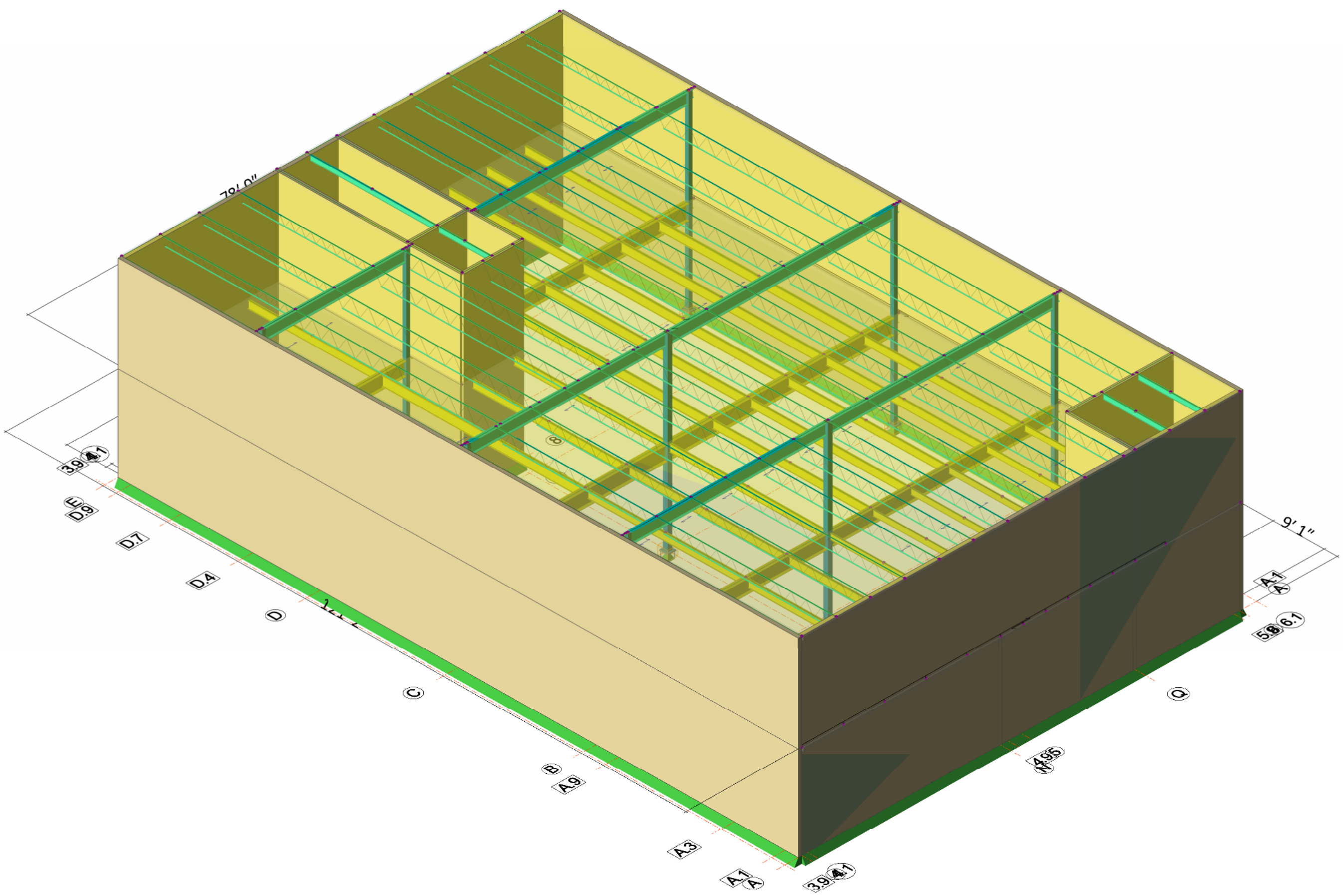
Diaphragm Forces excluding parallel exterior walls

Diaphragm Force Fpx Parallel to Bldg Length V= 160k							Diaphragm Force Fpx Normal to Bldg Length V= 179k					
Cvx =	Fx=CvxV	Σ Fx (k)	Σ Wi (k)	Fpx	Design Fpx	Level (x)	Cvx =	Fx=CvxV	Σ Fx (k)	Σ Wi (k)	Fpx	Design Fpx
0.357	57.11	57.1	288	57.1	57.1	Roof	0.375	67.3	67.3	343	67.3	67.3
0.643	102.76	159.9	1,336	125.4	137.9	2	0.625	112.2	179.5	1,499	138.4	152.2
0.000	0.00	0.0	0	0.0	0.0	1	0.000	0.0	0.0	0	0.0	0.0
1.000		159.9				Base	1.000		179.5			

N/S Seismic Direction

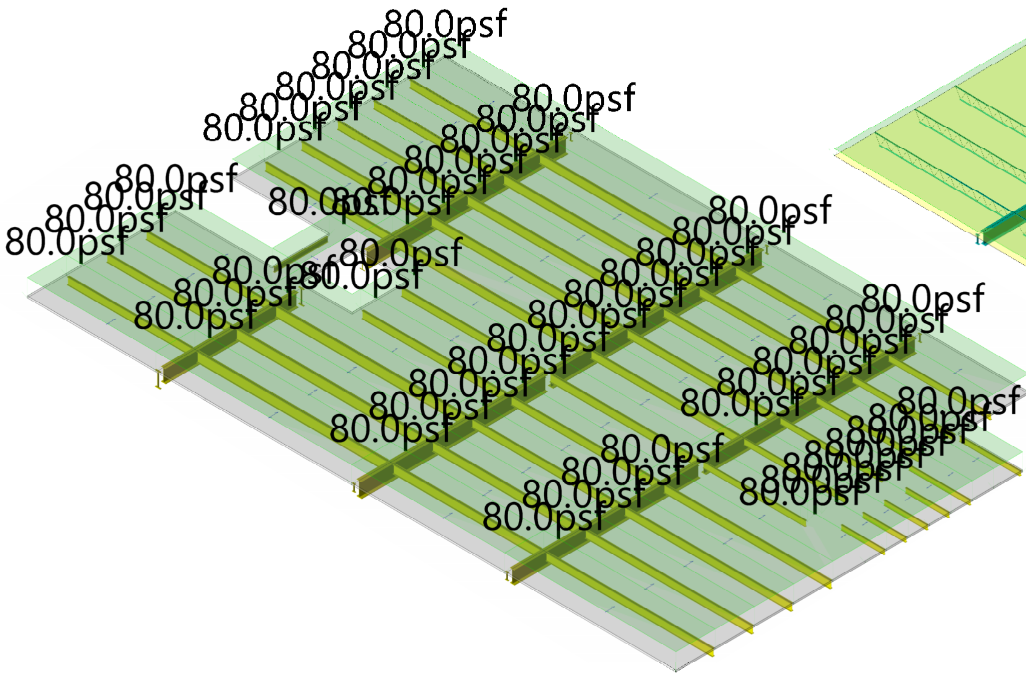
E/W Seismic Direction

Tekla Structural Designer	Project: Creech DRP PH2				Job Ref.	
	Structure: Admin Building				Sheet no.: Page 1/1	
	Calc. by: Allison Valencia	Date: 10/16/2025	Chk'd by	Date	App'd by	Date

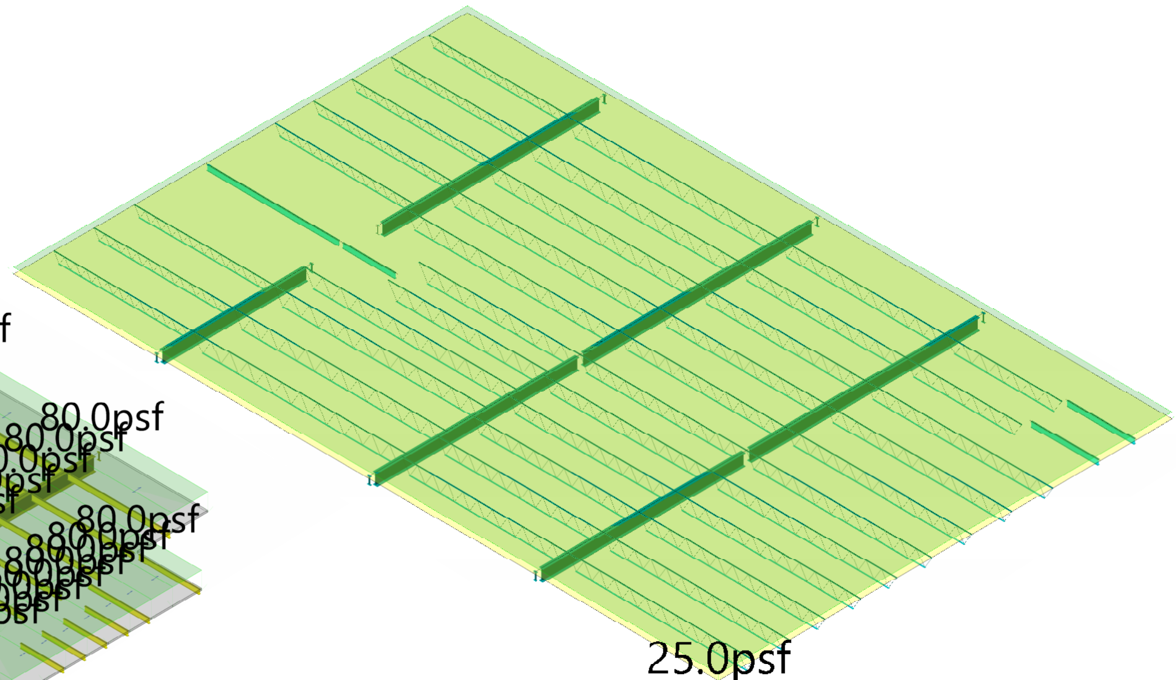


Structure

Tekla Structural Designer, version: 25.0.1.45

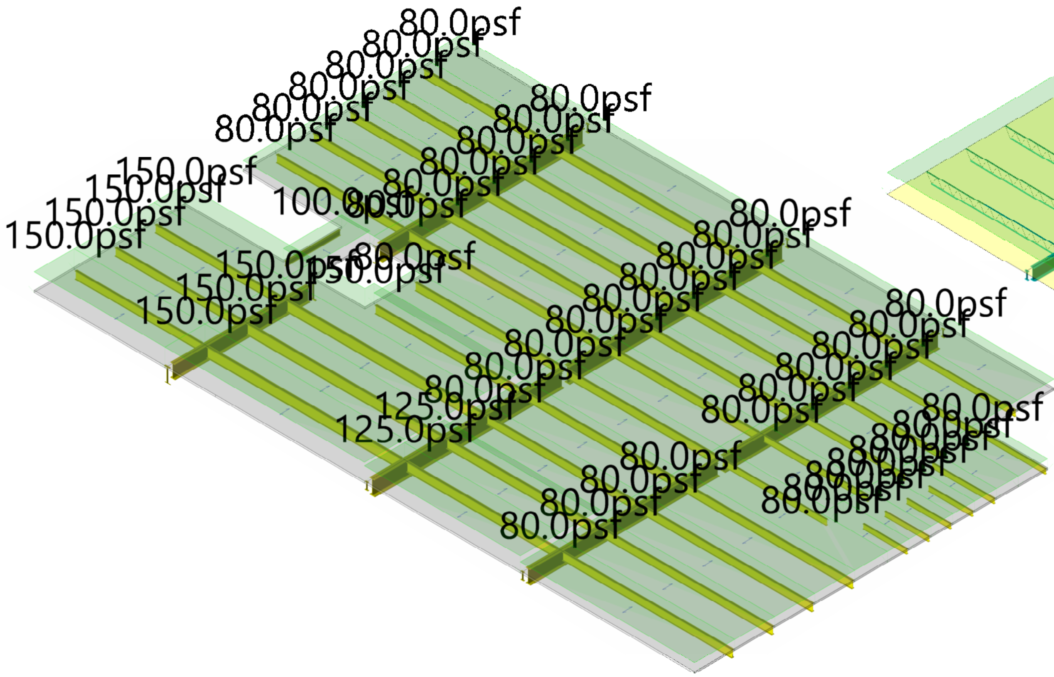


2nd Floor

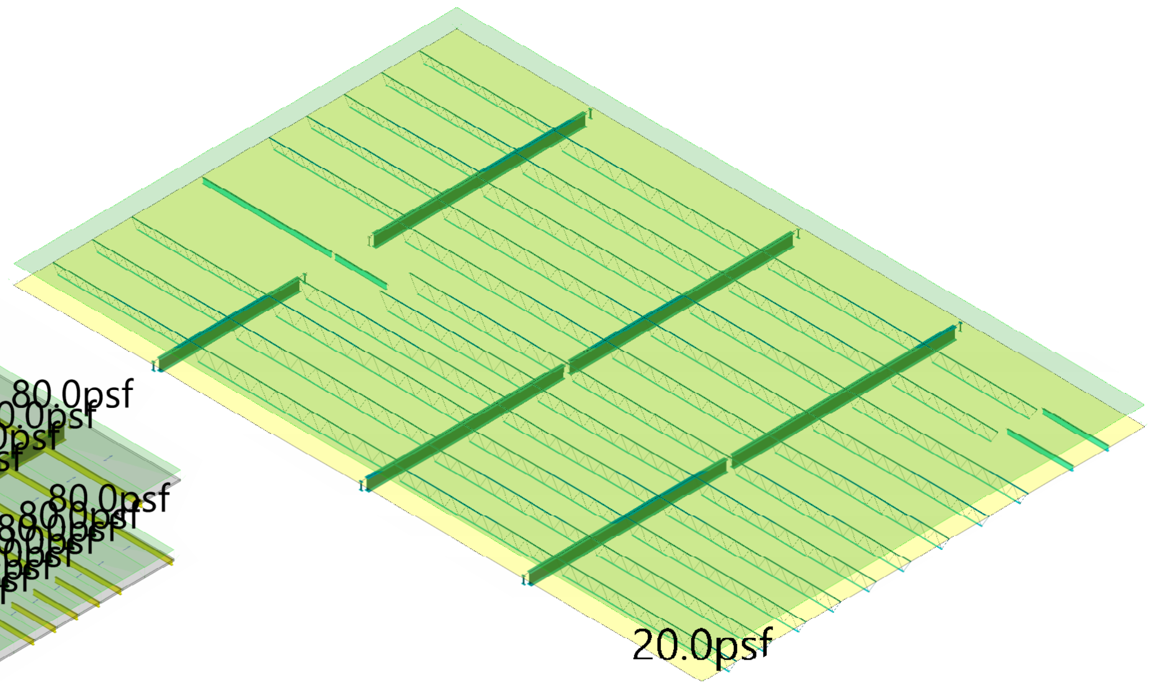


Roof

Tekla Structural Designer: Dead Loading

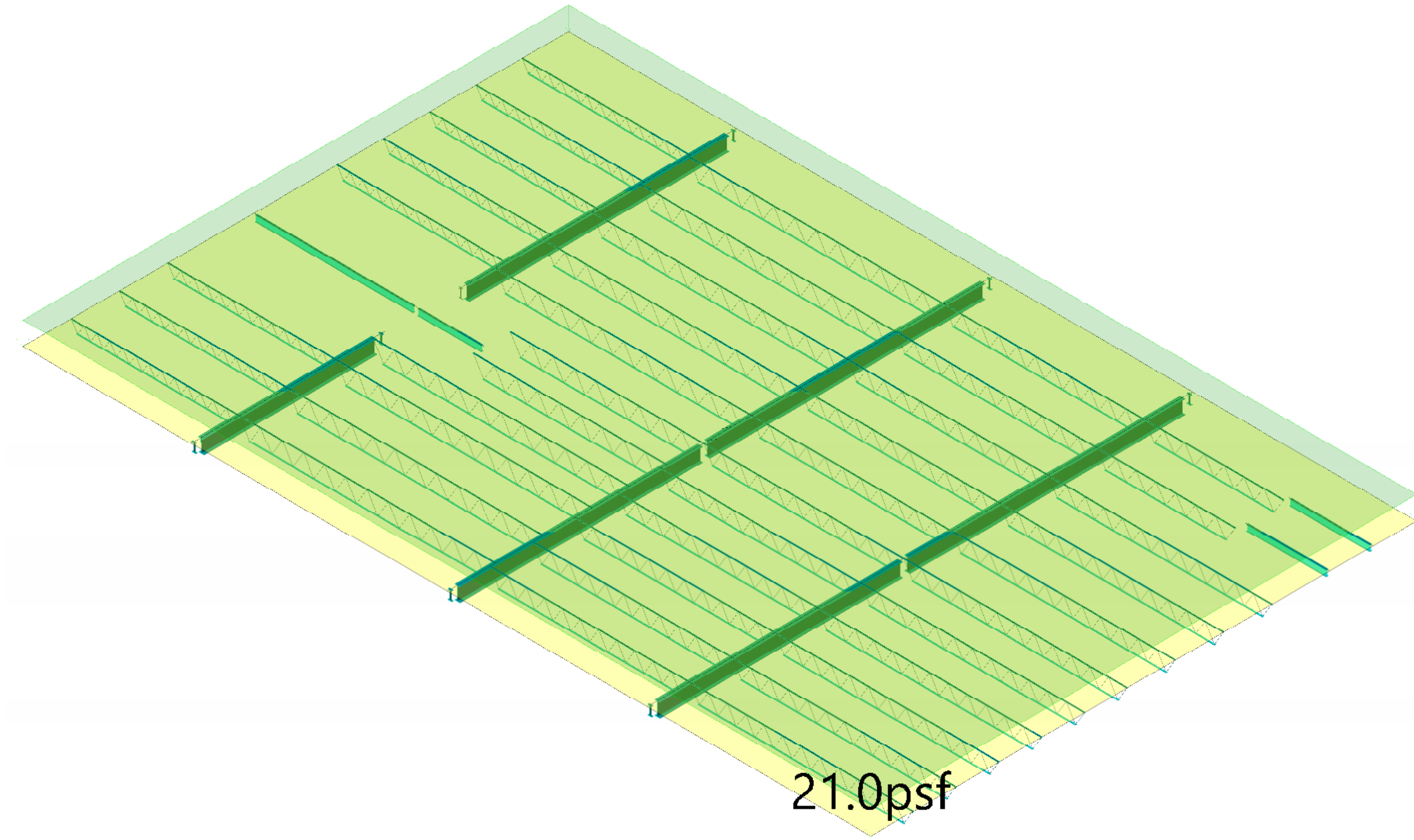


2nd Floor

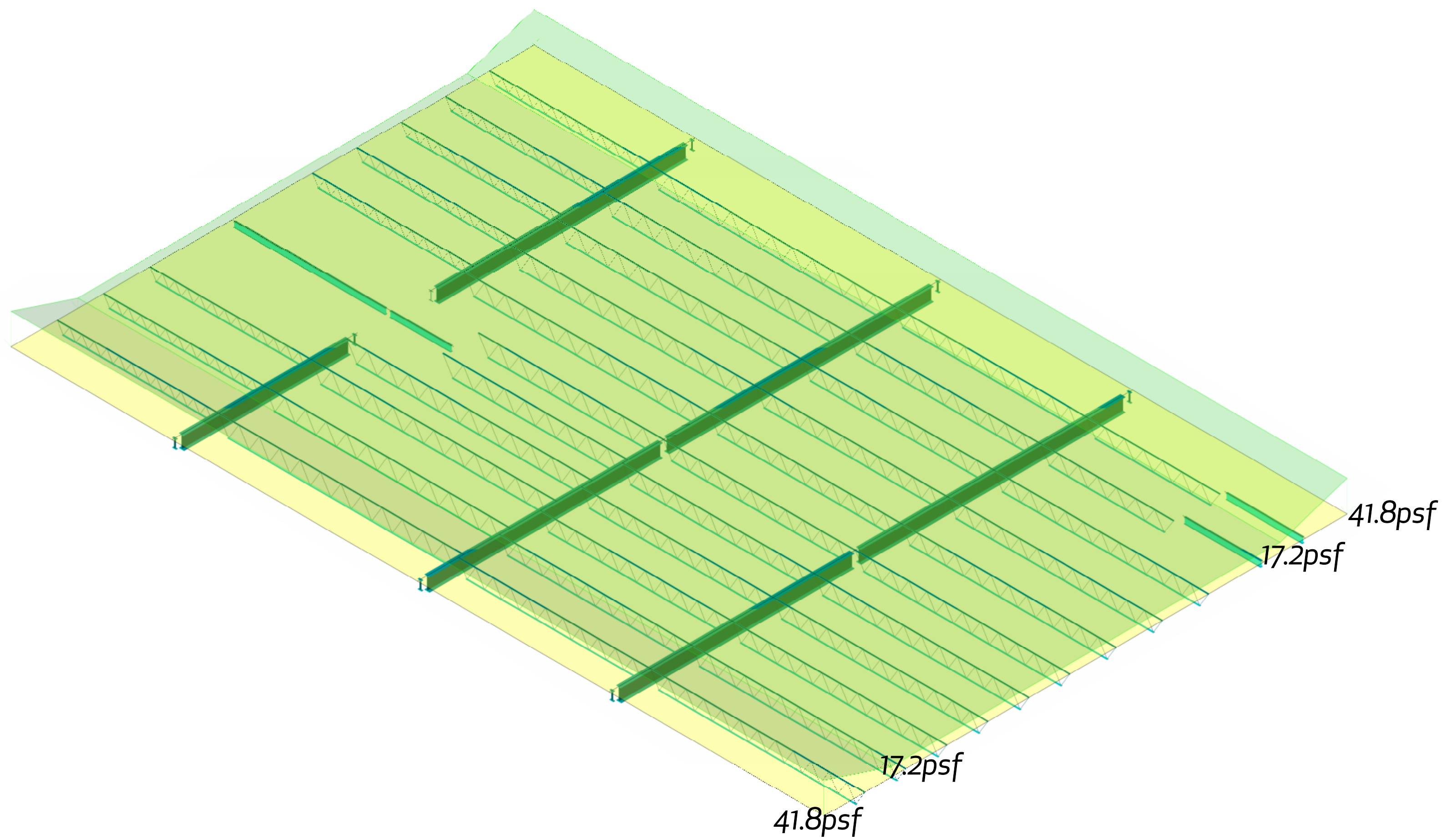


Roof

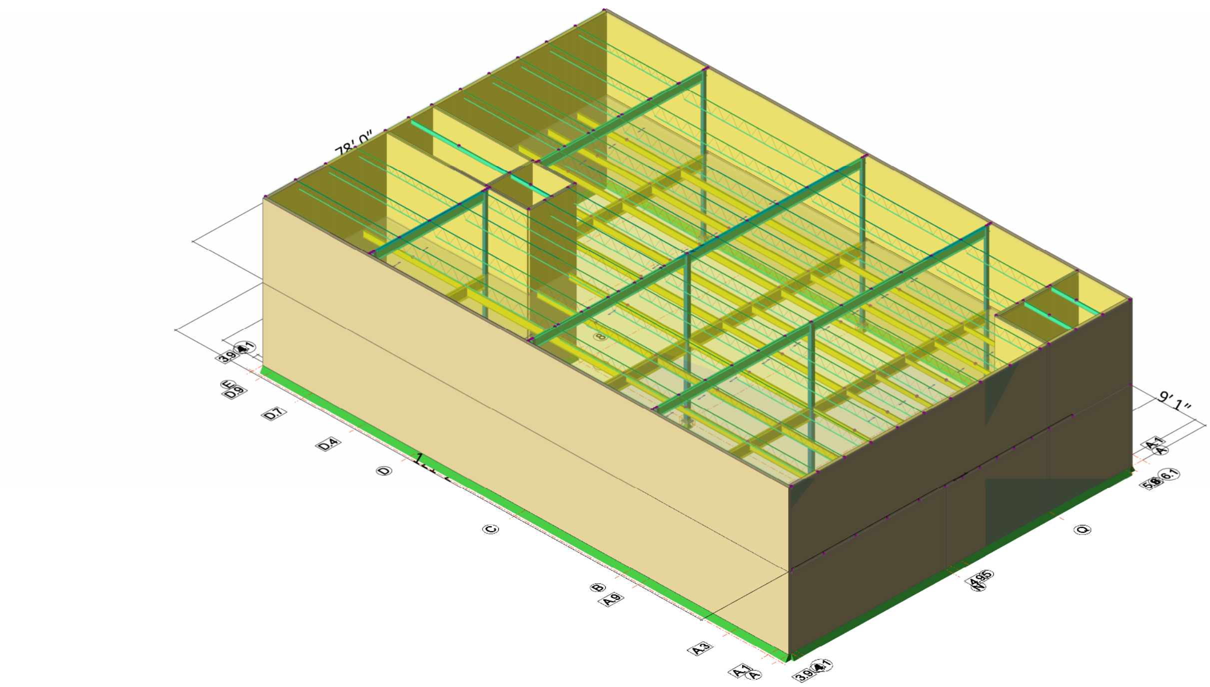
Tekla Structural Designer: Live/Roof Live Loading



Tekla Structural Designer: Minimum Snow Loading



Tekla Structural Designer: Drift Snow Loading



Structure

Structure

Action Codes

General Loading	ASCE7 (2022)
Wind Loading	ASCE7 (2022)
Snow Loading	ASCE7 (2022)
Seismic Loading	ASCE7 (2022)
Combinations	ASCE7 (2022)

Resistance Codes

Steel Design	AISC 360/341 LRFD (2016)
Concrete Design	ACI 318 (2022)
Composite Design	AISC 360/341 LRFD (2016)
Timber Design	NDS LRFD (2018)
Masonry Design	TMS 402/602 ASD (2022)
Foundation Design	No Design Code
Seismic Design and Detailing	ASCE7 (2016)
Steel Fire Design	No Design Code

ConstructionLevels

Ref.	Name	Type	Level [ft, in]	Spacing [ft, in]	Source	Slab Thickness [in]	Floor
2	2	T.O.S	135′ 0″	17′ 8″			●
1	1	T.O.S	117′ 4″	17′ 4″			●
Base	Base	T.O.F	100′ 0″			0	●

Combinations

Name	Class	Active	Strength	Service
1 ASD_1-D	Gravity	●	●	●
2 ASD_2-D+L	Gravity	●	●	●
3 ASD_3-D+Lr	Gravity	●	●	●
4 $ASD_{4.1}$-D+0.7S	Gravity	●	●	●
5 ASD_5-D+0.75L+0.75Lr	Gravity	●	●	●
6 $ASD_{6.1}$-D+0.75L+0.525S	Gravity	●	●	●
7 $LRFD_1$-1.4D	Gravity	●	●	
8 $LRFD_2$-1.2D+1.6L	Gravity	●	●	
9 $LRFD_3$-1.2D+1.6L+0.5Lr	Gravity	●	●	
10 $LRFD_{4.1}$-1.2D+1.6L+0.3S	Gravity	●	●	
11 $LRFD_5$-1.2D+L+1.6Lr	Gravity	●	●	
12 $LRFD_{6.1}$-1.2D+L+S	Gravity	●	●	

1 ASD_1-D

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
2 Slab self weight	1.000	1.000
3 Dead	1.000	1.000

2 ASD_2-D+L

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
2 Slab self weight	1.000	1.000
3 Dead	1.000	1.000
4 Live	1.000	1.000

3 ASD_3-D+Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
2 Slab self weight	1.000	1.000
3 Dead	1.000	1.000
5 Roof Live	1.000	1.000

4 $ASD_{4.1}$-D+0.7S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
2 Slab self weight	1.000	1.000
3 Dead	1.000	1.000

5 ASD_5-D+0.75L+0.75Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
2 Slab self weight	1.000	1.000
3 Dead	1.000	1.000
4 Live	0.750	0.750
5 Roof Live	0.750	0.750

6 $ASD_{6.1}$-D+0.75L+0.525S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.000	1.000
2 Slab self weight	1.000	1.000
3 Dead	1.000	1.000

Loadcase Title	Strength	Service
4 Live	0.750	0.750

7 $LRFD_1$-1.4D

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.400	1.000
2 Slab self weight	1.400	1.000
3 Dead	1.400	1.000

8 $LRFD_2$-1.2D+1.6L

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
4 Live	1.600	1.000

9 $LRFD_3$-1.2D+1.6L+0.5Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
4 Live	1.600	0.750
5 Roof Live	0.500	0.750

10 $LRFD_{4.1}$-1.2D+1.6L+0.3S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
4 Live	1.600	0.750

11 $LRFD_5$-1.2D+L+1.6Lr

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
4 Live	1.000	0.000
5 Roof Live	1.600	1.000

12 $LRFD_{6.1}$-1.2D+L+S

Loadcase Title	Strength	Service
1 Self weight - excluding slabs	1.200	1.000
2 Slab self weight	1.200	1.000
3 Dead	1.200	1.000
4 Live	1.000	0.000

Wind Data

General

Method
ASCE/SEI 7-22 Chapter 27: Directional Procedure

Site Details

Site Ground Level	0”	ft, in
Orientation of Principal Axes relative to Global axes	0.0000	°
Mean Roof Height, h	135’ 0”	ft, in
Level of Highest Opening in Building (Use Mean Roof Height)	135’ 0”	ft, in

Overall Building X' Dimension	121’ 2”	ft, in
Overall Building Y' Dimension	78’ 0”	ft, in
Torsion Design Pressure Factor	75.00	%
Torsion Eccentricity	15.00	%
Wind Loading		
Basic Wind Speed, V	105.00	mph
Directionality Factor, K_d	0.850	
Ground Elevation above Sea Level, z_g	3097	ft
Ground Elevation Factor, K_e	0.894	
Enclosure Classification	Enclosed	
Gust Effect factor, G	0.850	
Principal Axis	+X'	
Exposure Category	B	
Topographic Feature	None	
Principal Axis	+Y'	
Exposure Category	B	
Topographic Feature	None	
Principal Axis	-X'	
Exposure Category	B	
Topographic Feature	None	
Principal Axis	-Y'	
Exposure Category	B	
Topographic Feature	None	

Intermediate Factors

+X'

Height above ground, z	15’ 0”	ft, in
Exposure Coefficient, K_z	0.573	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	20’ 0”	ft, in
Exposure Coefficient, K_z	0.619	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	25’ 0”	ft, in
Exposure Coefficient, K_z	0.656	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	30’ 0”	ft, in
Exposure Coefficient, K_z	0.689	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	40’ 0”	ft, in
Exposure Coefficient, K_z	0.744	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	50’ 0”	ft, in
Exposure Coefficient, K_z	0.790	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	60’ 0”	ft, in
Exposure Coefficient, K_z	0.829	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	80’ 0”	ft, in
Exposure Coefficient, K_z	0.895	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	100’ 0”	ft, in
Exposure Coefficient, K_z	0.950	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	120’ 0”	ft, in

Exposure Coefficient, K_z	0.997	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	135′ 0″	ft, in
Exposure Coefficient, K_z	1.029	
Topographic Factor, K_{zt}	1.000	

+Y'

Height above ground, z	15′ 0″	ft, in
Exposure Coefficient, K_z	0.573	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	20′ 0″	ft, in
Exposure Coefficient, K_z	0.619	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	25′ 0″	ft, in
Exposure Coefficient, K_z	0.656	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	30′ 0″	ft, in
Exposure Coefficient, K_z	0.689	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	40′ 0″	ft, in
Exposure Coefficient, K_z	0.744	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	50′ 0″	ft, in
Exposure Coefficient, K_z	0.790	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	60′ 0″	ft, in
Exposure Coefficient, K_z	0.829	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	80′ 0″	ft, in
Exposure Coefficient, K_z	0.895	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	100′ 0″	ft, in
Exposure Coefficient, K_z	0.950	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	120′ 0″	ft, in
Exposure Coefficient, K_z	0.997	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	135′ 0″	ft, in
Exposure Coefficient, K_z	1.029	
Topographic Factor, K_{zt}	1.000	

-X'

Height above ground, z	15′ 0″	ft, in
Exposure Coefficient, K_z	0.573	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	20′ 0″	ft, in
Exposure Coefficient, K_z	0.619	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	25′ 0″	ft, in
Exposure Coefficient, K_z	0.656	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	30′ 0″	ft, in
Exposure Coefficient, K_z	0.689	
Topographic Factor, K_{zt}	1.000	

Height above ground, z	40’ 0”	ft, in
Exposure Coefficient, K_z	0.744	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	50’ 0”	ft, in
Exposure Coefficient, K_z	0.790	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	60’ 0”	ft, in
Exposure Coefficient, K_z	0.829	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	80’ 0”	ft, in
Exposure Coefficient, K_z	0.895	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	100’ 0”	ft, in
Exposure Coefficient, K_z	0.950	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	120’ 0”	ft, in
Exposure Coefficient, K_z	0.997	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	135’ 0”	ft, in
Exposure Coefficient, K_z	1.029	
Topographic Factor, K_{zt}	1.000	

-Y'

Height above ground, z	15’ 0”	ft, in
Exposure Coefficient, K_z	0.573	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	20’ 0”	ft, in
Exposure Coefficient, K_z	0.619	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	25’ 0”	ft, in
Exposure Coefficient, K_z	0.656	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	30’ 0”	ft, in
Exposure Coefficient, K_z	0.689	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	40’ 0”	ft, in
Exposure Coefficient, K_z	0.744	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	50’ 0”	ft, in
Exposure Coefficient, K_z	0.790	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	60’ 0”	ft, in
Exposure Coefficient, K_z	0.829	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	80’ 0”	ft, in
Exposure Coefficient, K_z	0.895	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	100’ 0”	ft, in
Exposure Coefficient, K_z	0.950	
Topographic Factor, K_{zt}	1.000	
Height above ground, z	120’ 0”	ft, in
Exposure Coefficient, K_z	0.997	
Topographic Factor, K_{zt}	1.000	

Height above ground, z	135′ 0″	ft, in
Exposure Coefficient, K_z	1.029	
Topographic Factor, K_{zt}	1.000	

Velocity Pressures

+X'

Height above ground, z	15′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	12.3	psf
Height above ground, z	20′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	13.3	psf
Height above ground, z	25′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.1	psf
Height above ground, z	30′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.8	psf
Height above ground, z	40′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	15.9	psf
Height above ground, z	50′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	16.9	psf
Height above ground, z	60′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	17.8	psf
Height above ground, z	80′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	19.2	psf
Height above ground, z	100′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	20.4	psf
Height above ground, z	120′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	21.4	psf
Height above ground, z	135′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	22.1	psf

+Y'

Height above ground, z	15′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	12.3	psf
Height above ground, z	20′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	13.3	psf
Height above ground, z	25′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.1	psf
Height above ground, z	30′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.8	psf
Height above ground, z	40′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	15.9	psf
Height above ground, z	50′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	16.9	psf
Height above ground, z	60′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	17.8	psf
Height above ground, z	80′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	19.2	psf
Height above ground, z	100′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	20.4	psf
Height above ground, z	120′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	21.4	psf
Height above ground, z	135′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	22.1	psf

-X'

Height above ground, z	15′ 0″	ft, in

Velocity Pressure, $q_z \times K_d$	12.3	psf
Height above ground, z	20′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	13.3	psf
Height above ground, z	25′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.1	psf
Height above ground, z	30′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.8	psf
Height above ground, z	40′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	15.9	psf
Height above ground, z	50′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	16.9	psf
Height above ground, z	60′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	17.8	psf
Height above ground, z	80′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	19.2	psf
Height above ground, z	100′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	20.4	psf
Height above ground, z	120′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	21.4	psf
Height above ground, z	135′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	22.1	psf

-Y'

Height above ground, z	15′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	12.3	psf
Height above ground, z	20′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	13.3	psf
Height above ground, z	25′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.1	psf
Height above ground, z	30′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	14.8	psf
Height above ground, z	40′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	15.9	psf
Height above ground, z	50′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	16.9	psf
Height above ground, z	60′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	17.8	psf
Height above ground, z	80′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	19.2	psf
Height above ground, z	100′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	20.4	psf
Height above ground, z	120′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	21.4	psf
Height above ground, z	135′ 0″	ft, in
Velocity Pressure, $q_z \times K_d$	22.1	psf

Generated Loadcases

11 Wind +X',GCpi 0.18,-Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load [kip]
WI 1	W120	10.6	1560	16.5
	W135	11.0	1170	12.9
WI 2	S	-17.1	4241	-72.5

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load [kip]
WI 3	L	-11.3	2730	-30.8
WI 4	S	-17.1	4241	-72.5

Average Wall Loads

Windward	10.8	psf
Leeward	-11.3	psf
Left	-17.1	psf
Right	-17.1	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
RI 1	1	-23.5	5265	-123.6
	2	-17.1	4186	-71.6

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	0.0	14.8	42′ 0″	-
St. 1 (1)	0.0	30.0	42′ 0″	-
St. 2 (2)	0.0	15.4	42′ 0″	-
Total	**0.0**	**60.2**	-	-

Total Loads

X	0.0	kip
Y	60.2	kip
Z	-195.1	kip

12 Wind +X',GCpi 0.18,+Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
WI 1	W120	10.6	1560	16.5
	W135	11.0	1170	12.9
WI 2	S	-17.1	4241	-72.5
WI 3	L	-11.3	2730	-30.8
WI 4	S	-17.1	4241	-72.5

Average Wall Loads

Windward	10.8	psf
Leeward	-11.3	psf
Left	-17.1	psf
Right	-17.1	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
RI 1	1	-7.3	5265	-38.7
	2	-7.3	4186	-30.7

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	0.0	14.8	42′ 0″	-
St. 1 (1)	0.0	30.0	42′ 0″	-

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. 2 (2)	0.0	15.4	42′ 0″	-
Total	**0.0**	**60.2**	-	-

Total Loads

X	0.0	kip
Y	60.2	kip
Z	-69.4	kip

13 Wind +X'+Y',GCpi 0.18

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
WI 1	WX120	7.9	1560	12.4
	WX135	8.3	1170	9.7
WI 2	WY120	7.9	2423	19.2
	WY135	8.3	1818	15.0
WI 3	LX	-8.5	2730	-23.1
WI 4	LY	-10.0	4241	-42.4

Average Wall Loads

Windward	8.1	psf
Leeward	-9.4	psf
Left	0.0	psf
Right	0.0	psf

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-18.8	11.1	42′ 0″	60′ 9″
St. 1 (1)	-38.3	22.5	42′ 0″	60′ 9″
St. 2 (2)	-19.6	11.5	42′ 0″	60′ 9″
Total	**-76.7**	**45.1**	-	-

Total Loads

X	-76.7	kip
Y	45.1	kip
Z	0.0	kip

14 Wind +Y',GCpi 0.18,-Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
WI 1	S	-17.1	2730	-46.7
WI 2	W120	10.6	2423	25.6
	W135	11.0	1818	20.0
WI 3	S	-17.1	2730	-46.7
WI 4	L	-13.3	4241	-56.6

Average Wall Loads

Windward	10.8	psf
Leeward	-13.3	psf
Left	-17.1	psf
Right	-17.1	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
RI 1	1	-23.5	8179	-191.9
	2	-17.1	1272	-21.7

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-25.1	0.0	-	60' 9"
St. 1 (1)	-51.1	0.0	-	60' 9"
St. 2 (2)	-26.1	0.0	-	60' 9"
Total	**-102.2**	**0.0**	**-**	-

Total Loads

X	-102.2	kip
Y	0.0	kip
Z	-213.7	kip

15 Wind +Y',GCpi 0.18,+Cp

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
WI 1	S	-17.1	2730	-46.7
WI 2	W120	10.6	2423	25.6
	W135	11.0	1818	20.0
WI 3	S	-17.1	2730	-46.7
WI 4	L	-13.3	4241	-56.6

Average Wall Loads

Windward	10.8	psf
Leeward	-13.3	psf
Left	-17.1	psf
Right	-17.1	psf

Roof Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
RI 1	1	-7.3	8179	-60.1
	2	-7.3	1272	-9.3

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-25.1	0.0	-	60' 9"
St. 1 (1)	-51.1	0.0	-	60' 9"
St. 2 (2)	-26.1	0.0	-	60' 9"
Total	**-102.2**	**0.0**	**-**	-

Total Loads

X	-102.2	kip
Y	0.0	kip
Z	-69.4	kip

16 Wind -X'+Y',GCpi 0.18

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
WI 1	LX	-8.5	2730	-23.1
WI 2	WY120	7.9	2423	19.2
	WY135	8.3	1818	15.0
WI 3	WX120	7.9	1560	12.4
	WX135	8.3	1170	9.7
WI 4	LY	-10.0	4241	-42.4

Average Wall Loads

Windward	8.1	psf
Leeward	-9.4	psf
Left	0.0	psf
Right	0.0	psf

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	-18.8	-11.1	42′ 0″	60′ 9″
St. 1 (1)	-38.3	-22.5	42′ 0″	60′ 9″
St. 2 (2)	-19.6	-11.5	42′ 0″	60′ 9″
Total	**-76.7**	**-45.1**	-	-

Total Loads

X	-76.7	kip
Y	-45.1	kip
Z	0.0	kip

17 Wind -X'-Y',GCpi 0.18

Wall Zone Loads

Reference	Zone	Design Pressure [psf]	Area [ft^2]	Applied Load
WI 1	LX	-8.5	2730	-23.1
WI 2	LY	-10.0	4241	-42.4
WI 3	WX120	7.9	1560	12.4
	WX135	8.3	1170	9.7
WI 4	WY120	7.9	2423	19.2
	WY135	8.3	1818	15.0

Average Wall Loads

Windward	8.1	psf
Leeward	-9.4	psf
Left	0.0	psf
Right	0.0	psf

Lateral Loads by Level

Level	Total [kip]		Center [ft, in]	
	X	Y	X	Y
St. Base (Base)	18.8	-11.1	42′ 0″	60′ 9″
St. 1 (1)	38.3	-22.5	42′ 0″	60′ 9″
St. 2 (2)	19.6	-11.5	42′ 0″	60′ 9″
Total	**76.7**	**-45.1**	-	-

Total Loads

X	76.7	kip

Y -45.1 kip
Z 0.0 kip

Snow Data

General

Method ASCE7

Basic data

Ground Snow Load, p_g	21.0	psf	ASCE7-22 Clause 7.2
Terrain Category	C (open terrain scattered obstructions)		
Exposure	Partially exposed		
Exposure Factor, C_e	1.000		ASCE7-22 Clause 7.3.1
Thermal Condition	All except below		
Roof Heat Transfer Resistance, R_{roof}	30.0	ft^2°Fh/Btu	
Roof Heat Transfer Measure, U_{roof}	0.03	Btu/h/ft^2/°F	
Thermal Factor, C_t	1.167		ASCE7-22 Clause 7.3.2
Risk Category	III		
Winter Wind Parameter, W_2	0.350		
Flat Roof Snow Load, p_f	17.2	psf	ASCE7-22 Clause 7.3

Snow load cases

Minimum Snow Load	Yes		
Minimum snow load, p_m	35.0	psf	ASCE7-22 Clause 7.3.3
Balanced Snow Load	Yes		
Unbalanced Snow Load	Yes		
Drift Snow Load	Yes		
Rain on Snow Surcharge	No		

Structure

Loadcase Summary

1 Self weight - excluding slabs

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	101.8
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	5.3	0.0	0.0
Total NHF Dir2	0.0	5.3	0.0
Decomposable Loads	0.0	0.0	1662.1
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	1763.9
Total Load on Structure	0.0	0.0	1763.9
Total Reaction	0.0	0.0	1763.9

2 Slab self weight

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	1.1	0.0	0.0
Total NHF Dir2	0.0	1.1	0.0
Decomposable Loads	0.0	0.0	350.8
1 Way Decomp Results	0.0	0.0	350.8
2 Way Decomp Results	0.0	0.0	0.0

Load Type	Force [kip]		
	X	Y	Z
Total User Applied Load	0.0	0.0	350.8
Total Load on Structure	0.0	0.0	350.8
Total Reaction	0.0	0.0	350.8

3 Dead

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	2.9	0.0	0.0
Total NHF Dir2	0.0	2.9	0.0
Decomposable Loads	0.0	0.0	955.8
1 Way Decomp Results	0.0	0.0	955.8
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	955.8
Total Load on Structure	0.0	0.0	955.8
Total Reaction	0.0	0.0	955.8

4 Live

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	2.6	0.0	0.0
Total NHF Dir2	0.0	2.6	0.0
Decomposable Loads	0.0	0.0	860.0
1 Way Decomp Results	0.0	0.0	860.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	860.0
Total Load on Structure	0.0	0.0	860.0
Total Reaction	0.0	0.0	860.0

5 Roof Live

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.6	0.0	0.0
Total NHF Dir2	0.0	0.6	0.0
Decomposable Loads	0.0	0.0	189.0
1 Way Decomp Results	0.0	0.0	189.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	189.0
Total Load on Structure	0.0	0.0	189.0
Total Reaction	0.0	0.0	189.0

7 Min Lateral Load $_{\text{A Dir1+}}$

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0

Load Type	Force [kip]		
	X	Y	Z
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	0.0
Total Load on Structure	0.0	0.0	0.0
Total Reaction	0.0	0.0	0.0

8 Min Lateral Load $_{\text{A Dir1-}}$

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	0.0
Total Load on Structure	0.0	0.0	0.0
Total Reaction	0.0	0.0	0.0

9 Min Lateral Load $_{\text{A Dir2+}}$

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	0.0
Total Load on Structure	0.0	0.0	0.0
Total Reaction	0.0	0.0	0.0

10 Min Lateral Load $_{\text{A Dir2-}}$

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	0.0
Total Load on Structure	0.0	0.0	0.0
Total Reaction	0.0	0.0	0.0

11 Wind +X',GCpi 0.18,-Cp

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	60.2	-195.1
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	60.2	-195.1
Total Load on Structure	0.0	60.2	-195.1
Total Reaction	0.0	-60.2	-195.1

12 Wind +X',GCpi 0.18,+Cp

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	60.2	-69.4
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	60.2	-69.4
Total Load on Structure	0.0	60.2	-69.4
Total Reaction	0.0	-60.2	-69.4

13 Wind +X'+Y',GCpi 0.18

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-76.7	45.1	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-76.7	45.1	0.0
Total Load on Structure	-76.7	45.1	0.0
Total Reaction	76.7	-45.1	0.0

14 Wind +Y',GCpi 0.18,-Cp

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-102.2	0.0	-213.7
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-102.2	0.0	-213.7

Load Type	Force [kip]		
	X	Y	Z
Total Load on Structure	-102.2	0.0	-213.7
Total Reaction	102.2	0.0	-213.7

15 Wind +Y',GCpi 0.18,+Cp

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-102.2	0.0	-69.4
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-102.2	0.0	-69.4
Total Load on Structure	-102.2	0.0	-69.4
Total Reaction	102.2	0.0	-69.4

16 Wind -X'+Y',GCpi 0.18

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	-76.7	-45.1	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	-76.7	-45.1	0.0
Total Load on Structure	-76.7	-45.1	0.0
Total Reaction	76.7	45.1	0.0

17 Wind -X'-Y',GCpi 0.18

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	76.7	-45.1	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	76.7	-45.1	0.0
Total Load on Structure	76.7	-45.1	0.0
Total Reaction	-76.7	45.1	0.0

27 Minimum Snow Load

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0

Load Type	Force [kip]		
	X	Y	Z
Decomposable Loads	0.0	0.0	198.5
1 Way Decomp Results	0.0	0.0	198.5
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	198.5
Total Load on Structure	0.0	0.0	198.5
Total Reaction	0.0	0.0	198.5

28 Balanced Snow Load

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	162.1
1 Way Decomp Results	0.0	0.0	162.1
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	162.1
Total Load on Structure	0.0	0.0	162.1
Total Reaction	0.0	0.0	162.1

29 Unbalanced Snow Load 1

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	0.0
1 Way Decomp Results	0.0	0.0	0.0
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	0.0
Total Load on Structure	0.0	0.0	0.0
Total Reaction	0.0	0.0	0.0

30 Drift Snow Load 1

Load Type	Force [kip]		
	X	Y	Z
Member Loads	0.0	0.0	0.0
Nodal Loads	0.0	0.0	0.0
Total NHF Dir1	0.0	0.0	0.0
Total NHF Dir2	0.0	0.0	0.0
Decomposable Loads	0.0	0.0	212.7
1 Way Decomp Results	0.0	0.0	212.7
2 Way Decomp Results	0.0	0.0	0.0
Total User Applied Load	0.0	0.0	212.7
Total Load on Structure	0.0	0.0	212.7
Total Reaction	0.0	0.0	212.7

Michael Baker INTERNATIONAL

Gravity Framing and Column Calculations

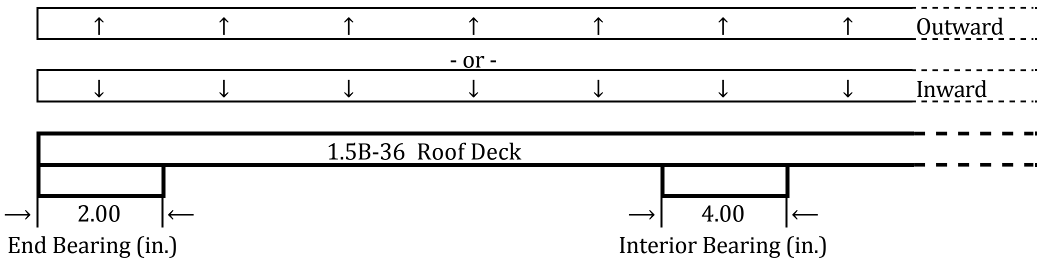
20 Gage 1.5B-36 Grade 50

Uniform Design Load Table, LRFD (psf)

For End Lapped Deck

36 / 7/ 4 Connection Pattern to Supports with
Hilti X-HSN 24 PAF

Support Member A572 GR50
$0.25 \le t_2$ (in.) ≤ 0.375



Inward Uniform Design Load Table, LRFD (psf)

Span	(ft-in)	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"
1	ΦWn	187	159	137	119	105	93	83	74
	L/240	60	47	38	31	25	21	18	15
2	ΦWn	189	161	139	121	107	95	84	76
	L/240	159	125	100	81	67	56	47	40
3	ΦWn	235	201	173	151	133	118	105	95
	L/240	124	98	78	64	52	44	37	31

$> 1.2D + 1.6LL = 1.2(25) + 1.6(20) = 62$psf

Uplift (Outward) Uniform Design Load Table, LRFD (psf)

Span	(ft-in)	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"
1	ΦWn	191	163	140	122	107	95	85	76
	ΦRn	276	255	236	221	207	195	184	174
	L/240	66	52	41	34	28	23	20	17
2	ΦWn	110	102	95	88	83	78	74	70
	ΦRn	110	102	95	88	83	78	74	70
	L/240	144	113	91	74	61	51	43	36
3	ΦWn	125	116	107	100	94	88	84	79
	ΦRn	125	116	107	100	94	88	84	79
	L/240	113	89	71	58	48	40	33	28

$> 0.9D + W = 0.9(9) + (-47.3) = -39.2$ psf

Steel Deck Properties

t	Fy	wdd	Id+	Id-	Se+	Se-	ΦMn+	ΦMn-	ΦVn
in	ksi	psf	$in.^4$/ft	$in.^4$/ft	$in.^3$/ft	$in.^3$/ft	lbs-ft/ft	lbs-ft/ft	lbs/ft
0.0358	50	2	0.197	0.217	0.224	0.229	840	859	4874

Where: W ≤ ΦWn

W = Required strength of the governing LRFD load combintaion

ΦWn = Design Strength

Bare Deck Uniform Load V3.1 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-0652
CAN/CSA-S136 (R2021) for Canadian references

Date: 10/16/2025

Vulcraft Open Web Steel Joist Depth Selector
Parallel Chord Joist Only

Key: R4cnlF4d3gg

NUCOR VULCRAFT

Project Information:

Allowable Stress Design

Project:	CREECH DRP PH 2
Design Professional:	ACV

Input Design Criteria

Bay	Span			Spacing			Maximum Depth	Loading Information			
								Dead	Live	Snow/Rain	Wind Down
	(ft)	(in.)	(fraction in,)	(ft)	(in.)	(fraction in.)	(in)	(psf)	(psf)	(psf)	(psf)
1	36	7		7	0			25	20	21	9.3
2	32	0		7	0			25	20	21	9.3
3	24	0		7	0			25	20	21	9.3
4											

Note:

Snow drift, partial span uniform and wind uplift loads are beyond the scope of this tool.

Results are based on the least cost joist. It is possible that alternate depths will result in nearly identical cost and thus have a similar cost ratio.

Blank rows indicate information is not available for the next depth increment either above or below the previous row.

Bay	Joist Designation Depth(in) Spec. TL/LL(plf)	Self Weight (psf)	Weight Ratio	Cost Ratio	LL Defl. Ratio Span/XX	Bearing Depth (in)	Estimated TC Width (in)	Required Bridging H/X/EX	Explanation
1	26 K 322 / 147	1.4	0.985	1.016	564	2 1/2"	5	2/0/0	Second depth less
	28 K 322 / 147	1.4	0.986	1.004	598	2 1/2"	5	2/0/0	First depth less
	30 K 322 / 147	**1.4**	**1.000**	**1.000**	**648**	**2 1/2"**	**5**	**2/0/0**	**Least Cost Depth**
	32 LH 322 / 147	1.6	1.105	1.062	764	5"	5	2/0/0	First depth greater
	36 LH 322 / 147	1.5	1.017	1.171	927	5"	5	2/0/0	Second depth greater
	22 K 322 / 147	**1.5**	**1.059**	**1.125**	**469**	**2 1/2"**	**5**	**2/0/0**	**Commonly selected depth**
2	22 K 322 / 147	1.3	0.921	1.040	511	2 1/2"	5	1/0/0	Second depth less
	24 K 322 / 147	1.4	0.985	1.023	561	2 1/2"	5	2/0/0	First depth less
	26 K 322 / 147	**1.4**	**1.000**	**1.000**	**631**	**2 1/2"**	**5**	**2/0/0**	**Least Cost Depth**
	28 K 322 / 147	1.4	1.012	1.006	737	2 1/2"	5	2/0/0	First depth greater
	30 K 322 / 147	1.4	1.027	1.033	807	2 1/2"	5	2/0/0	Second depth greater
	20 K 322 / 147	**1.4**	**0.992**	**1.098**	**444**	**2 1/2"**	**5**	**2/0/0**	**Commonly selected depth**
3	14 K 322 / 147	1.3	1.075	1.104	450	2 1/2"	4	2/0/0	Second depth less
	16 K 322 / 147	1.2	1.026	1.017	597	2 1/2"	4	2/0/0	First depth less
	18 K 322 / 147	**1.2**	**1.000**	**1.000**	**656**	**2 1/2"**	**4**	**2/0/0**	**Least Cost Depth**
	20 K 322 / 147	1.3	1.078	1.087	794	2 1/2"	5	1/0/0	First depth greater
	22 K 322 / 147	1.3	1.087	1.075	966	2 1/2"	5	1/0/0	Second depth greater
	16 K 322 / 147	**1.2**	**1.026**	**1.017**	**597**	**2 1/2"**	**4**	**2/0/0**	**Commonly selected depth**
4									

Depth selection is based on standard Steel Joist Institute (SJI) depths (parallel chord only): 10" through 32" deep - 2" increments, greater than 32" through 72" deep - 4" increments, greater than 72" deep - 8" increments. For alternate depths and configurations please contact your local Vulcraft Sales Representative.

The commonly selected joist depth is provided as a comparison of a typical depth for the span to the least cost depth for the span recognizing that there may be head room considerations for the joist depth selection.

Table generated using calculator V1.21 Released 3/2023

Date: 11/6/2025

Steel

Beam Design

Beam Design Summary

Static

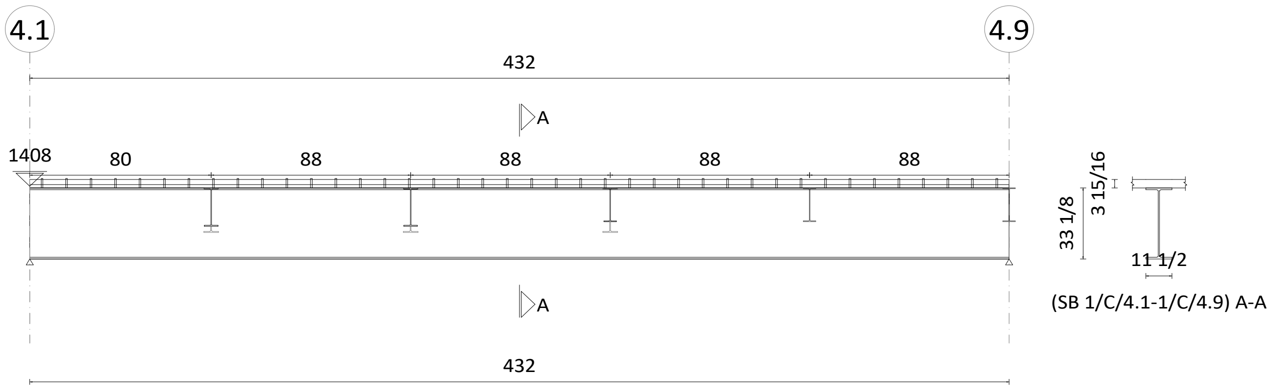
Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	No. Connectors	Camber	Utilization	Status
SB 1/D.4/#443-1/E/#444	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.497	Pass
SB 1/D.4/#445-1/E/#446	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.467	Pass
SB 1/%69-1/A/#1043	SBRC18	1	W 8x10	A992-50	7′ 0″	7	0.00	0.158	Warning
SB 1/D.4/4.1-1/D.4/4.7	SBRC2	1	W 30x99	A992-50	26′ 0″	24	0.00	0.735	Pass
SB 1/D.4/5-1/D.4/5.9	SBRC4	1	W 33x130	A992-50	38′ 4″	40	0.00	0.719	Pass
SB 1/D.4/5.9-1/D.4/6	SBRC10	1	W 16x36	A992-50	8″	2	0.00	0.000	Pass
SB 1/8/4.9-1/C/4.9	SBRC1	1	W 16x26	A992-50	27′ 7″	30	0.00	0.667	Pass
SB 1/C/4-1/C/4.1	SBRC6	1	W 16x36	A992-50	1′ 0″	2	0.00	0.000	Pass
SB 1/C/4.1-1/C/4.9	SBRC7	1	W 33x130	A992-50	36′ 0″	40	0.00	0.825	Pass
SB 1/C/4.9-1/C/5.9	SBRC8	1	W 33x130	A992-50	40′ 8″	40	0.00	0.867	Pass
SB 1/C/5.9-1/C/6	SBRC10	1	W 16x36	A992-50	8″	2	0.00	0.000	Pass
SB 1/C/4.9-1/A.9/4.9	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.705	Pass
SB 1/A.9/4-1/A.9/4.1	SBRC6	1	W 16x36	A992-50	1′ 0″	2	0.00	0.000	Pass
SB 1/A.9/4.1-1/A.9/4.9	SBRC7	1	W 30x99	A992-50	36′ 0″	40	0.00	0.869	Pass
SB 1/A.9/4.9-1/A.9/5.9	SBRC8	1	W 30x99	A992-50	40′ 8″	40	0.00	0.901	Pass
SB 1/A.9/5.9-1/A.9/6	SBRC10	1	W 16x36	A992-50	8″	2	0.00	0.000	Pass
SB 1/A.9/4.9-1/10/4.9	SBRC9	1	W 12x14	A992-50	19′ 11″	18	0.00	0.572	Warning
SB 1/D.4/#439-1/E/#440	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.497	Pass
SB 1/D.4/#441-1/E/#442	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.497	Pass
SB 2/9/#924-2/8/#925	SBR1	1	W 10x12	A992-50	9′ 9″		0.00	0.087	Pass
SB 2/9/#924-2/E/#930	SBR10	1	W 10x12	A992-50	23′ 3″		0.00	0.497	Pass
SB 2/10/#928-2/A/#929	SBR10	1	W 10x12	A992-50	12′ 1″		0.00	0.156	Pass
SB 1/10/#462-1/A.9/#454	SBRC9	1	W 12x14	A992-50	19′ 11″	18	0.00	0.706	Warning
SB 1/C/#450-1/D.4/#443	SBRC12	1	W 18x40	A992-50	36′ 7″	40	0.00	0.657	Pass
SB 1/C/#447-1/D.4/#441	SBRC12	1	W 18x40	A992-50	36′ 7″	40	0.00	0.657	Pass
SB 1/D.4/#437-1/E/#438	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.497	Pass
SB 1/A.9/#483-1/C/#476	SBRC1	1	W 18x40	A992-50	28′ 7″	32	0.00	0.579	Pass
SB 1/A.1/#487-1/A.9/#1035	SBRC15	1	W 18x40	A992-50	29′ 8″	30	0.00	0.526	Pass
SB 1/%70-1/A/#1042	SBRC18	1	W 8x10	A992-50	7′ 0″	7	0.00	0.158	Warning
SB 1/A.1/#487-1/A/#487	SBRC16	1	W 18x40	A992-50	2′ 4″	2	0.00	0.011	Pass
SB 1/A.9/#456-1/C/#455	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.679	Pass
SB 1/A.9/#451-1/C/#450	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.641	Pass
SB 1/A.9/#452-1/C/#447	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.641	Pass
SB 1/A.9/#453-1/C/#448	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.641	Pass
SB 1/10/#457-1/A.9/#456	SBRC9	1	W 12x14	A992-50	19′ 11″	18	0.00	0.749	Warning
SB 1/10/#459-1/A.9/#451	SBRC9	1	W 12x14	A992-50	19′ 11″	18	0.00	0.706	Warning
SB 1/10/#460-1/A.9/#452	SBRC9	1	W 12x14	A992-50	19′ 11″	18	0.00	0.706	Warning
SB 1/10/#461-1/A.9/#453	SBRC9	1	W 12x14	A992-50	19′ 11″	18	0.00	0.706	Warning
SB 1/A.9/#474-1/C/#473	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.694	Pass

Note: Warning is from stud diameter, stud diameter will fit within flange of beam.

Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	No. Connectors	Camber	Utilization	Status
SB 1/D.4/#480-1/E/#490	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.759	Pass
SB 1/8/#472-1/C/#473	SBRC1	1	W 16x26	A992-50	27′ 7″	30	0.00	0.790	Pass
SB 2/D.4/4.7-2/D.4/P	SBR2	1	W 16x36	A992-50	1′ 0″		0.00	0.000	Not required
SB 2/D.4/4-2/D.4/4.1	SBR9	1	W 16x36	A992-50	1′ 0″		0.00	0.000	Not required
SB 2/D.4/4.1-2/D.4/4.7	SBR3	1	W 24x68	A992-50	26′ 0″		0.00	0.263	Pass
SB 2/D.4/5-2/D.4/5.9	SBR4	1	W 24x68	A992-50	38′ 4″		0.00	0.559	Pass
SB 2/D.4/5.9-2/D.4/6	SBR7	1	W 16x36	A992-50	8″		0.00	0.000	Not required
SB 2/C/4-2/C/4.1	SBR9	1	W 16x36	A992-50	1′ 0″		0.00	0.000	Not required
SB 2/C/4.1-2/C/4.9	SBR5	1	W 24x68	A992-50	36′ 0″		0.00	0.517	Pass
SB 2/C/4.9-2/C/5.9	SBR6	1	W 24x68	A992-50	40′ 8″		0.00	0.674	Pass
SB 2/C/5.9-2/C/6	SBR7	1	W 16x36	A992-50	8″		0.00	0.000	Not required
SB 2/A.9/4-2/A.9/4.1	SBR9	1	W 16x36	A992-50	1′ 0″		0.00	0.000	Not required
SB 2/A.9/4.1-2/A.9/4.9	SBR5	1	W 24x68	A992-50	36′ 0″		0.00	0.495	Pass
SB 2/A.9/4.9-2/A.9/5.9	SBR6	1	W 24x68	A992-50	40′ 8″		0.00	0.584	Pass
SB 2/A.9/5.9-2/A.9/6	SBR7	1	W 16x36	A992-50	8″		0.00	0.000	Not required
SB 1/A/#1040-1/A.9/#1041	SBRC6	1	W 16x26	A992-50	32′ 0″	32	0.00	0.718	Pass
SB 1/%70-1/A.9/#1039	SBRC11	1	W 12x14	A992-50	25′ 0″	25	0.00	0.779	Warning
SB 1/%69-1/A.9/#1038	SBRC11	1	W 12x14	A992-50	25′ 0″	25	0.00	0.779	Warning
SB 1/#992/#1037-1/#487/#1037	SBRC17	1	W 16x26	A992-50	16′ 0″	16	0.00	0.540	Warning
SB 2/10/#926-2/A/#927	SBR10	1	W 10x12	A992-50	12′ 1″		0.00	0.156	Pass
SB 1/A.1/#992-1/A/#992	SBRC16	1	W 18x40	A992-50	2′ 4″	2	0.00	0.011	Pass
SB 1/A.1/#992-1/A.9/#1036	SBRC15	1	W 18x40	A992-50	29′ 8″	30	0.00	0.516	Pass
SB 2/D.4/O-2/D.4/5	SBR8	1	W 16x36	A992-50	8″		0.00	0.000	Not required
SB 1/D.4/4-1/D.4/4.1	SBRC6	1	W 33x130	A992-50	1′ 0″	2	0.00	0.000	Pass
SB 1/C/#448-1/D.4/#439	SBRC12	1	W 18x40	A992-50	36′ 7″	40	0.00	0.657	Pass
SB 1/C/#449-1/D.4/#437	SBRC12	1	W 18x40	A992-50	36′ 7″	40	0.00	0.657	Pass
SB 1/A.9/#454-1/C/#449	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.641	Pass
SB 1/C/#455-1/D.4/#445	SBRC12	1	W 18x40	A992-50	36′ 7″	40	0.00	0.688	Pass
SB 1/D.4/#482-1/E/#489	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.674	Pass
SB 1/D.4/#477-1/E/#478	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.777	Pass
SB 1/C/#476-1/D.4/#477	SBRC12	1	W 21x44	A992-50	36′ 7″	40	0.00	0.834	Pass
SB 1/C/#479-1/D.4/#480	SBRC12	1	W 21x44	A992-50	36′ 7″	40	0.00	0.816	Pass
SB 1/C/#481-1/D.4/#482	SBRC12	1	W 21x44	A992-50	36′ 7″	40	0.00	0.808	Pass
SB 1/A.9/#484-1/C/#479	SBRC1	1	W 18x40	A992-50	28′ 7″	32	0.00	0.508	Pass
SB 1/A.9/#485-1/C/#481	SBRC1	1	W 16x26	A992-50	28′ 7″	32	0.00	0.694	Pass
SB 1/11/#623-1/A/#993	SBRC3	1	W 10x12	A992-50	8′ 5″	10	0.00	0.108	Warning
SB 1/12/#623-1/A/#994	SBRC3	1	W 10x12	A992-50	8′ 5″	10	0.00	0.112	Warning
SB 1/13/#623-1/A/#995	SBRC3	1	W 10x12	A992-50	8′ 5″	10	0.00	0.112	Warning
SB 1/14/#623-1/A/#996	SBRC3	1	W 10x12	A992-50	8′ 5″	10	0.00	0.127	Warning
SB 1/10/#999-1/A/#1000	SBRC9	1	W 10x12	A992-50	12′ 1″	10	0.00	0.291	Warning
SB 1/D.4/4.7-1/D.4/P	SBRC5	1	W 33x130	A992-50	1′ 0″	2	0.00	0.000	Pass
SB 1/10/#813-1/A/#814	SBRC9	1	W 10x12	A992-50	12′ 1″	10	0.00	0.159	Warning
SB 1/D.4/O-1/D.4/5	SBRC13	1	W 33x130	A992-50	8″	2	0.00	0.000	Pass
SB 1/P/#850-1/O/#850	SBRC14	1	W 10x12	A992-50	10′ 8″	10	0.00	0.140	Warning

Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	No. Connectors	Camber	Utilization	Status
SB 1/A.9/#1044-1/N/10	SBRC9	1	W 12x14	A992-50	19' 11"	18	0.00	0.473	⚠ Warning

SB 1/C/4.1-1/C/4.9



St. 1 (1): SB 1/C/4.1-1/C/4.9 - 1 (W 33x130 A992-50 (40) [Deformed # 3 @ 8])

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	6′ 8″		1.000		1.000		1.000		1.000
member	6′ 8″	●						●	
sub-beam	7′ 4″		1.000		1.000		1.000		1.000
member	14′ 0″	●						●	
sub-beam	7′ 4″		1.000		1.000		1.000		1.000
member	21′ 4″	●						●	
sub-beam	7′ 4″		1.000		1.000		1.000		1.000
member	28′ 8″	●						●	
sub-beam	7′ 4″		1.000		1.000		1.000		1.000
support	36′ 0″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing distance [in]
0″	36′ 0″	40	1	10 51/64

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	8′ 11″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 33x130 (40 (40))

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	Pass
Vertical Shear	8	174.6	575.9	kip	0.303	Pass
Flexure	8	1761.7	2136.4	kip ft	0.825	Pass
Connector Resistance	7	16	40 (40)		-	Pass
Natural Frequency	-	0.00	0.00	Hz	0.000	Not Yet Designed

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 33x130 A992-50 - Critical

Position 17′ 5 1/4″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**51.700**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 33x130 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry} =**0″** ft, in

Required major axis shear strength, V_{ry} =**174.6** kip

Design shear strength =**575.9** kip AISC 360 G2

Ratio =**0.303**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 33x130 A992-50 - Critical

14′ 0″ Max Moment Position - Critical

Distance of M along member =**14′ 0″** ft, in

Required flexural strength, M_x =**1761.7** kip ft

Design flexural strength =**2136.4** kip ft

Ratio =**0.825**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 33x130 A992-50 - Critical

14′ 0″ Max UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**10 51/64** in

Maximum permitted group spacing =**31 1/2** in

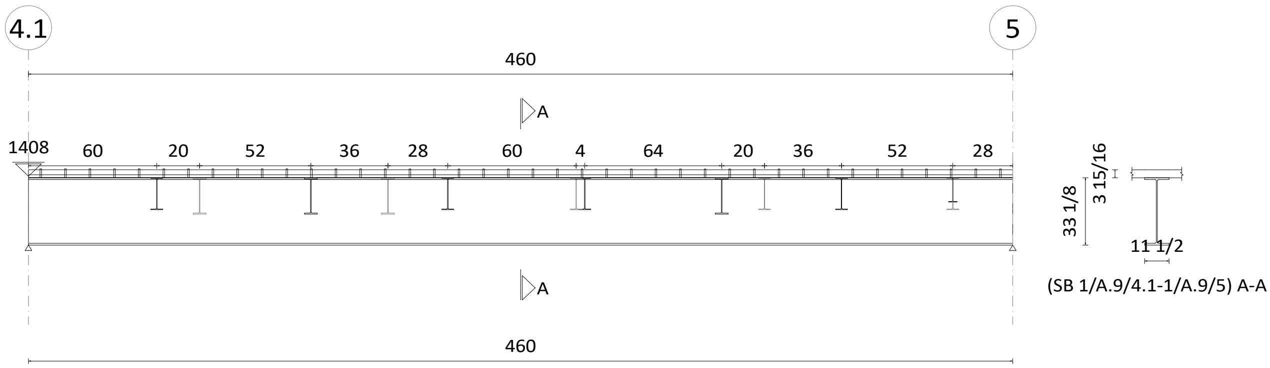
Number of shear connectors over full beam length	40
Number of shear connectors to critical point, N_a	16
Degree of shear connection	=**0.271**
Absolute minimum degree of shear connection	=**0.250**
Optimum amount of shear connection	=**0.500**

Partial shear connection

✔ Pass

Tekla Structural Designer	Project: Creech DRP PH2				Job Ref.	
	Structure: Admin Building				Sheet no.: Page 1/3	
	Calc. by: Allison Valencia	Date: 11/7/2025	Chk'd by	Date	App'd by	Date

SB 1/A.9/4.1-1/A.9/5



St. 1 (1): SB 1/A.9/4.1-1/A.9/5 - 1 (W 33x130 A992-50 (40) [Deformed # 3 @ 8])

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	5′ 0″		1.000		1.000		1.000		1.000
member	5′ 0″	●						●	
sub-beam	1′ 8″		1.000		1.000		1.000		1.000
member	6′ 8″	●						●	
sub-beam	4′ 4″		1.000		1.000		1.000		1.000
member	11′ 0″	●						●	
sub-beam	3′ 0″		1.000		1.000		1.000		1.000
member	14′ 0″	●						●	
sub-beam	2′ 4″		1.000		1.000		1.000		1.000
member	16′ 4″	●						●	
sub-beam	5′ 0″		1.000		1.000		1.000		1.000
member	21′ 4″	●						●	
sub-beam	4″		1.000		1.000		1.000		1.000
member	21′ 8″	●						●	
sub-beam	5′ 4″		1.000		1.000		1.000		1.000
member	27′ 0″	●						●	
sub-beam	1′ 8″		1.000		1.000		1.000		1.000
member	28′ 8″	●						●	
sub-beam	3′ 0″		1.000		1.000		1.000		1.000
member	31′ 8″	●						●	
sub-beam	4′ 4″		1.000		1.000		1.000		1.000
member	36′ 0″	●						●	
sub-beam	2′ 4″		1.000		1.000		1.000		1.000
support	38′ 4″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing distance [in]
0″	38′ 4″	40	1	11 1/2

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	8′ 11″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 33x130 (40 (40))

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	-164.9	575.9	kip	0.286	✔ Pass
Flexure	8	1545.2	2156.2	kip ft	0.717	✔ Pass
Connector Resistance	7	18	40 (40)		-	✔ Pass
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.1	-	in	-	-
Deflection Slab	14	0.3	1.9	in	0.166	✔ Pass
Deflection Dead	14	0.5	1.3	in	0.396	✔ Pass
Deflection Post Composite	14	0.5	1.3	in	0.396	✔ Pass
Deflection Total	14	0.9	1.9	in	0.471	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 33x130 A992-50 - Critical

Position 21′ 4″ - Critical

Web class	Compact	AISC 360 Table B4.1b
h / t_w	=**51.700**	
h / t_w limit for plastic stress distribution	=**90.553**	

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 33x130 A992-50 - Critical

Position 38′ 4″ - Critical

Position of V_{ry}	=**38′ 4″** ft, in	
Required major axis shear strength, V_{ry}	=**-164.9** kip	
Design shear strength	=**575.9** kip	AISC 360 G2
Ratio	=**0.286**	

✔ Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 33x130 A992-50 - Critical

21’ 8” Point Load Position - Critical

Distance of M along member	= **21’ 8”** ft, in
Required flexural strength, M_x	= **1545.2** kip ft
Design flexural strength	= **2156.2** kip ft
Ratio	= **0.717**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 33x130 A992-50 - Critical

21’ 4” Max Moment Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s	= **11 1/2** in
Maximum permitted group spacing	= **31 1/2** in
Number of shear connectors over full beam length	40
Number of shear connectors to critical point, N_a	18
Degree of shear connection	= **0.305**
Absolute minimum degree of shear connection	= **0.250**
Optimum amount of shear connection	= **0.500**

Partial shear connection

Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

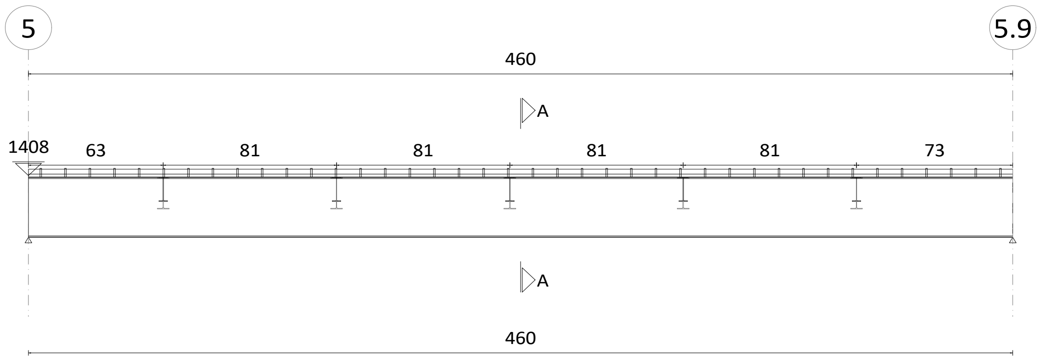
Span 1 W 33x130 A992-50 - Critical

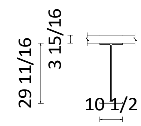
Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	= **13893.6** in^4
Long term inertia, $I_{\alpha L}$	= **10370.7** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	= **10678.3** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	= **8732.2** in^4
Position Total load deflection	= **19’ 2 5/8”** ft, in
Deflection with full shear connection n, δ	= **0.8** in
Deflection of steel beam alone, δ_s	= **1.0** in
Minimum degree of shear connection	= **30.52** %
Deflection with partial shear connection, δ	= **0.9** in
Span over limit	= **1.9** in
Design limit	= **1.9** in
Utilization Ratio	= **0.471**

Pass

SB 1/A.9/5-1/A.9/5.9





(SB 1/A.9/5-1/A.9/5.9) A-A

St. 1 (1): SB 1/A.9/5-1/A.9/5.9 - 1 (W 30x99 A992-50 (40) [Deformed # 3 @ 8])

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	5′ 3″		1.000		1.000		1.000		1.000
member	5′ 3″	●						●	
sub-beam	6′ 9″		1.000		1.000		1.000		1.000
member	12′ 0″	●						●	
sub-beam	6′ 9″		1.000		1.000		1.000		1.000
member	18′ 9″	●						●	
sub-beam	6′ 9″		1.000		1.000		1.000		1.000
member	25′ 6″	●						●	
sub-beam	6′ 9″		1.000		1.000		1.000		1.000
member	32′ 3″	●						●	
sub-beam	6′ 1″		1.000		1.000		1.000		1.000
support	38′ 4″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing distance [in]
0″	38′ 4″	40	1	11 1/2

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	10′ 1″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement

Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi

Reinforcement for crack control or fire requirements

Type	Area	Yield strength
6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 30x99 (40 (40))

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	Pass
Vertical Shear	8	119.4	463.3	kip	0.258	Pass
Flexure	8	1271.5	1574.9	kip ft	0.807	Pass
Connector Resistance	7	20	40 (40)		-	Pass
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.1	-	in	-	-
Deflection Slab	14	0.4	1.9	in	0.230	Pass
Deflection Dead	14	0.6	1.3	in	0.505	Pass
Deflection Post Composite	14	0.6	1.3	in	0.505	Pass
Deflection Total	14	1.2	1.9	in	0.609	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 30x99 A992-50 - Critical

Position 18′ 9″ - Critical

Web class	Compact	AISC 360 Table B4.1b
h / t_w	=**51.900**	
h / t_w limit for plastic stress distribution	=**90.553**	

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 30x99 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry}	=**0″** ft, in	
Required major axis shear strength, V_{ry}	=**119.4** kip	
Design shear strength	=**463.3** kip	AISC 360 G2
Ratio	=**0.258**	

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 30x99 A992-50 - Critical

18′ 9″ Max Moment Position - Critical

Distance of M along member	=**18′ 9″** ft, in
Required flexural strength, M_x	=**1271.5** kip ft
Design flexural strength	=**1574.9** kip ft
Ratio	=**0.807**

Pass

Connector Resistance

3D Building Analysis - Critical

7 LRFD$_1$-1.4D - Critical

Span 1 W 30x99 A992-50 - Critical

18’ 9” Max Moment and UR Position - Critical

Layout and strength of shear connectors	
Maximum group spacing, s	=**11 1/2** in
Maximum permitted group spacing	=**31 1/2** in
Number of shear connectors over full beam length	40
Number of shear connectors to critical point, N_a	20
Degree of shear connection	=**0.300**
Absolute minimum degree of shear connection	=**0.250**
Optimum amount of shear connection	=**0.500**
Partial shear connection	

✔ Pass

Deflection

3D Building Analysis - Critical

14 (Operating) LRFD$_{11.1}$-1.2D+L+0.15S+E - Critical

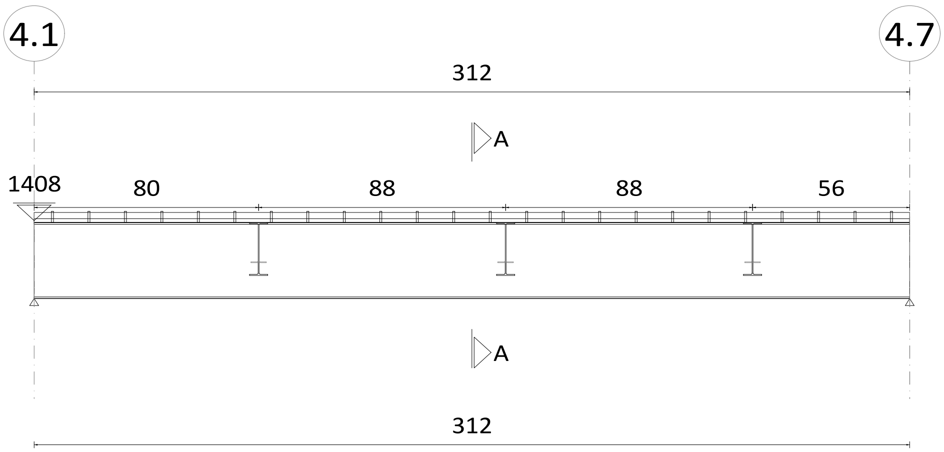
Span 1 W 30x99 A992-50 - Critical

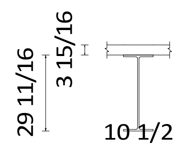
Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**9359.5** in^4
Long term inertia, $I_{\alpha L}$	=**7025.6** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**6930.1** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**5652.2** in^4
Position Total load deflection	=**19’ 1/4”** ft, in
Deflection with full shear connection n, δ	=**1.0** in
Deflection of steel beam alone, δ_s	=**1.4** in
Minimum degree of shear connection	=**29.98** %
Deflection with partial shear connection, δ	=**1.2** in
Span over limit	=**1.9** in
Design limit	=**1.9** in
Utilization Ratio	=**0.609**

✔ Pass

SB 1/D.4/4.1-1/D.4/4.7





(SB 1/D.4/4.1-1/D.4/4.7) A-A

St. 1 (1): SB 1/D.4/4.1-1/D.4/4.7 - 1
W 30x99 A992-50 (24) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	6′ 8″		1.000		1.000		1.000		1.000
member	6′ 8″	●						●	
sub-beam	7′ 4″		1.000		1.000		1.000		1.000
member	14′ 0″	●						●	
sub-beam	7′ 4″		1.000		1.000		1.000		1.000
member	21′ 4″	●						●	
sub-beam	4′ 8″		1.000		1.000		1.000		1.000
support	26′ 0″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing distance [in]
0″	26′ 0″	24	1	13

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	6′ 5″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 30x99 (24 (24))

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	Pass
Vertical Shear	8	-136.4	463.3	kip	0.294	Pass
Flexure	8	1041.0	1416.6	kip ft	0.735	Pass
Connector Resistance	7	11	24 (24)		-	Pass
Natural Frequency	-	0.00	0.00	Hz	0.000	Not Yet Designed

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 30x99 A992-50 - Critical

Position 14′ 0″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**51.900**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 30x99 A992-50 - Critical

Position 26′ 0″ - Critical

Position of V_{ry} =**26′ 0″** ft, in

Required major axis shear strength, V_{ry} =**-136.4** kip

Design shear strength =**463.3** kip AISC 360 G2

Ratio =**0.294**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 30x99 A992-50 - Critical

14′ 0″ Max Moment Position - Critical

Distance of M along member =**14′ 0″** ft, in

Required flexural strength, M_x =**1041.0** kip ft

Design flexural strength =**1416.6** kip ft

Ratio =**0.735**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 30x99 A992-50 - Critical

14′ 0″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**13** in

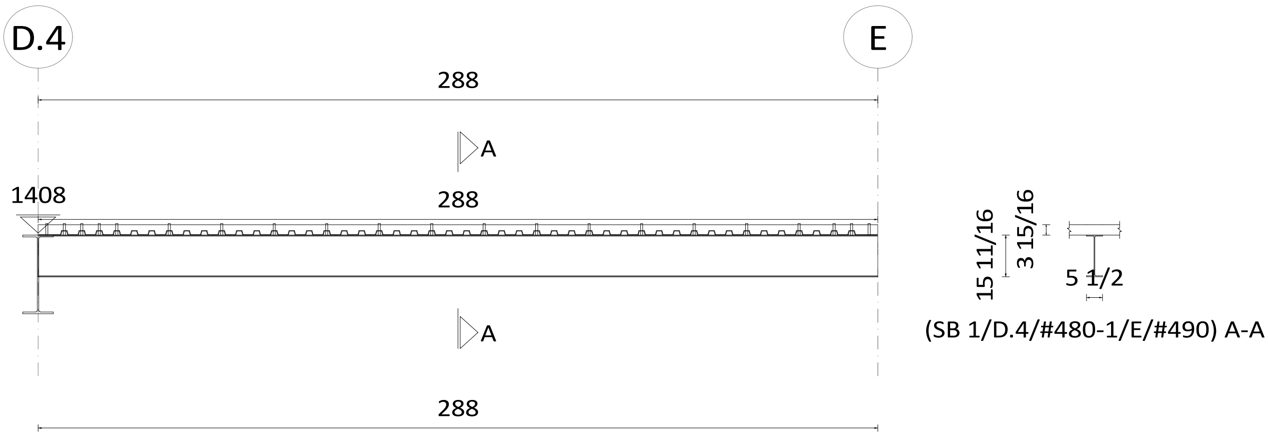
Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length	24
Number of shear connectors to critical point, N_a	11
Degree of shear connection	= **0.259**
Absolute minimum degree of shear connection	= **0.250**
Optimum amount of shear connection	= **0.500**

Partial shear connection

✔ Pass

SB 1/D.4/#480-1/E/#490



(SB 1/D.4/#480-1/E/#490) A-A

St. 1 (1): SB 1/D.4/#480-1/E/#490 - 1
W 16x26 A992-50 (21) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	24′ 0″	●	1.000		1.000		1.000		1.000
support	24′ 0″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	2′ 6″	5	1	1
3′ 6″	22′ 0″	13	1	3
22′ 6″	24′ 0″	3	1	1

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	6′ 0″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 16x26 (21)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	34.1	106.0	kip	0.321	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Flexure	8	204.4	269.1	kip ft	0.759	Pass
Connector Resistance	7	10	21		-	Pass
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.3	1.2	in	0.224	Pass
Deflection Dead	14	0.3	0.8	in	0.359	Pass
Deflection Post Composite	14	0.3	0.8	in	0.359	Pass
Deflection Total	14	0.6	1.2	in	0.484	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

Position 12′ 0″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**56.800**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry} =**0″** ft, in

Required major axis shear strength, V_{ry} =**34.1** kip

Design shear strength =**106.0** kip AISC 360 G2

Ratio =**0.321**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

12′ 0″ Max Moment Position - Critical

Distance of M along member =**12′ 0″** ft, in

Required flexural strength, M_x =**204.4** kip ft

Design flexural strength =**269.1** kip ft

Ratio =**0.759**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

12′ 0″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**18** in

Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length	21
Number of shear connectors to critical point, N_a	10
Degree of shear connection	= **0.449**
Absolute minimum degree of shear connection	= **0.250**
Optimum amount of shear connection	= **0.500**

Partial shear connection

✔ Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

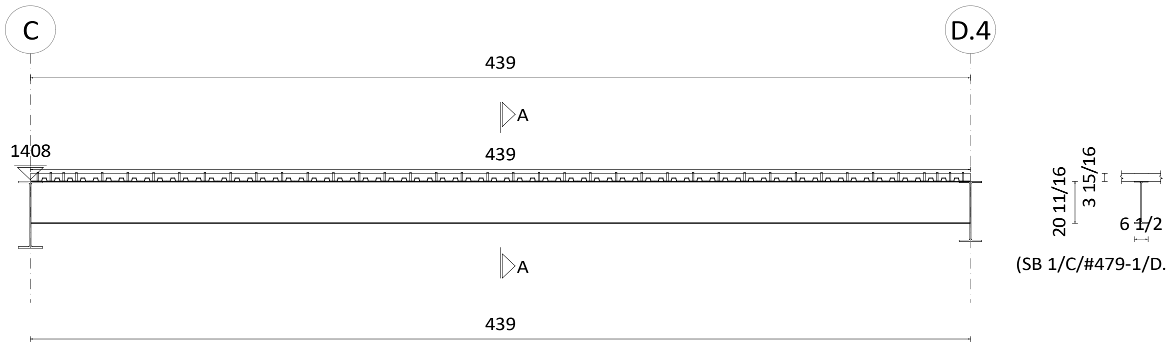
Span 1 W 16x26 A992-50 - Critical

Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	= **955.4** in^4
Long term inertia, $I_{\alpha L}$	= **733.2** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	= **739.3** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	= **590.5** in^4
Position Total load deflection	= **12' 0"** ft, in
Deflection with full shear connection n, δ	= **0.5** in
Deflection of steel beam alone, δ_s	= **0.8** in
Minimum degree of shear connection	= **44.87** %
Deflection with partial shear connection, δ	= **0.6** in
Span over limit	= **1.2** in
Design limit	= **1.2** in
Utilization Ratio	= **0.484**

✔ Pass

SB 1/C/#479-1/D.4/#480



(SB 1/C/#479-1/D.4/#480) A-A

St. 1 (1): SB 1/C/#479-1/D.4/#480 - 1 (W 21x44 A992-50 (40) [Deformed # 3 @ 8])

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	36′ 7″	●	1.000		1.000		1.000		1.000
support	36′ 7″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	2′ 1/2″	4	1	1
2′ 6 1/2″	34′ 1/2″	32	1	2
34′ 6 1/2″	36′ 7″	4	1	1

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	7′ 4″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 21x44 (40)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	-52.3	217.4	kip	0.241	✔ Pass
Flexure	8	478.5	586.2	kip ft	0.816	✔ Pass
Connector Resistance	7	20	40		-	✔ Pass
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.1	-	in	-	-

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Deflection Slab	14	0.5	1.8	in	0.281	✔ Pass
Deflection Dead	14	0.6	1.2	in	0.484	✔ Pass
Deflection Post Composite	14	0.6	1.2	in	0.484	✔ Pass
Deflection Total	14	1.2	1.8	in	0.647	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 21x44 A992-50 - Critical

Position 18’ 3 1/2” - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**53.600**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 21x44 A992-50 - Critical

Position 36’ 7” - Critical

Position of V_{ry} =**36’ 7”** ft, in

Required major axis shear strength, V_{ry} =**-52.3** kip

Design shear strength =**217.4** kip AISC 360 G2

Ratio =**0.241**

✔ Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 21x44 A992-50 - Critical

18’ 3 1/2” Max Moment Position - Critical

Distance of M along member =**18’ 3 1/2”** ft, in

Required flexural strength, M_x =**478.5** kip ft

Design flexural strength =**586.2** kip ft

Ratio =**0.816**

✔ Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 21x44 A992-50 - Critical

18’ 3 1/2” Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**12** in

Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length 40

Number of shear connectors to critical point, N_a 20

Degree of shear connection =**0.530**

Absolute minimum degree of shear connection =**0.250**

Optimum amount of shear connection =**0.500**

Partial shear connection

Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

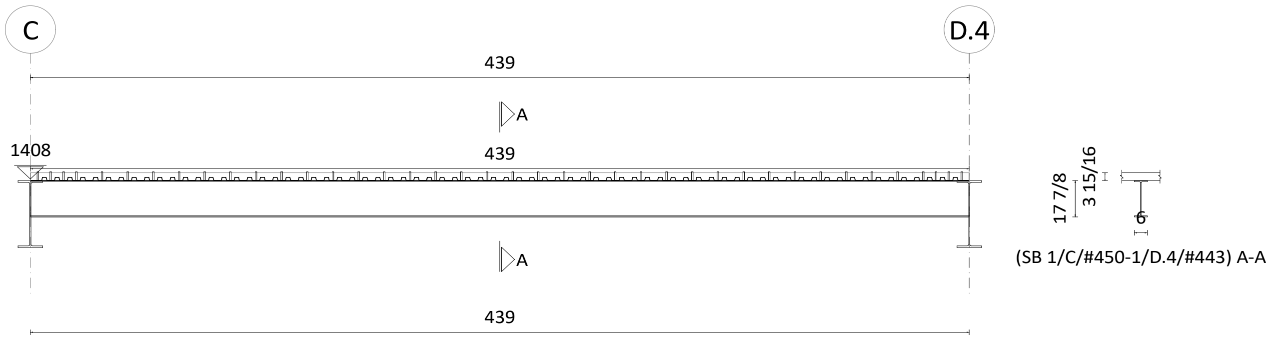
Span 1 W 21x44 A992-50 - Critical

Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**2374.9** in^4
Long term inertia, $I_{\alpha L}$	=**1777.4** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**1958.4** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**1523.4** in^4
Position Total load deflection	=**18′ 3 1/2″** ft, in
Deflection with full shear connection n, δ	=**1.1** in
Deflection of steel beam alone, δ_s	=**1.6** in
Minimum degree of shear connection	=**53.01** %
Deflection with partial shear connection, δ	=**1.2** in
Span over limit	=**1.8** in
Design limit	=**1.8** in
Utilization Ratio	=**0.647**

Pass

SB 1/C/#450-1/D.4/#443



St. 1 (1): SB 1/C/#450-1/D.4/#443 - 1 (W 18x40 A992-50 (40) [Deformed # 3 @ 8])

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	36′ 7″	•	1.000		1.000		1.000		1.000
support	36′ 7″	•		•		•		•	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	2′ 1/2″	4	1	1
2′ 6 1/2″	34′ 1/2″	32	1	2
34′ 6 1/2″	36′ 7″	4	1	1

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	6′ 9″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 18x40 (40)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	-34.3	169.2	kip	0.203	✔ Pass
Flexure	8	313.9	478.0	kip ft	0.657	✔ Pass
Connector Resistance	7	20	40		-	✔ Pass
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.1	-	in	-	-
Deflection Slab	14	0.7	1.8	in	0.356	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Deflection Dead	14	0.7	1.2	in	0.598	Pass
Deflection Post Composite	14	0.7	1.2	in	0.598	Pass
Deflection Total	14	1.5	1.8	in	0.809	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 18x40 A992-50 - Critical

Position 18′ 3 1/2″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**50.900**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 18x40 A992-50 - Critical

Position 36′ 7″ - Critical

Position of V_{ry} =**36′ 7″** ft, in

Required major axis shear strength, V_{ry} =**-34.3** kip

Design shear strength =**169.2** kip AISC 360 G2

Ratio =**0.203**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 18x40 A992-50 - Critical

18′ 3 1/2″ Max Moment Position - Critical

Distance of M along member =**18′ 3 1/2″** ft, in

Required flexural strength, M_x =**313.9** kip ft

Design flexural strength =**478.0** kip ft

Ratio =**0.657**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 18x40 A992-50 - Critical

18′ 3 1/2″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**12** in

Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length 40

Number of shear connectors to critical point, N_a 20

Degree of shear connection =**0.584**

Absolute minimum degree of shear connection =**0.250**

Optimum amount of shear connection =**0.500**

Partial shear connection

Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

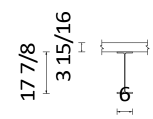
Span 1 W 18x40 A992-50 - Critical

Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**1728.1** in^4
Long term inertia, $I_{\alpha L}$	=**1294.6** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**1465.0** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**1133.6** in^4
Position Total load deflection	=**18' 3 1/2"** ft, in
Deflection with full shear connection n, δ	=**1.4** in
Deflection of steel beam alone, δ_s	=**2.1** in
Minimum degree of shear connection	=**58.41** %
Deflection with partial shear connection, δ	=**1.5** in
Span over limit	=**1.8** in
Design limit	=**1.8** in
Utilization Ratio	=**0.809**

Pass

SB 1/A.9/#484-1/C/#479



(SB 1/A.9/#484-1/C/#479) A-A

St. 1 (1): SB 1/A.9/#484-1/C/#479 - 1
W 18x40 A992-50 (32) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	28′ 7″	●	1.000		1.000		1.000		1.000
support	28′ 7″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	2′ 1/2″	4	1	1
2′ 6 1/2″	26′ 1/2″	24	1	2
26′ 6 1/2″	28′ 7″	4	1	1

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	7′ 1 3/4″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 18x40 (32)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	32.8	169.2	kip	0.194	✔ Pass
Flexure	8	234.7	461.7	kip ft	0.508	✔ Pass
Connector Resistance	7	16	32		-	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.3	1.4	in	0.186	✔ Pass
Deflection Dead	14	0.3	1.0	in	0.326	✔ Pass
Deflection Post Composite	14	0.3	1.0	in	0.326	✔ Pass
Deflection Total	14	0.6	1.4	in	0.430	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 18x40 A992-50 - Critical

Position 14' 3 1/2" - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**50.900**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 18x40 A992-50 - Critical

Position 0" - Critical

Position of V_{ry} =**0"** ft, in

Required major axis shear strength, V_{ry} =**32.8** kip

Design shear strength =**169.2** kip AISC 360 G2

Ratio =**0.194**

✔ Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 18x40 A992-50 - Critical

14' 3 1/2" Max Moment Position - Critical

Distance of M along member =**14' 3 1/2"** ft, in

Required flexural strength, M_x =**234.7** kip ft

Design flexural strength =**461.7** kip ft

Ratio =**0.508**

✔ Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 18x40 A992-50 - Critical

14' 3 1/2" Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**12** in

Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length 32

Number of shear connectors to critical point, N_a 16

Degree of shear connection =**0.467**

Absolute minimum degree of shear connection =**0.250**
Optimum amount of shear connection =**0.500**
Partial shear connection

✔ Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

Span 1 W 18x40 A992-50 - Critical

Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**1748.3** in^4
Long term inertia, $I_{\alpha L}$	=**1317.2** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**1388.7** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**1094.1** in^4
Position Total load deflection	=**14' 3 1/2"** ft, in
Deflection with full shear connection n, δ	=**0.6** in
Deflection of steel beam alone, δ_s	=**0.8** in
Minimum degree of shear connection	=**46.72** %
Deflection with partial shear connection, δ	=**0.6** in
Span over limit	=**1.4** in
Design limit	=**1.4** in
Utilization Ratio	=**0.430**

✔ Pass

SB 1/A.9/#452-1/C/#447

(SB 1/A.9/#452-1/C/#447) A-A

St. 1 (1): SB 1/A.9/#452-1/C/#447 - 1
W 16x26 A992-50 (32) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	•		•		•		•	
sub-beam	28′ 7″	•	1.000		1.000		1.000		1.000
support	28′ 7″	•		•		•		•	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	2′ 1/2″	4	1	1
2′ 6 1/2″	26′ 1/2″	24	1	2
26′ 6 1/2″	28′ 7″	4	1	1

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	6′ 9″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 16x26 (32)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	-26.6	106.0	kip	0.251	✔ Pass
Flexure	8	189.9	296.4	kip ft	0.641	✔ Pass
Connector Resistance	7	16	32		-	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.5	1.4	in	0.346	✔ Pass
Deflection Dead	14	0.5	1.0	in	0.475	✔ Pass
Deflection Post Composite	14	0.5	1.0	in	0.475	✔ Pass
Deflection Total	14	1.0	1.4	in	0.697	✔ Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

Position 14′ 3 1/2″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**56.800**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

Position 28′ 7″ - Critical

Position of V_{ry} =**28′ 7″** ft, in

Required major axis shear strength, V_{ry} =**-26.6** kip

Design shear strength =**106.0** kip AISC 360 G2

Ratio =**0.251**

✔ Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

14′ 3 1/2″ Max Moment Position - Critical

Distance of M along member =**14′ 3 1/2″** ft, in

Required flexural strength, M_x =**189.9** kip ft

Design flexural strength =**296.4** kip ft

Ratio =**0.641**

✔ Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

14′ 3 1/2″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**12** in

Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length 32

Number of shear connectors to critical point, N_a 16

Degree of shear connection =**0.718**

Absolute minimum degree of shear connection =**0.250**

Optimum amount of shear connection =**0.500**

Partial shear connection

Pass

Deflection

3D Building Analysis - Critical

14 (Operating) LRFD$_{11.1}$-1.2D+L+0.15S+E - Critical

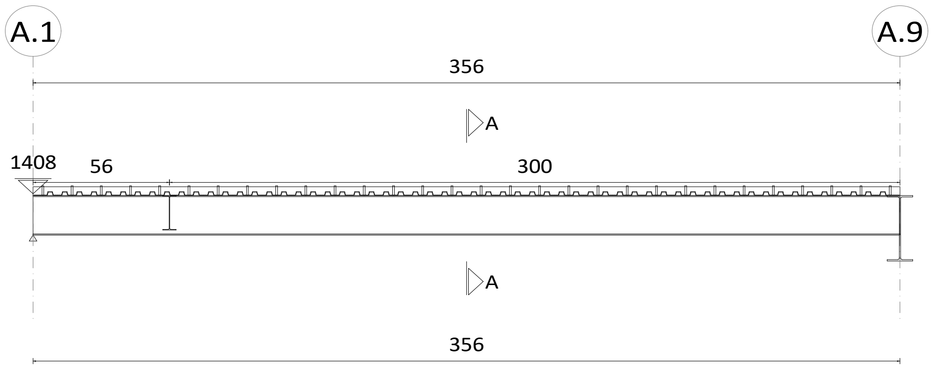
Span 1 W 16x26 A992-50 - Critical

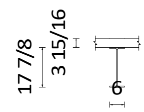
Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**975.2** in^4
Long term inertia, $I_{\alpha L}$	=**759.0** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**872.3** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**689.0** in^4
Position Total load deflection	=**14’ 3 1/2”** ft, in
Deflection with full shear connection n, δ	=**1.0** in
Deflection of steel beam alone, δ_s	=**1.6** in
Minimum degree of shear connection	=**71.79** %
Deflection with partial shear connection, δ	=**1.0** in
Span over limit	=**1.4** in
Design limit	=**1.4** in
Utilization Ratio	=**0.697**

Pass

SB 1/A.1/#487-1/A.9/#1035





(SB 1/A.1/#487-1/A.9/#1035) A-A

St. 1 (1): SB 1/A.1/#487-1/A.9/#1035 - 1
W 18x40 A992-50 (30) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	4′ 8″	●	1.000		1.000		1.000		1.000
member	4′ 8″	●						●	
sub-beam	25′ 0″	●	1.000		1.000		1.000		1.000
support	29′ 8″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	29′ 8″	30	1	2

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	5′ 8 1/8″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 18x40 (30)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	43.5	169.2	kip	0.257	✔ Pass
Flexure	8	234.1	444.0	kip ft	0.527	✔ Pass
Connector Resistance	7	13	30		-	✔ Pass
Natural Frequency	14	0.00	-	Hz	-	Not required

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Deflection Self weight	14	0.1	-	in	-	-
Deflection Slab	14	0.3	1.5	in	0.220	Pass
Deflection Dead	14	0.4	1.0	in	0.420	Pass
Deflection Post Composite	14	0.4	1.0	in	0.420	Pass
Deflection Total	14	0.8	1.5	in	0.540	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 18x40 A992-50 - Critical

Position 12′ 5 5/8″ - Critical

Web class	Compact	AISC 360 Table B4.1b
h / t_w	=**50.900**	
h / t_w limit for plastic stress distribution	=**90.553**	

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 18x40 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry}	=**0″** ft, in	
Required major axis shear strength, V_{ry}	=**43.5** kip	
Design shear strength	=**169.2** kip	AISC 360 G2
Ratio	=**0.257**	

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 18x40 A992-50 - Critical

12′ 5 5/8″ Max Moment Position - Critical

Distance of M along member	=**12′ 5 5/8″** ft, in
Required flexural strength, M_x	=**234.1** kip ft
Design flexural strength	=**444.0** kip ft
Ratio	=**0.527**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 18x40 A992-50 - Critical

12′ 5 5/8″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s	=**12** in
Maximum permitted group spacing	=**31 1/2** in
Number of shear connectors over full beam length	30
Number of shear connectors to critical point, N_a	13
Degree of shear connection	=**0.397**
Absolute minimum degree of shear connection	=**0.250**

Optimum amount of shear connection =**0.500**
Partial shear connection
Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

Span 1 W 18x40 A992-50 - Critical

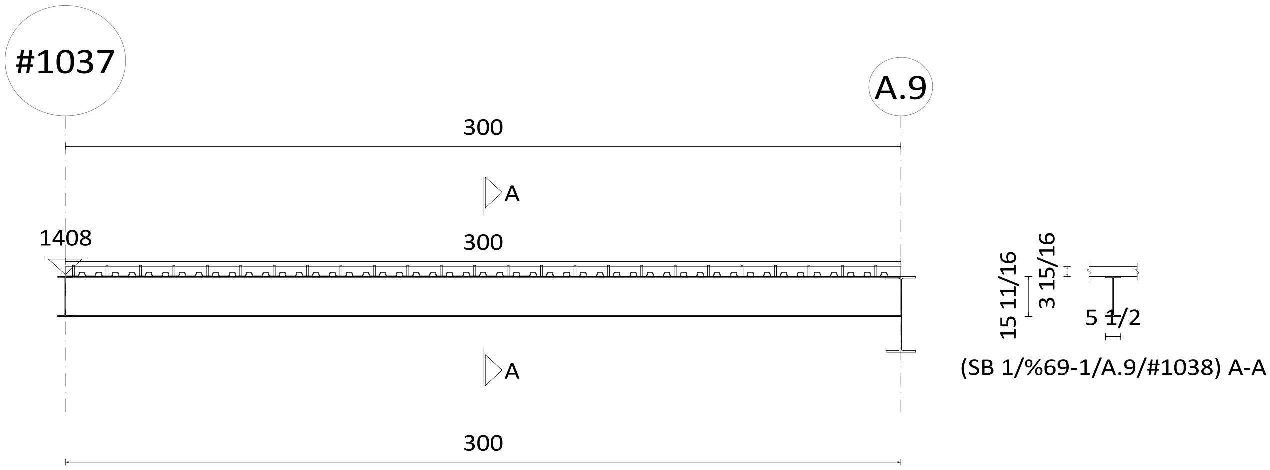
Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**1664.4** in^4
Long term inertia, $I_{\alpha L}$	=**1227.1** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**1274.9** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**999.5** in^4
Position Total load deflection	=**14' 3 1/8"** ft, in
Deflection with full shear connection n, δ	=**0.7** in
Deflection of steel beam alone, δ_s	=**1.1** in
Minimum degree of shear connection	=**39.68** %
Deflection with partial shear connection, δ	=**0.8** in
Span over limit	=**1.5** in
Design limit	=**1.5** in
Utilization Ratio	=**0.540**

Pass

Tekla Structural Designer	Project: Creech DRP PH2				Job Ref.	
	Structure: Admin Building				Sheet no.: Page 1/3	
	Calc. by: Allison Valencia	Date: 11/6/2025	Chk'd by	Date	App'd by	Date

SB 1/%69-1/A.9/#1038



St. 1 (1): SB 1/%69-1/A.9/#1038 - 1
W 16x26 A992-50 (25) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	25′ 0″	●	1.000		1.000		1.000		1.000
support	25′ 0″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	24′ 6″	25	1	2

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	5′ 4″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 16x26 (25)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	18.4	106.0	kip	0.174	✔ Pass
Flexure	8	115.3	277.7	kip ft	0.415	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Connector Resistance	7	12	25		-	Pass
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.2	1.3	in	0.184	Pass
Deflection Dead	14	0.2	0.8	in	0.290	Pass
Deflection Post Composite	14	0.2	0.8	in	0.290	Pass
Deflection Total	14	0.5	1.3	in	0.400	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

Position 12′ 6″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**56.800**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry} =**0″** ft, in

Required major axis shear strength, V_{ry} =**18.4** kip

Design shear strength =**106.0** kip AISC 360 G2

Ratio =**0.174**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

12′ 6″ Max Moment Position - Critical

Distance of M along member =**12′ 6″** ft, in

Required flexural strength, M_x =**115.3** kip ft

Design flexural strength =**277.7** kip ft

Ratio =**0.415**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

12′ 6″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**12** in

Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length 25

Number of shear connectors to critical point, N_a 12

Degree of shear connection =**0.538**

Absolute minimum degree of shear connection =**0.250**

Optimum amount of shear connection =**0.500**

Partial shear connection

✔ Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $\text{LRFD}_{11.1}$-1.2D+L+0.15S+E - Critical

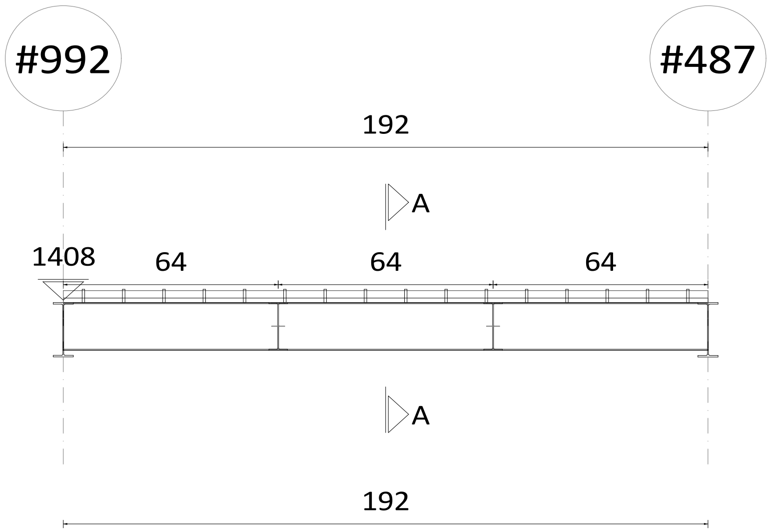
Span 1 W 16x26 A992-50 - Critical

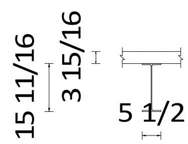
Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**934.6** in^4
Long term inertia, $I_{\alpha L}$	=**707.6** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**765.9** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**599.4** in^4
Position Total load deflection	=**12′ 6″** ft, in
Deflection with full shear connection n, δ	=**0.5** in
Deflection of steel beam alone, δ_s	=**0.7** in
Minimum degree of shear connection	=**53.84** %
Deflection with partial shear connection, δ	=**0.5** in
Span over limit	=**1.3** in
Design limit	=**1.3** in
Utilization Ratio	=**0.400**

✔ Pass

SB 1/#992/#1037-1/#487/#1037





(SB 1/#992/#1037-1/#487/#1037) A-A

St. 1 (1): SB 1/#992/#1037-1/#487/#1037 - 1 W 16x26 A992-50 (16) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	5′ 4″		1.000		1.000		1.000		1.000
member	5′ 4″	●						●	
sub-beam	5′ 4″		1.000		1.000		1.000		1.000
member	10′ 8″	●						●	
sub-beam	5′ 4″		1.000		1.000		1.000		1.000
support	16′ 0″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing distance [in]
0″	16′ 0″	16	1	12

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	4′ 0″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 16x26 (16 (16))

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	Pass
Vertical Shear	8	-23.8	106.0	kip	0.225	Pass
Flexure	8	126.5	232.3	kip ft	0.544	Pass
Connector Resistance	7	5	16 (16)		-	Warning
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.1	0.8	in	0.135	Pass
Deflection Dead	14	0.1	0.5	in	0.246	Pass
Deflection Post Composite	14	0.1	0.5	in	0.246	Pass
Deflection Total	14	0.3	0.8	in	0.320	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

Position 8′ 0″ - Critical

Web class	Compact	AISC 360 Table B4.1b
h / t_w	=**56.800**	
h / t_w limit for plastic stress distribution	=**90.553**	

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

Position 16′ 0″ - Critical

Position of V_{ry}	=**16′ 0″** ft, in	
Required major axis shear strength, V_{ry}	=**-23.8** kip	
Design shear strength	=**106.0** kip	AISC 360 G2
Ratio	=**0.225**	

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

10′ 8″ Point Load Position - Critical

Distance of M along member	=**10′ 8″** ft, in
Required flexural strength, M_x	=**126.5** kip ft
Design flexural strength	=**232.3** kip ft
Ratio	=**0.544**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

10′ 8″ Max UR Position - Critical

Layout and strength of shear connectors	
Maximum group spacing, s	=**12** in
Maximum permitted group spacing	=**31 1/2** in
Number of shear connectors over full beam length	16
Number of shear connectors to critical point, N_a	5
Degree of shear connection	=**0.238**
Absolute minimum degree of shear connection	=**0.250**
Optimum amount of shear connection	=**0.500**
Partial shear connection	

Warning

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

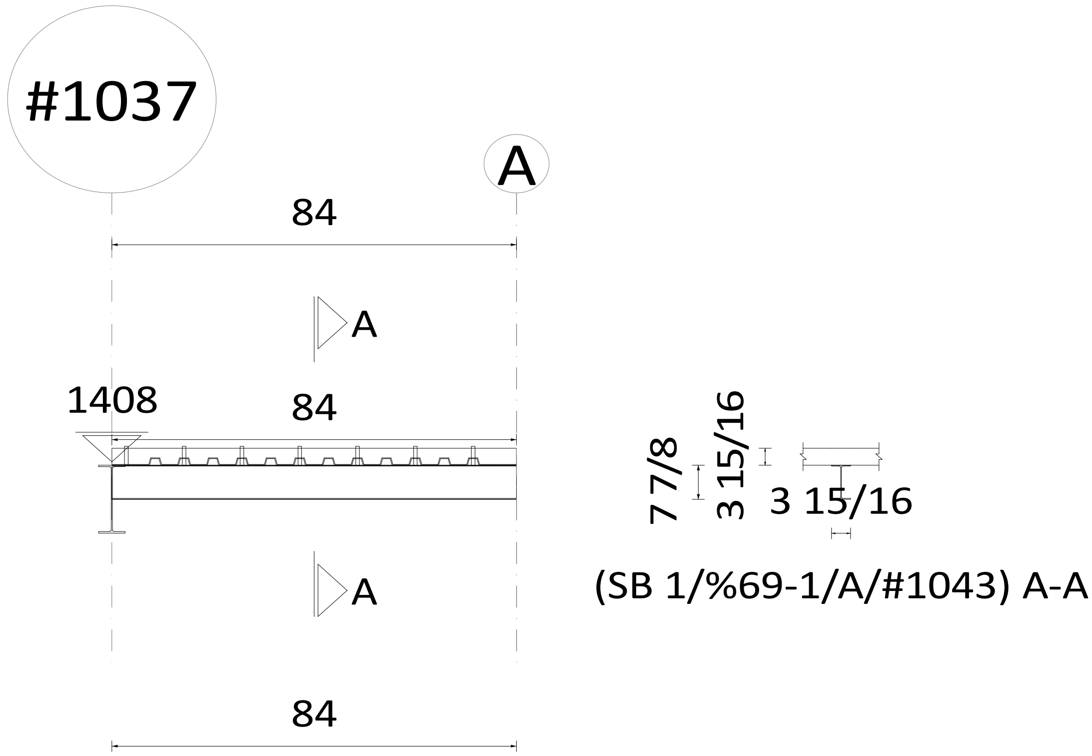
Span 1 W 16x26 A992-50 - Critical

Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**888.2** in^4
Long term inertia, $I_{\alpha L}$	=**666.5** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**663.6** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**526.7** in^4
Position Total load deflection	=**8′ 0″** ft, in
Deflection with full shear connection n, δ	=**0.2** in
Deflection of steel beam alone, δ_s	=**0.3** in
Minimum degree of shear connection	=**38.14** %
Deflection with partial shear connection, δ	=**0.3** in
Span over limit	=**0.8** in
Design limit	=**0.8** in
Utilization Ratio	=**0.320**

Pass

SB 1/%69-1/A/#1043



St. 1 (1): SB 1/%69-1/A/#1043 - 1
W 8x10 A992-50 (7) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	7′ 0″	●	1.000		1.000		1.000		1.000
support	7′ 0″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	6′ 6″	7	1	2

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	1′ 9″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 8x10 (7)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	Pass
Vertical Shear	8	-5.1	40.2	kip	0.127	Pass
Flexure	8	8.9	56.5	kip ft	0.158	Pass
Connector Resistance	7	3	7		-	Warning
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.0	0.4	in	0.042	Pass
Deflection Dead	14	0.0	0.2	in	0.066	Pass
Deflection Post Composite	14	0.0	0.2	in	0.066	Pass
Deflection Total	14	0.0	0.4	in	0.088	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 8x10 A992-50 - Critical

Position 3′ 6″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**40.500**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 8x10 A992-50 - Critical

Position 7′ 0″ - Critical

Position of V_{ry} =**7′ 0″** ft, in

Required major axis shear strength, V_{ry} =**-5.1** kip

Design shear strength =**40.2** kip AISC 360 G2

Ratio =**0.127**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 8x10 A992-50 - Critical

3′ 6″ Max Moment Position - Critical

Distance of M along member =**3′ 6″** ft, in

Required flexural strength, M_x =**8.9** kip ft

Design flexural strength =**56.5** kip ft

Ratio =**0.158**

Pass

Connector Resistance

3D Building Analysis - Critical

7 LRFD$_1$-1.4D - Critical

Span 1 W 8x10 A992-50 - Critical

3′ 6″ Max Moment and UR Position - Critical

Layout and strength of shear connectors	
Maximum group spacing, s	=**12** in
Maximum permitted group spacing	=**31 1/2** in
Number of shear connectors over full beam length	7
Number of shear connectors to critical point, N_a	3
Degree of shear connection	=**0.349**
Absolute minimum degree of shear connection	=**0.250**
Optimum amount of shear connection	=**0.500**
Partial shear connection	

Note: Stud dia. > 2.5 x flange thickness

Warning

Deflection

3D Building Analysis - Critical

14 (Operating) LRFD$_{11.1}$-1.2D+L+0.15S+E - Critical

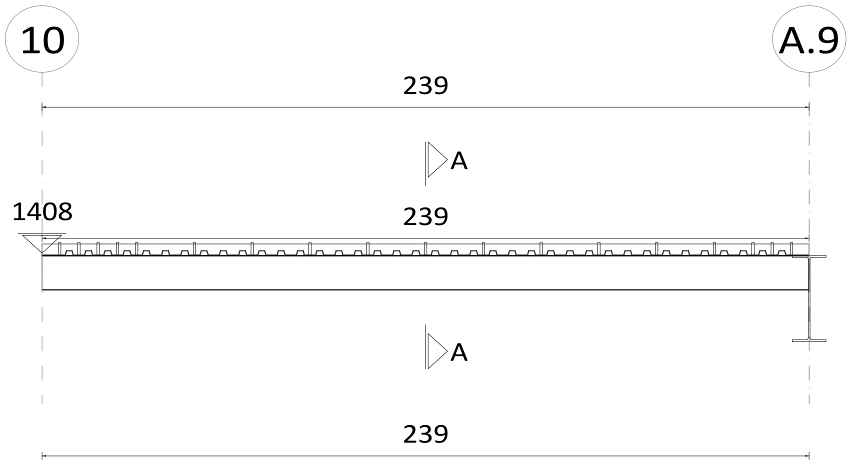
Span 1 W 8x10 A992-50 - Critical

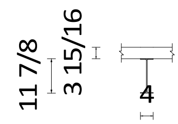
Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**125.3** in^4
Long term inertia, $I_{\alpha L}$	=**88.5** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**86.6** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**64.9** in^4
Position Total load deflection	=**3′ 6″** ft, in
Deflection with full shear connection n, δ	=**0.0** in
Deflection of steel beam alone, δ_s	=**0.0** in
Minimum degree of shear connection	=**34.93** %
Deflection with partial shear connection, δ	=**0.0** in
Span over limit	=**0.4** in
Design limit	=**0.4** in
Utilization Ratio	=**0.088**

Pass

SB 1/10/#457-1/A.9/#456





(SB 1/10/#457-1/A.9/#456) A-A

St. 1 (1): SB 1/10/#457-1/A.9/#456 - 1
W 12x14 A992-50 (18) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	19′ 11″	●	1.000		1.000		1.000		1.000
support	19′ 11″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	2′ 8 1/2″	5	1	1
3′ 8 1/2″	17′ 8 1/2″	10	1	3
18′ 2 1/2″	19′ 11″	3	1	1

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	4′ 11 3/4″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 12x14 (18)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Vertical Shear	8	19.5	64.3	kip	0.303	Pass
Flexure	8	97.1	129.6	kip ft	0.749	Pass
Connector Resistance	7	8	18		-	Warning
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.4	1.0	in	0.422	Pass
Deflection Dead	14	0.3	0.7	in	0.483	Pass
Deflection Post Composite	14	0.3	0.7	in	0.483	Pass
Deflection Total	14	0.8	1.0	in	0.766	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

Position 9′ 11 1/2″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**54.300**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 12x14 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry} =**0″** ft, in

Required major axis shear strength, V_{ry} =**19.5** kip

Design shear strength =**64.3** kip AISC 360 G2

Ratio =**0.303**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 12x14 A992-50 - Critical

9′ 11 1/2″ Max Moment Position - Critical

Distance of M along member =**9′ 11 1/2″** ft, in

Required flexural strength, M_x =**97.1** kip ft

Design flexural strength =**129.6** kip ft

Ratio =**0.749**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 12x14 A992-50 - Critical

9′ 11 1/2″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**18** in

Maximum permitted group spacing	=**31 1/2** in
Number of shear connectors over full beam length	18
Number of shear connectors to critical point, N_a	8
Degree of shear connection	=**0.663**
Absolute minimum degree of shear connection	=**0.250**
Optimum amount of shear connection	=**0.500**

Partial shear connection

Note: Stud dia. > 2.5 x flange thickness

Warning

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

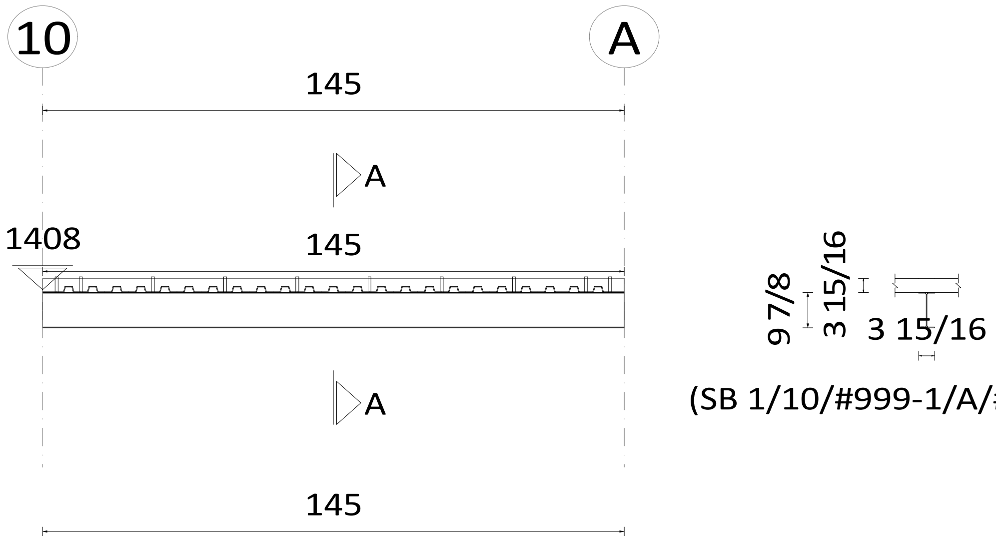
Span 1 W 12x14 A992-50 - Critical

Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**354.7** in^4
Long term inertia, $I_{\alpha L}$	=**280.7** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**305.2** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**245.0** in^4
Position Total load deflection	=**9' 11 1/2"** ft, in
Deflection with full shear connection n, δ	=**0.7** in
Deflection of steel beam alone, δ_s	=**1.3** in
Minimum degree of shear connection	=**66.27** %
Deflection with partial shear connection, δ	=**0.8** in
Span over limit	=**1.0** in
Design limit	=**1.0** in
Utilization Ratio	=**0.766**

Pass

SB 1/10/#999-1/A/#1000



(SB 1/10/#999-1/A/#1000) A-A

St. 1 (1): SB 1/10/#999-1/A/#1000 - 1
W 10x12 A992-50 (10) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	12′ 1″	●	1.000		1.000		1.000		1.000
support	12′ 1″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	1′ 1/2″	2	1	1
2′ 1/2″	10′ 1/2″	6	1	3
11′ 1/2″	12′ 1″	2	1	1

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	3′ 1/4″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 10x12 (10)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	Pass
Vertical Shear	8	8.5	56.3	kip	0.151	Pass
Flexure	8	25.6	87.9	kip ft	0.291	Pass
Connector Resistance	7	5	10		-	Warning
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.0	-	in	-	-
Deflection Slab	14	0.1	0.6	in	0.113	Pass
Deflection Dead	14	0.1	0.4	in	0.150	Pass
Deflection Post Composite	14	0.1	0.4	in	0.150	Pass
Deflection Total	14	0.1	0.6	in	0.220	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 10x12 A992-50 - Critical

Position 6′ 1/2″ - Critical

Web class	Compact	AISC 360 Table B4.1b
h / t_w	=**46.600**	
h / t_w limit for plastic stress distribution	=**90.553**	

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 10x12 A992-50 - Critical

Position 0″ - Critical

Position of V_{ry}	=**0″** ft, in	
Required major axis shear strength, V_{ry}	=**8.5** kip	
Design shear strength	=**56.3** kip	AISC 360 G2
Ratio	=**0.151**	

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 10x12 A992-50 - Critical

6′ 1/2″ Max Moment Position - Critical

Distance of M along member	=**6′ 1/2″** ft, in
Required flexural strength, M_x	=**25.6** kip ft
Design flexural strength	=**87.9** kip ft
Ratio	=**0.291**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 10x12 A992-50 - Critical

6′ 1/2″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s	=**18** in
Maximum permitted group spacing	=**31 1/2** in
Number of shear connectors over full beam length	10
Number of shear connectors to critical point, N_a	5
Degree of shear connection	=**0.487**
Absolute minimum degree of shear connection	=**0.250**
Optimum amount of shear connection	=**0.500**

Partial shear connection

Note: Stud dia. > 2.5 x flange thickness

Warning

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

Span 1 W 10x12 A992-50 - Critical

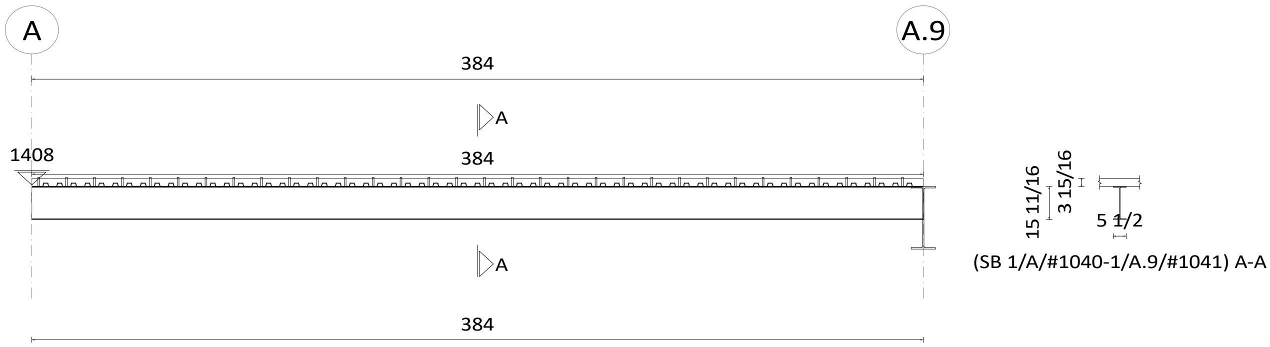
Deflection Total - Critical

Short term inertia, $I_{\alpha S}$	=**218.3** in^4
Long term inertia, $I_{\alpha L}$	=**163.9** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**168.6** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**130.6** in^4
Position Total load deflection	=**6′ 1/2″** ft, in
Deflection with full shear connection n, δ	=**0.1** in
Deflection of steel beam alone, δ_s	=**0.2** in
Minimum degree of shear connection	=**48.67** %
Deflection with partial shear connection, δ	=**0.1** in
Span over limit	=**0.6** in
Design limit	=**0.6** in
Utilization Ratio	=**0.220**

Pass

Tekla Structural Designer	Project: Creech DRP PH2			Job Ref.	
	Structure: Admin Building			Sheet no.: Page 1/3	
	Calc. by: Allison Valencia	Date: 11/6/2025	Chk'd by:	Date:	App'd by:	Date:

SB 1/A/#1040-1/A.9/#1041



St. 1 (1): SB 1/A/#1040-1/A.9/#1041 - 1
W 16x26 A992-50 (32) [Deformed # 3 @ 8]

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	32′ 0″	●	1.000		1.000		1.000		1.000
support	32′ 0″	●		●		●		●	

Connectors\Layout

Name	Diameter	As welded height	Specified ultimate tensile strength
AISC 3/4 x 4 3/8	3/4	4	65.00

Distance end 1 [ft, in]	Distance end 2 [ft, in]	Number of connectors in length	Number of connectors in group	Group spacing × rib
0″	31′ 6″	32	1	2

Decking

Manufacturer	Name	Gauge	Yield strength
USD	B-LOK 33ksi 30"	1/32 in	33.00 ksi

Slab

Overall depth	Effective Width	Concrete type	Minimum compressive strength, f_c'
3 15/16 in	6′ 1/8″ ft, in	Normal	4.00 ksi

Reinforcement

Transverse reinforcement			Reinforcement for crack control or fire requirements		
Type	Area	Yield strength	Type	Area	Yield strength
Deformed # 3 @ 8	0.17 in^2/ft	60.00 ksi	6x6 W1.4xW1.4	0.03 in^2/ft	65.00 ksi

Static Design Summary

Summary W 16x26 (32)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Construction Stage	-	-	-	-	-1.000	Not required
Classification	7	Compact	-	-	-	✔ Pass
Vertical Shear	8	-26.5	106.0	kip	0.250	✔ Pass
Flexure	8	212.0	295.1	kip ft	0.718	✔ Pass
Connector Resistance	7	16	32		-	✔ Pass
Natural Frequency	14	0.00	-	Hz	-	Not required
Deflection Self weight	14	0.1	-	in	-	-
Deflection Slab	14	0.7	1.6	in	0.430	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Deflection Dead	14	0.6	1.1	in	0.606	Pass
Deflection Post Composite	14	0.6	1.1	in	0.606	Pass
Deflection Total	14	1.4	1.6	in	0.882	Pass

Regional code: United States (ACI/AISC), design code: AISC 360/341 LRFD (2016)

Static Design Calculations

Classification

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

Position 16′ 0″ - Critical

Web class Compact AISC 360 Table B4.1b

h / t_w =**56.800**

h / t_w limit for plastic stress distribution =**90.553**

Vertical Shear

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

Position 32′ 0″ - Critical

Position of V_{ry} =**32′ 0″** ft, in

Required major axis shear strength, V_{ry} =**-26.5** kip

Design shear strength =**106.0** kip AISC 360 G2

Ratio =**0.250**

Pass

Flexure

3D Building Analysis - Critical

8 $LRFD_2$-1.2D+1.6L - Critical

Span 1 W 16x26 A992-50 - Critical

16′ 0″ Max Moment Position - Critical

Distance of M along member =**16′ 0″** ft, in

Required flexural strength, M_x =**212.0** kip ft

Design flexural strength =**295.1** kip ft

Ratio =**0.718**

Pass

Connector Resistance

3D Building Analysis - Critical

7 $LRFD_1$-1.4D - Critical

Span 1 W 16x26 A992-50 - Critical

16′ 0″ Max Moment and UR Position - Critical

Layout and strength of shear connectors

Maximum group spacing, s =**12** in

Maximum permitted group spacing =**31 1/2** in

Number of shear connectors over full beam length 32

Number of shear connectors to critical point, N_a 16

Degree of shear connection =**0.718**

Absolute minimum degree of shear connection =**0.250**

Optimum amount of shear connection =**0.500**

Partial shear connection

Pass

Deflection

3D Building Analysis - Critical

14 (Operating) $LRFD_{11.1}$-1.2D+L+0.15S+E - Critical

Span 1 W 16x26 A992-50 - Critical

Deflection Total - Critical

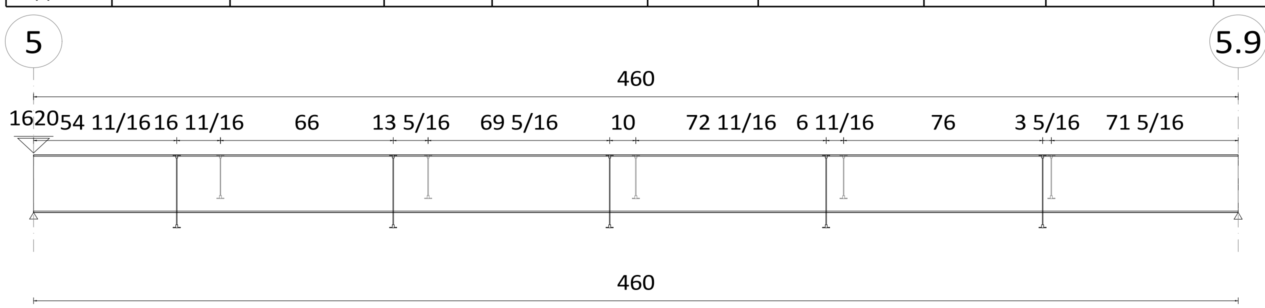
Short term inertia, $I_{\alpha S}$	= **955.7** in^4
Long term inertia, $I_{\alpha L}$	= **733.6** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	= **855.7** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	= **667.6** in^4
Position Total load deflection	= **16' 0"** ft, in
Deflection with full shear connection n, δ	= **1.4** in
Deflection of steel beam alone, δ_s	= **2.2** in
Minimum degree of shear connection	= **71.79** %
Deflection with partial shear connection, δ	= **1.4** in
Span over limit	= **1.6** in
Design limit	= **1.6** in
Utilization Ratio	= **0.882**

Pass

SB 2/D.4/5-2/D.4/5.9

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	4′ 6 5/8″	●	1.000		1.000		1.000		1.000
member	4′ 6 5/8″	●						●	
sub-beam	1′ 4 5/8″	●	1.000		1.000		1.000		1.000
member	5′ 11 3/8″	●						●	
sub-beam	5′ 6″	●	1.000		1.000		1.000		1.000
member	11′ 5 3/8″	●						●	
sub-beam	1′ 1 3/8″	●	1.000		1.000		1.000		1.000
member	12′ 6 5/8″	●						●	
sub-beam	5′ 9 3/8″	●	1.000		1.000		1.000		1.000
member	18′ 4″	●						●	
sub-beam	10″	●	1.000		1.000		1.000		1.000
member	19′ 2″	●						●	
sub-beam	6′ 5/8″	●	1.000		1.000		1.000		1.000
member	25′ 2 5/8″	●						●	
sub-beam	6 5/8″	●	1.000		1.000		1.000		1.000
member	25′ 9 3/8″	●						●	
sub-beam	6′ 4″	●	1.000		1.000		1.000		1.000
member	32′ 1 3/8″	●						●	
sub-beam	3 3/8″	●	1.000		1.000		1.000		1.000
member	32′ 4 5/8″	●						●	
sub-beam	5′ 11 3/8″	●	1.000		1.000		1.000		1.000
support	38′ 4″	●		●		●		●	



St. 2 (2): SB 2/D.4/5-2/D.4/5.9 - 1 (W 24x68 A992-50)

Loading

Loadcase	Source	Direction	In Proj.	Span/Stack	Type	Q_1 [klf]	Length [ft, in]
1 Self weight - excluding slabs	User	Global Z		1	Full UDL	0.07	38′ 4″

Live Load Reduction

Loadcase	Imposed Load Reductions	Span	Reduction Factor	Tributary Area [ft^2]	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Static Design Summary

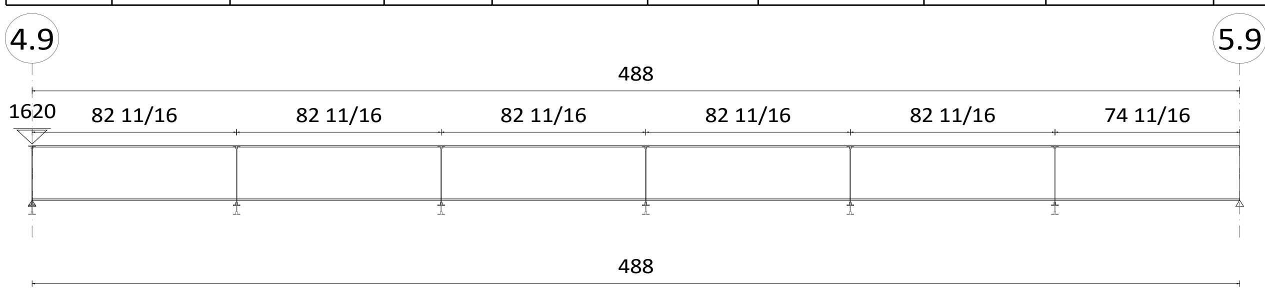
Summary W 24x68

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	15	Slender	-	-	-	Pass
Flexural Classification	7	Compact	-	-	-	Pass
Shear Major	11	34.7	295.1	kip	0.118	Pass
Shear Minor	17	-5.9	283.4	kip	0.021	Pass
Flexure Major	11	371.1	663.7	kip ft	0.559	Pass
Flexure Minor	17	-15.6	91.9	kip ft	0.169	Pass
Axial Tension	14	-3.4	904.5	kip	0.004	Pass
Axial Compression	19	3.4	280.6	kip	0.012	Pass
Combined Forces	16	-	-	-	0.506	Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	14	0.1	-	in	-	-
Deflection Dead	14	0.8	1.3	in	0.591	Pass
Deflection Total	17	0.9	1.9	in	0.450	Pass

SB 2/C/4.9-2/C/5.9

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	6′ 10 5/8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	13′ 9 3/8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	20′ 8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	27′ 6 5/8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	34′ 5 3/8″	●						●	
sub-beam	6′ 2 5/8″	●	1.000		1.000		1.000		1.000
support	40′ 8″	●		●		●		●	



St. 2 (2): SB 2/C/4.9-2/C/5.9 - 1 (W 24x68 A992-50)

Loading

Loadcase	Source	Direction	In Proj.	Span/Stack	Type	Q_1 [klf]	Length [ft, in]
1 Self weight - excluding slabs	User	Global Z		1	Full UDL	0.07	40′ 8″

Live Load Reduction

Loadcase	Imposed Load Reductions	Span	Reduction Factor	TributaryArea [ft^2]	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Static Design Summary

Summary W 24x68

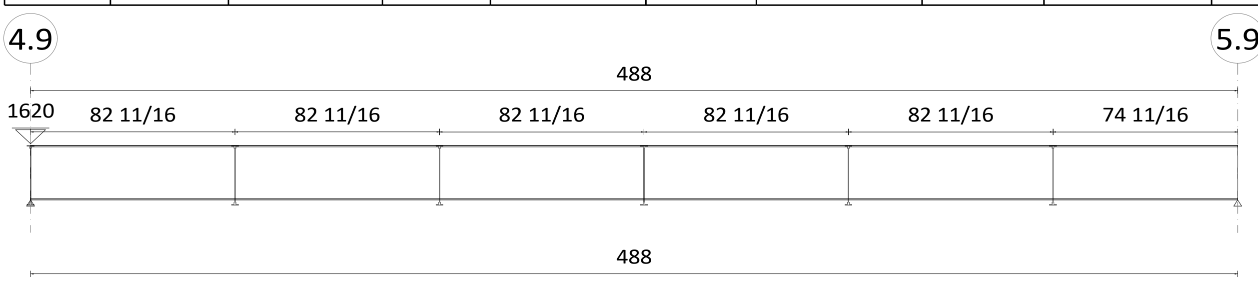
Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	14	Slender	-	-	-	✔ Pass
Flexural Classification	7	Compact	-	-	-	✔ Pass
Shear Major	11	-37.9	295.1	kip	0.129	✔ Pass
Shear Minor	17	3.0	283.4	kip	0.011	✔ Pass
Flexure Major	11	447.4	663.7	kip ft	0.674	✔ Pass
Flexure Minor	17	-18.5	91.9	kip ft	0.202	✔ Pass
Axial Tension	15	-1.9	904.5	kip	0.002	✔ Pass
Axial Compression	18	1.9	268.7	kip	0.007	✔ Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Combined Forces	17	-	-	-	0.606	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	14	0.1	-	in	-	-
Deflection Dead	14	1.0	1.4	in	0.750	✔ Pass
Deflection Total	15	1.2	2.0	in	0.570	✔ Pass

SB 2/A.9/4.9-2/A.9/5.9

Restraints

Source	Distance / Length [ft, in]	LTB Top / Sub -Beam	LTB Top Factor	LTB Btm / Sub -Beam	LTB Btm Factor	Strut Major / Sub-Beam	Strut Major Factor	Strut Minor / Sub-Beam	Strut Minor Factor
support	0″	●		●		●		●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	6′ 10 5/8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	13′ 9 3/8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	20′ 8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	27′ 6 5/8″	●						●	
sub-beam	6′ 10 5/8″	●	1.000		1.000		1.000		1.000
member	34′ 5 3/8″	●						●	
sub-beam	6′ 2 5/8″	●	1.000		1.000		1.000		1.000
support	40′ 8″	●		●		●		●	



St. 2 (2): SB 2/A.9/4.9-2/A.9/5.9 - 1 (W 24x68 A992-50)

Loading

Loadcase	Source	Direction	In Proj.	Span/Stack	Type	Q_1 [klf]	Length [ft, in]
1 Self weight - excluding slabs	User	Global Z		1	Full UDL	0.07	40′ 8″

Live Load Reduction

Loadcase	Imposed Load Reductions	Span	Reduction Factor	Tributary Area [ft^2]	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Static Design Summary

Summary W 24x68

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	14	Slender	-	-	-	Pass
Flexural Classification	7	Compact	-	-	-	Pass
Shear Major	11	32.9	295.1	kip	0.111	Pass
Shear Minor	17	2.4	283.4	kip	0.008	Pass
Flexure Major	11	387.4	663.7	kip ft	0.584	Pass
Flexure Minor	17	-14.7	91.9	kip ft	0.160	Pass
Axial Tension	19	-3.5	904.5	kip	0.004	Pass
Axial Compression	14	3.5	268.7	kip	0.013	Pass

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Combined Forces	17	-	-	-	0.512	✔ Pass
Torsion	No Significant Forces			-	-	Not required
Deflection Self weight	14	0.1	-	in	-	-
Deflection Dead	14	0.9	1.4	in	0.644	✔ Pass
Deflection Total	15	1.0	2.0	in	0.494	✔ Pass

Tekla Structural Designer	Project Creech DRP PH2				Job Ref.	
	Structure Admin Building				Sheet no. Page 1/1	
	Calc. by Allison Valencia	Date 11/6/2025	Chk'd by	Date	App'd by	Date

Steel

Column Design

Column Design Summary

Static

Member Reference	Group Ref.	Stack	Section	Grade	Length [ft, in]	Utilization	Status
SC D.4/4.1	SCR4	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.474	Warning
SC D.4/4.1	SCR4	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.265	Warning
SC D.4/5	SCR1	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.697	Warning
SC D.4/5	SCR1	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.370	Warning
SC C/4.9	SCR3	1	HSS 10x10x1/2	A500B-46	17′ 4″	0.861	Warning
SC C/4.9	SCR3	2	HSS 10x10x1/2	A500B-46	17′ 8″	0.317	Pass
SC D.4/5.9	SCR1	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.672	Warning
SC D.4/5.9	SCR1	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.355	Warning
SC C/4.1	SCR1	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.594	Warning
SC C/4.1	SCR1	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.505	Warning
SC C/5.9	SCR1	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.861	Warning
SC C/5.9	SCR1	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.531	Warning
SC A.9/4.1	SCR1	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.692	Warning
SC A.9/4.1	SCR1	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.414	Warning
SC A.9/4.9	SCR3	1	HSS 10x10x1/2	A500B-46	17′ 4″	0.690	Pass
SC A.9/4.9	SCR3	2	HSS 10x10x1/2	A500B-46	17′ 8″	0.250	Pass
SC A.9/5.9	SCR1	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.630	Warning
SC A.9/5.9	SCR1	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.382	Warning
SC A.1/P	SCR2	1	HSS 6x6x1/4	A992-50	17′ 4″	0.739	Pass
SC D.4/4.7	SCR1	1	HSS 8x8x1/2	A500B-46	17′ 4″	0.638	Warning
SC D.4/4.7	SCR1	2	HSS 8x8x1/2	A500B-46	17′ 8″	0.305	Warning
SC A.1/#487	SCR5	1	HSS 6x6x1/4	A992-50	17′ 4″	0.751	Pass

NOTE: Columns are braced by composite deck and metal roof deck in both axis.

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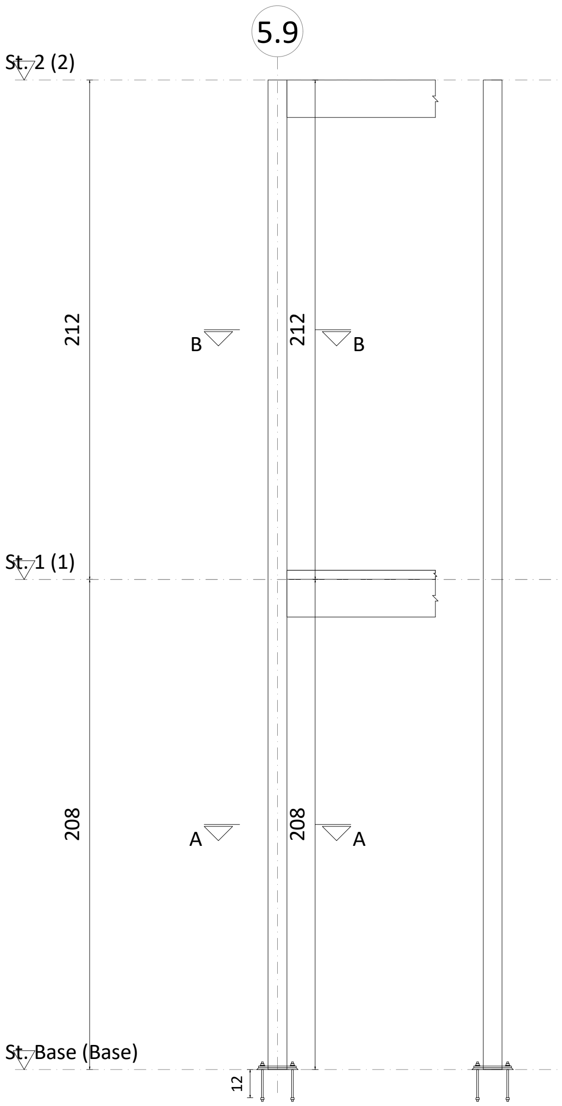
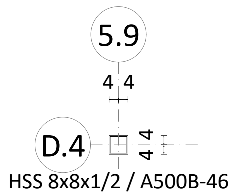
	Structure: Admin Building				Sheet no.: Page 1/2	
	Calc. by: Allison Valencia	Date: 11/6/2025	Chk'd by	Date	App'd by	Date

SC D.4/5.9

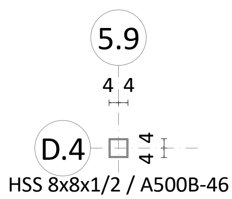


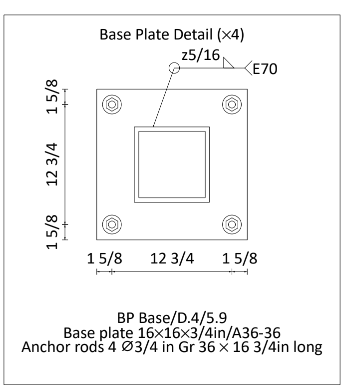
SC D.4/5.9

SC D.4/5.9 B-B



SC D.4/5.9 A-A





Loading

Loadcase	Source	Direction	In Proj.	Span/Stack	Type	Q_1 [klf]	…ngt… [ft, in]
1 Self weight - excluding slabs	User	Global Z		1	Full UDL	0.05	17′ 4″
1 Self weight - excluding slabs	User	Global Z		2	Full UDL	0.05	17′ 8″

Live Load Reduction

Loadcase	Imposed Load	Stack	Reduction Factor	Tributary Area	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack	Face A factor	Face C restrained/ Sub-stack	Face C factor
3	floor	35′ 0″	Yes		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack	…aj… …ct…	Minor restrained / Sub-stack	…in… …ct…
3	floor	35′ 0″	No		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

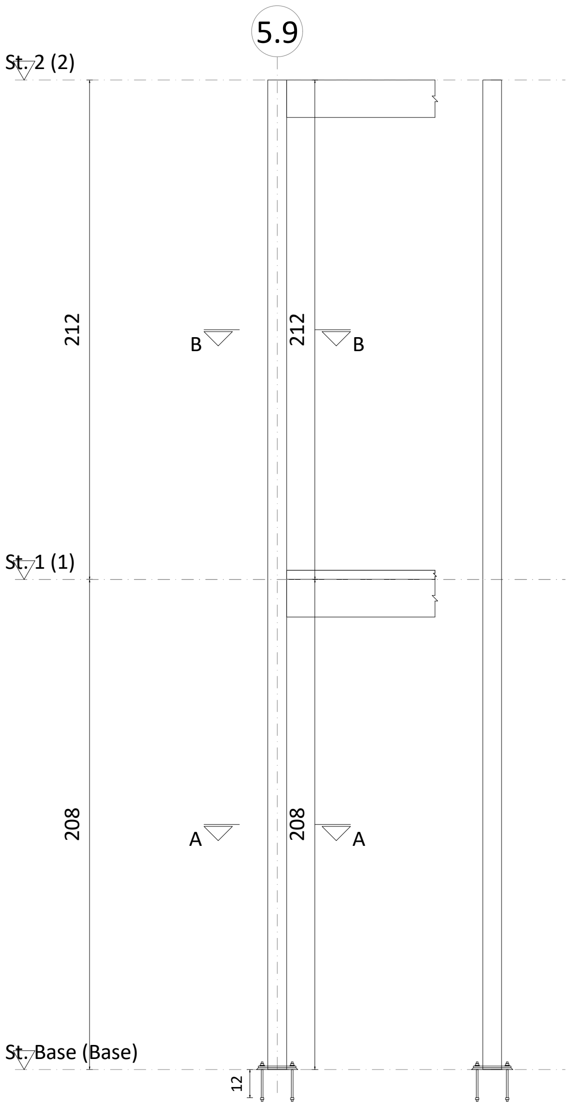
Static Design Summary

Summary HSS 8x8x1/2 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	7	Non-slender	-	-	-	✔ Pass
Flexural Classification	7	Compact	-	-	-	✔ Pass
Shear Major	16	0.5	152.6	kip	0.003	✔ Pass
Shear Minor	15	0.2	152.6	kip	0.002	✔ Pass
Flexure Major	16	-9.1	129.4	kip ft	0.070	✔ Pass
Flexure Minor	8	38.6	129.4	kip ft	0.299	✔ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	9	165.6	407.8	kip	0.406	✔ Pass
Combined Forces	9	-	-	-	0.672	✔ Pass
Design Note	Major and/or Minor Axes Strut Restrained					⚠ Warning

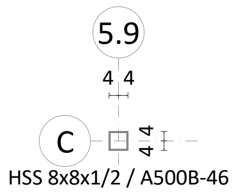
Tekla Structural Designer	Project: Creech DRP PH2			Job Ref.	
	Structure: Admin Building			Sheet no.: Page 1/2	
	Calc. by: Allison Valencia	Date: 11/6/2025	Chk'd by:	Date:	App'd by:	Date:

SC C/5.9

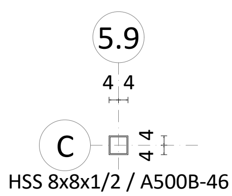


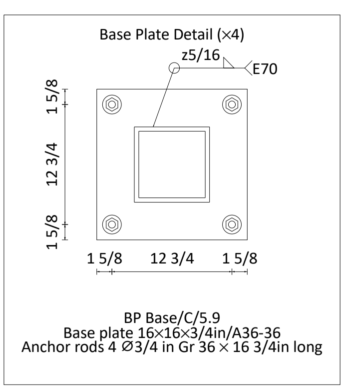
SC C/5.9

SC C/5.9 B-B



SC C/5.9 A-A





Loading

Loadcase	Source	Direction	In Proj.	Span/Stack	Type	Q_1 [klf]	...ngt... [ft, in]
1 Self weight - excluding slabs	User	Global Z		1	Full UDL	0.05	17′ 4″
1 Self weight - excluding slabs	User	Global Z		2	Full UDL	0.05	17′ 8″

Live Load Reduction

Loadcase	Imposed Load	Stack	Reduction Factor	Tributary Area	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack	Face A factor	Face C restrained/ Sub-stack	Face C factor
3	floor	35′ 0″	Yes		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack	...aj... ...ct...	Minor restrained / Sub-stack	...in... ...ct...
3	floor	35′ 0″	No		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 8x8x1/2 (A500B-46)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	7	Non-slender	-	-	-	✔ Pass
Flexural Classification	7	Compact	-	-	-	✔ Pass
Shear Major	16	0.6	152.6	kip	0.004	✔ Pass
Shear Minor	15	1.5	152.6	kip	0.010	✔ Pass
Flexure Major	16	-10.7	129.4	kip ft	0.082	✔ Pass
Flexure Minor	15	65.6	129.4	kip ft	0.507	✔ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	9	189.0	407.8	kip	0.464	✔ Pass
Combined Forces	15	-	-	-	0.861	✔ Pass
Design Note	Major and/or Minor Axes Strut Restrained					⚠ Warning

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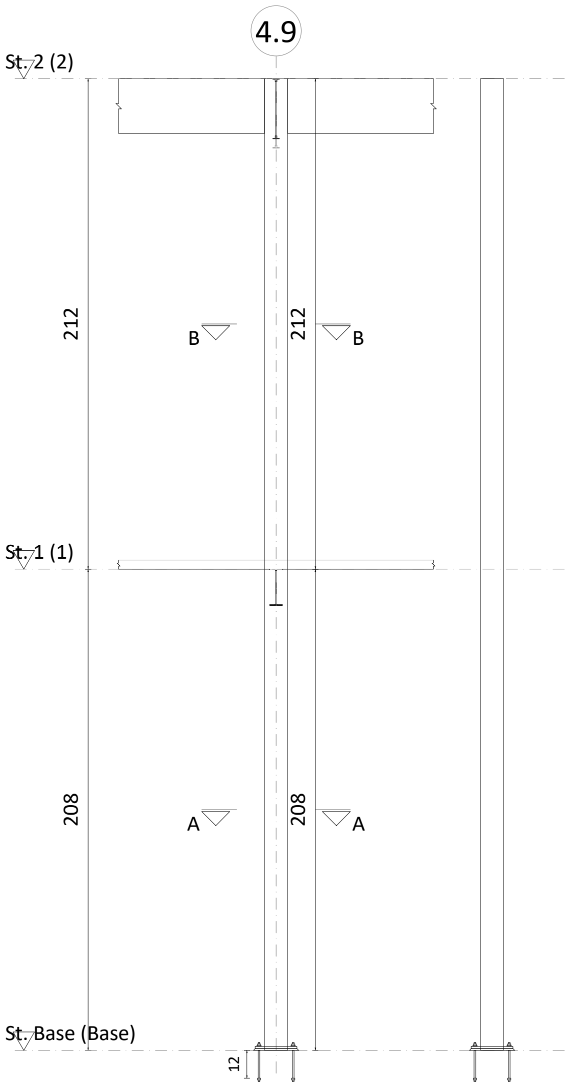
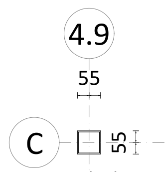
	Structure: Admin Building				Sheet no.: Page 1/2	
	Calc. by: Allison Valencia	Date: 11/6/2025	Chk'd by	Date	App'd by	Date

SC C/4.9



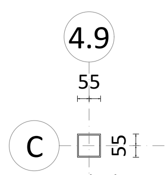
SC C/4.9

SC C/4.9 B-B



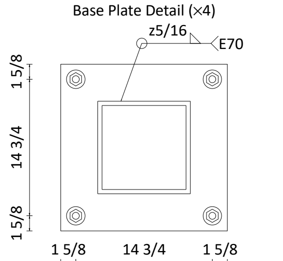
HSS 10x10x1/2 / A500B-46

SC C/4.9 A-A



HSS 10x10x1/2 / A500B-46

Base Plate Detail (×4)



BP Base/C/4.9
Base plate 18×18×1in/A36-36
Anchor rods 4 Ø3/4 in Gr 36 × 17in long

Loading

Loadcase	Source	Direction	In Proj.	Span/Stack	Type	Q_1 [klf]	...ngt... [ft, in]
1 Self weight - excluding slabs	User	Global Z		1	Full UDL	0.06	17′ 4″
1 Self weight - excluding slabs	User	Global Z		2	Full UDL	0.06	17′ 8″

Live Load Reduction

Loadcase	Imposed Load	Stack	Reduction Factor	Tributary Area	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack	Face A factor	Face C restrained/ Sub-stack	Face C factor
3	floor	35′ 0″	Yes		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Strut Restraints

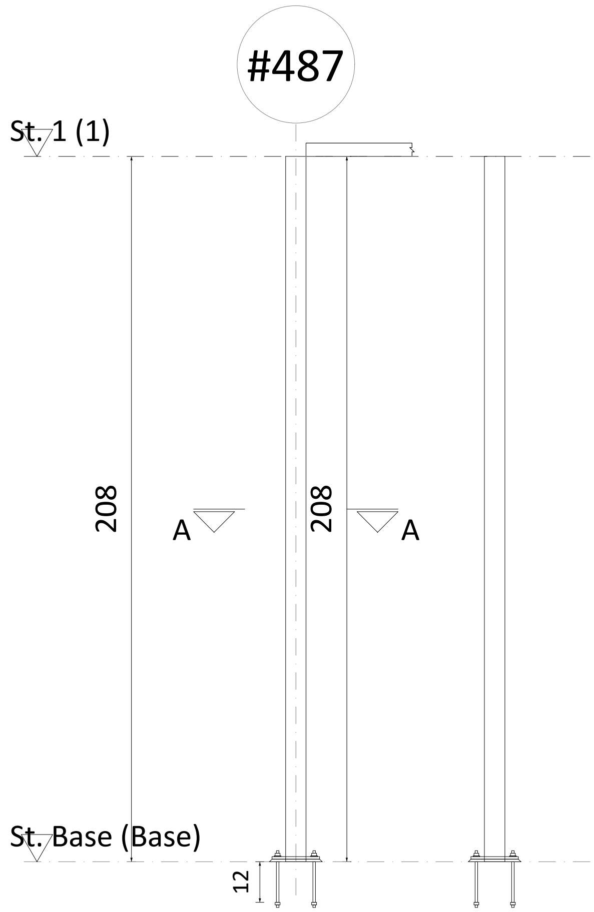
Level	Source	Distance / Length	Major restrained / Sub-stack	...aj... ...ct...	Minor restrained / Sub-stack	...in... ...ct...
3	floor	35′ 0″	Yes		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 10x10x1/2 (A500B-46)

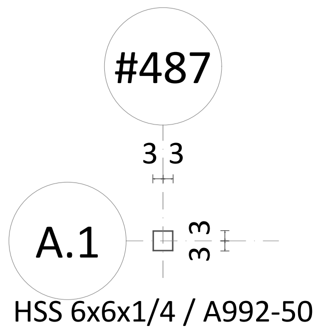
Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	7	Non-slender	-	-	-	✔ Pass
Flexural Classification	7	Compact	-	-	-	✔ Pass
Shear Major	17	0.7	198.8	kip	0.004	✔ Pass
Shear Minor	15	3.2	198.8	kip	0.016	✔ Pass
Flexure Major	17	13.0	209.4	kip ft	0.062	✔ Pass
Flexure Minor	18	-57.8	209.4	kip ft	0.276	✔ Pass
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	9	416.3	585.6	kip	0.711	⚠ Warning
Combined Forces	9	-	-	-	0.715	⚠ Warning

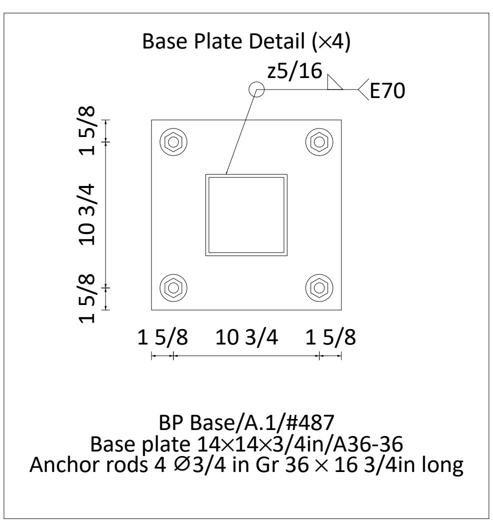
SC A.1/#487



SC A.1/#487

SC A.1/#487 A-A





Loading

Loadcase	Source	Direction	In Proj.	Span/Stack	Type	Q_1 [klf]	...ngt... [ft, in]
1 Self weight - excluding slabs	User	Global Z		1	Full UDL	0.02	17′ 4″

Live Load Reduction

Loadcase	Imposed Load	Stack	Reduction Factor	Tributary Area	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Lateral Restraints

Level	Source	Distance / Length	Face A restrained / Sub-stack	Face A factor	Face C restrained/ Sub-stack	Face C factor
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Strut Restraints

Level	Source	Distance / Length	Major restrained / Sub-stack	...aj... ...ct...	Minor restrained / Sub-stack	...in... ...ct...
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

Summary HSS 6x6x1/4 (A992-50)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	7	Non-slender	-	-	-	✔ Pass
Flexural Classification	7	Compact	-	-	-	✔ Pass
Shear Major	No	Significant	Forces	kip	-	Not required
Shear Minor	No	Significant	Forces	kip	-	Not required
Flexure Major	8	-19.1	42.0	kip ft	0.455	✔ Pass
Flexure Minor	No	Significant	Forces	kip ft	-	Not required
Axial Tension	No	Significant	Forces	kip	-	Not required
Axial Compression	8	45.7	132.2	kip	0.346	✔ Pass
Combined Forces	9	-	-	-	0.751	✔ Pass

Michael Baker
INTERNATIONAL

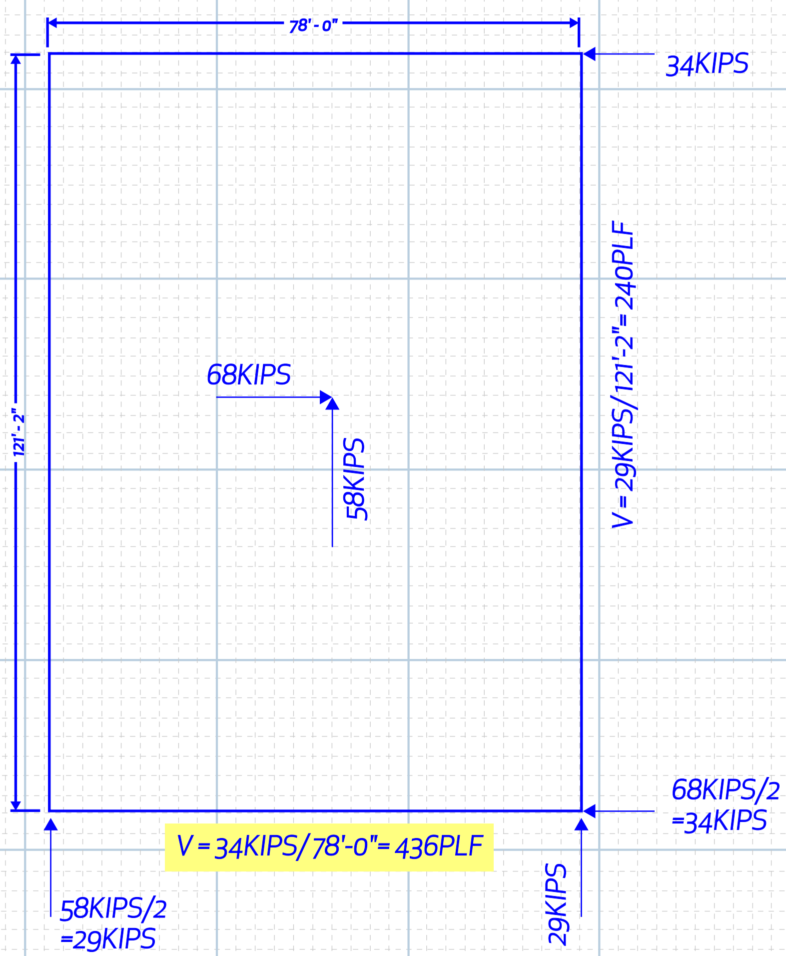
Lateral System Calculations

S.O. No. XXXXXX
Subject: Creech DRP PH 2 - Admin Building
Sheet No. 1 of 1
Drawing No.
Computed by ACV Checked by XXX Date 10/13/2025

Michael Baker INTERNATIONAL

Seismic Diaphram/Shear Wall Design

Roof Seismic Force - Flexible Diaphragm Distribution



S.O. No. XXXXXX

Subject: Creech DRP PH 2 - Admin Building

Sheet No. 1 of 1

Drawing No.

Computed by ACV Checked by XXX Date 10/13/2025

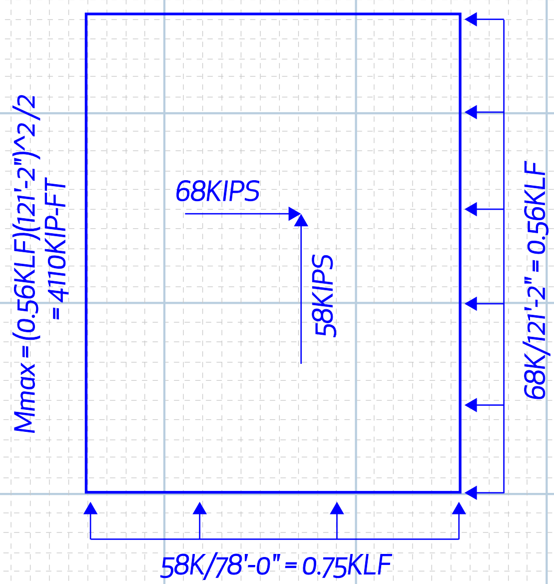
Michael Baker INTERNATIONAL

Seismic Shear Wall/Diaphragm Design

2nd Floor Seismic Force - Flexible Diaphragm Distribution

Chord Forces

Mmax = (0.75KLF)(78'-0")^2 /2
= 2282KIP-FT



Chord Force N/S:
Vh = 2282KIP-FT/121'-2"
= 18.9 KIPS

Area of Steel @ Masonry:
18.9 KIPS / 60KSI * 0.9
= 0.35sq in

Chord Force E/W:
Vh = 4110.8KIP-FT/78'-0"
= 52.7KIPS

Area of Steel @ Masonry:
52.7 KIPS / 60KSI * 0.9
= 0.98sq in

*Provide extra reinforcement at bond beam located at diaphragm

S.O. No. XXXXXX

Subject: Creech DRP PH 2 - Admin Building

Sheet No. 1 of 1

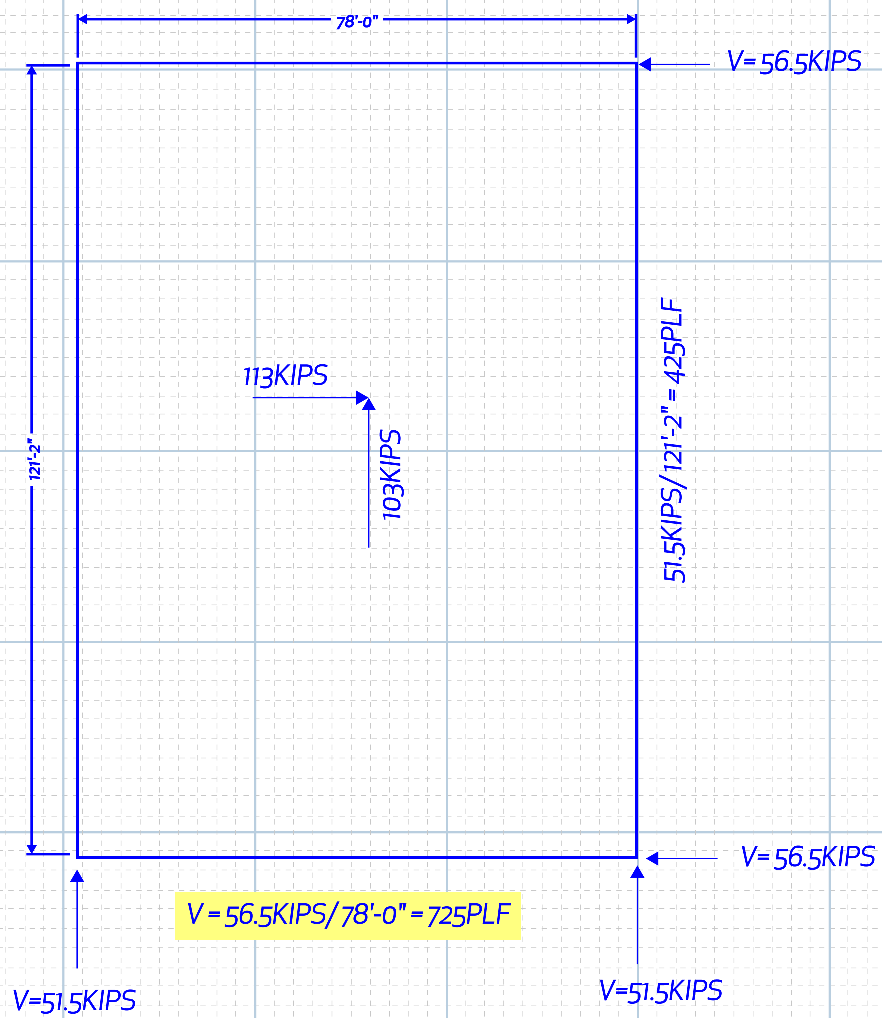
Drawing No.

Computed by ACV Checked by XXX Date 10/13/2025

Seismic Shear Wall/Diaphragm Design

2nd Floor Seismic Force - Rigid Diaphragm Distribution

Diaphragm Forces



S.O. No. XXXXXX

Subject: Creech DRP PH 2 - Admin Building

Michael Baker INTERNATIONAL

Sheet No. 1 of 1

Drawing No.

Computed by ACV Checked by XXX Date 10/13/2025

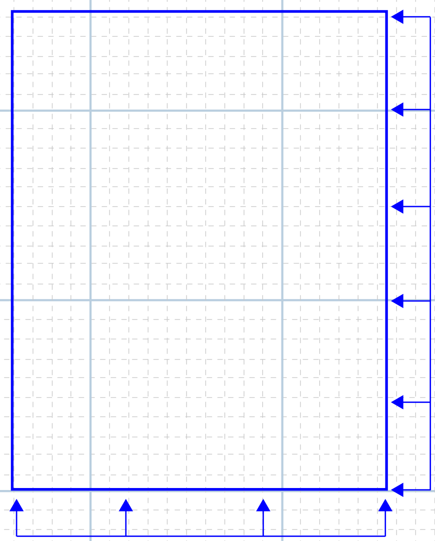
Seismic Shear Wall/Diaphragm Design

2nd Floor Seismic Force - Flexible Diaphragm Distribution

Chord Forces

Mmax = (1.32KLF)(78'-0")^2 /2
= 4016KIP-FT

Mmax = (0.94KLF)(121'-2")^2 /2
= 6900KIP-FT



113K/121'-2" = 0.94KLF

103K/78'-0" = 1.32KLF

Chord Force N/S:
Vh = 4016KIP-FT/121'-2"
= 33.1 KIPS

Area of Steel @ Masonry:
*33.1 KIPS / 60KSI * 0.9*
= 0.61sq in

Chord Force E/W:
Vh = 6900KIP-FT/78'-0"
= 89KIPS

Area of Steel @ Masonry:
*89 KIPS / 60KSI * 0.9*
= 1.65sq in

**Provide extra reinforcement at bond beam located at diaphragm*

20 ga 1.5B-36 Grade 50 Roof Deck

Diaphragm Shear & Wind Uplift Interaction

For Both Ends Butted Deck
with MWFRS Design Net Wind Uplift (LRFD) of 36.3 psf

Hilti X-HSN24 PAF Connections to Supports
36 / 7 / 4 Perpendicular Connection Pattern to Supports
#12 Screw Sidelap Connections

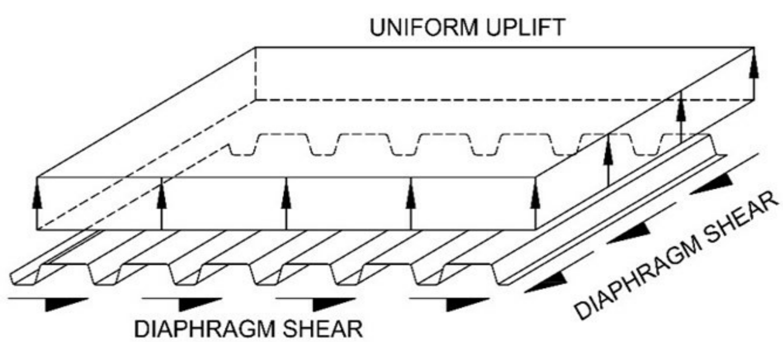
A572 GR50 Support Member or Equivalent
0.188 ≤ Support Thickness (in.) ≤ 0.375
2 in. Minimum Deck End Bearing Length

LRFD Design Combined Wind Uplift & Diaphragm Shear Strength ΦSn (plf) Generic 3 Span Condition

Sidelap Connections per Span	Span								
	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"
3	660	598	545	498	457	419	386	357	331
4	762	694	635	583	537	497	461	429	400
5	858	784	720	663	614	569	530	494	462
6	947	868	800	739	686	638	595	556	521
7	1029	947	875	811	753	702	656	615	577
8	1105	1020	944	877	817	763	714	670	629
9	1174	1087	1009	939	876	820	768	721	678

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span								
	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"
3	22	24	20	21	23	24	26	27	23
4	22	18	20	21	23	24	26	22	23
5	17	18	20	17	18	19	20	22	19
6	17	14	16	17	18	16	17	18	19
7	13	14	16	14	15	16	15	15	16
8	13	12	13	14	13	14	15	14	14
9	11	12	13	12	13	14	13	14	13



20 ga 1.5B-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Butted Deck

NUCOR VULCRAFT GROUP

Hilti X-HSN24 PAF Connections to Supports
36 / 7 / 4 Perpendicular Connection Pattern to Supports
#12 Screw Sidelap Connections

A572 GR50 Support Member or Equivalent
0.188 ≤ Support Thickness (in.) ≤ 0.375
2 in. Minimum Deck End Bearing Length

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.) — Generic 3 Span Condition

Sidelap Connections per Span	Span 5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"
3	80	79	78	77	75	74	72	70	69
4	85	85	84	83	82	80	79	77	75
5	90	89	89	88	87	86	84	83	82
6	93	93	93	93	92	91	90	88	87
7	96	97	97	97	96	95	94	93	92
8	99	100	100	100	100	99	98	97	96
9	101	102	103	103	103	102	102	101	100

LRFD Design Seismic Diaphragm Shear Strength ΦSn (plf) — Generic 3 Span Condition

Sidelap Connections per Span	Span 5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"
3	705	645	594	549	511	477	447	421	399
4	800	739	686	635	591	552	518	488	462
5	891	825	768	717	671	627	589	555	525
6	978	908	846	792	744	701	660	621	588
7	1062	987	922	864	813	767	726	688	652
8	1141	1064	995	934	880	831	787	747	711
9	1216	1136	1065	1002	945	893	847	805	766

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connections per Span	Span 5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"
3	22	24	20	21	23	24	26	27	23
4	22	24	20	21	23	24	26	27	23
5	22	24	20	21	23	24	26	27	23
6	17	18	20	21	23	24	26	27	23
7	17	18	20	21	23	24	26	27	23
8	17	18	20	21	18	19	20	22	23
9	13	14	16	17	18	19	20	22	23

Bare Deck Diaphragm V5.4 in accordance with:
AISI S100-16 (2020) w/ S2-20
IAPMO UES ER-0423
IAPMO UES ER-0652
CAN/CSA-S136 (R2021) for Canadian references

Date: 10/16/2025

Deck-Slab Diaphragm Shear Strength

20 ga 1.5VL-36 Grade 50 Composite Deck [4]

6 in. total slab depth, f'c = 4000 psi, 145 pcf NWC

Single Mat of Rebar #4 x #4 - 18 x 18 in o.c.

Hilti X-HSN-24 PAF at Chords & Collectors for Shear Transfer

Perpendicular Connection Pattern[1]	1 every other rib			
Parallel Connection Attachment (maximum)	1	row at	12	in. o.c.

Minimum Connections to Supporting Members[2]

Minimum connections to all supports may be any of the following: arc spot welds, fillet welds, PAF's, screws, Shearflex® anchors, welded studs or other mechanical connections[3].

Perpendicular Connection Pattern[1]	1 every other rib		
Parallel Perimeter Connections for deck spans greater than 5 ft		36	in. o.c.

Governing Deck-Slab Strength and Stiffness

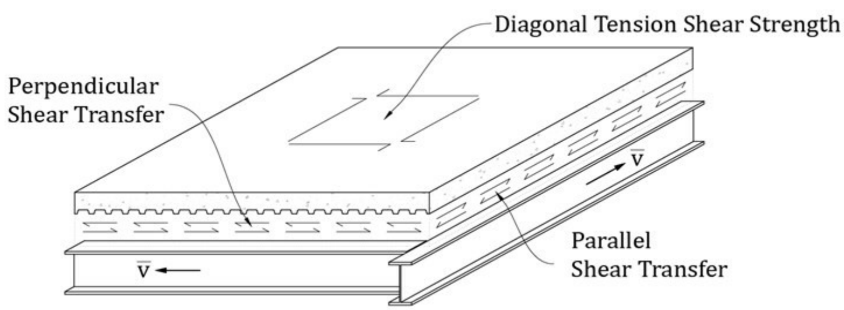
Available Diaphragm Design Strength	**Va = (ΦPn)N =**	**1257**	**plf**
Controlled by Connections to Chords and Collectors			
Deck-Slab Diaphragm Design Shear Stiffness	G' =	1657	kip/in

Deck-Slab Shear Strength

Reinforced Concrete Slab Design Shear Strength	Vu =	11123	plf

Chords & Collector Shear Transfer Strength

Chord & Collectors Design Shear Transfer Strength	(ΦPn)N =	1257	plf



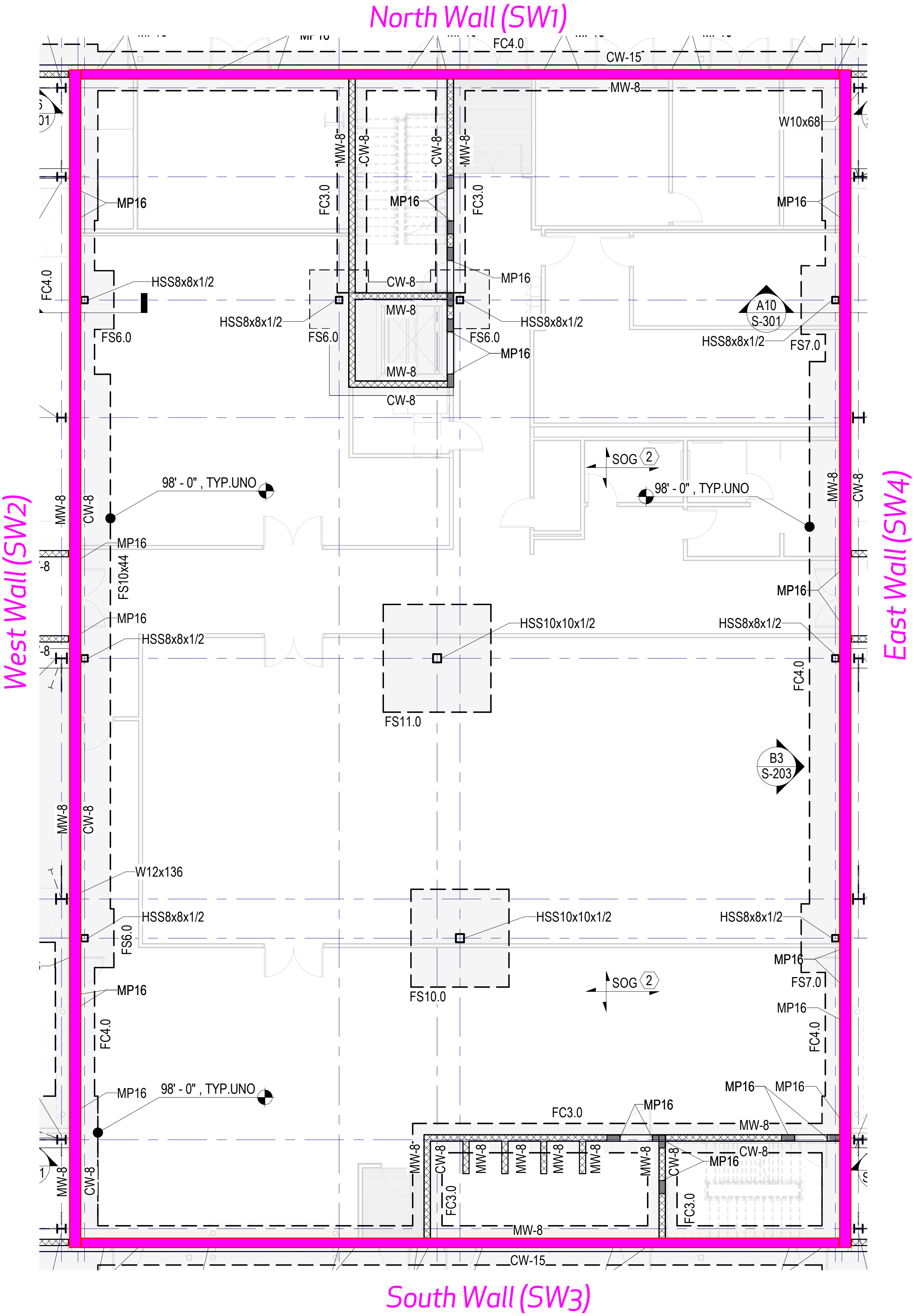
Notes:

1. For UL Fire rated assemblies, refer to UL Design Number for support and sidelap connection requirements.
2. Minimum connections to supporting members do not contribute to the diaphragm shear strength.
3. Support welds at interlocking sidelaps may be 3/8" x 1-1/4" arc seam welds in lieu of arc spot welds.
4. Sidelap connections between steel deck panels may be VSC2, button punch, screw, 1-1/2 in. arc seam weld or 1-1/2 in. top arc seam weld. The maximum sidelap connection spacing shall not exceed 36 in. o.c.
5. Dramix Steel Fibers up to 66 lb/cy are approved in lieu of welded wire fabric in all UL D700, D800 , and 900 Series Designs, and G229.

Deck-Slab Diaphragm V2.1.1 in accordance with: Date: 10/16/2025

AISI S100-16 (2020) w/ S2-20 & AISI S310-16 as modified by IAPMO ER-0652 or ER-0423

Shear Wall Plan/Labels



MASONRY WALL OPENING

CREECH DRP PH2 - N/S WALL: LINTEL DESIGN FOR ML1

Michael Baker INTERNATIONAL

General Design Input

Total Wall Height:	17.33 ft	Lintel Depth:	16.00 in	Vert DL:	2576 plf
Block Thickness:	8" CMU	No. of Long Bars:	(2)	Vert LL:	1280 plf
Opening Height:	7.50 ft	Bar Size:	#5	Vert SL:	947 plf
Opening Width:	4.00 ft	No. of Courses:	1	Horz Wind:	25 psf
Roof Deck Height:	17.33 ft			Horz Quake:	33 psf
Cells Grouted @:	Solid	Column Width:	16.00 in	Solid Opening:	Y
Masonry f'm:	2000 psi	Depth to Bar(s):	3.81 in		
		No. of Bars:	(1)		
		Bar Size:	#5		

Analysis and Calculations

<u>Masonry Lintel:</u>

Arching action: Yes

Effective Span, le: 5.33 ft

Depth to Reinf, d: 12.0 in

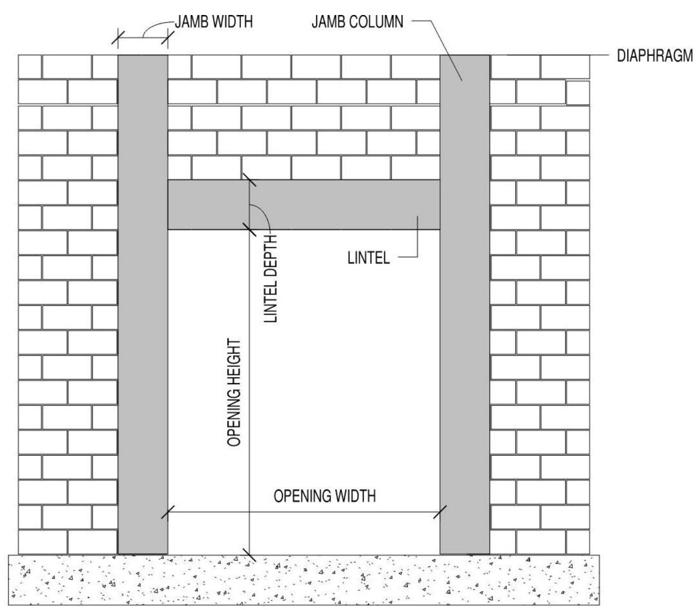
Demand	Capacity
Mu: 1.19 k-ft	fMn: 31.45 k-ft
Vu: 0.90 k	fVn: 9.82 k

Min. Capacity: 5.42 k-ft

Deflection: l/20159470

<u>Masonry Column:</u>

Demand	Capacity
Mu+: 4.1 k-ft	fMn: 4.2 k-ft
Mu-: 0.0 k-ft	fMn: 4.2 k-ft
Vu: 0.9 k	fVn: 22.5 k
Pu: 17.6 k	fPn: 83.6 k



Design Summary

<u>Masonry Lintel</u>

No. Grouted Courses:	2	
Flexural Reinforcing:	(2) #5	
hear Reinforcing (Not Req'd):	1-Leg	2-Legs
#3 @	-	-
#4 @	-	-
#5 @	-	-

<u>Masonry Column</u>

No. of Grouted Cells: 2

Long. Reinforcing: (1) #5

Depth to bar(s): 3.81 in

Overall Status: **OK**

MASONRY WALL OPENING

CREECH DRP PH2 - N/S WALL: LINTEL DESIGN FOR ML2

Michael Baker INTERNATIONAL

General Design Input

Total Wall Height:	17.33 ft	Lintel Depth:	24.00 in	Vert DL:	2576 plf
Block Thickness:	8" CMU	No. of Long Bars:	(2)	Vert LL:	1280 plf
Opening Height:	7.30 ft	Bar Size:	#5	Vert SL:	947 plf
Opening Width:	6.30 ft	No. of Courses:	1	Horz Wind:	25 psf
Roof Deck Height:	17.33 ft			Horz Quake:	33 psf
Cells Grouted @:	Solid	Column Width:	24.00 in	Solid Opening:	Y
Masonry f'm:	2000 psi	Depth to Bar(s):	5.31 in		
		No. of Bars:	(2)		
		Bar Size:	#5		

Analysis and Calculations

Masonry Lintel:

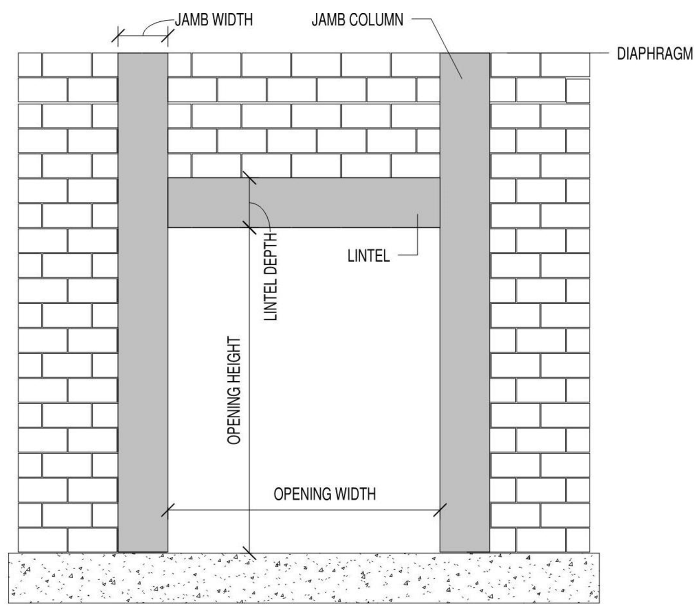
Arching action: Yes
Effective Span, le: 8.30 ft
Depth to Reinf, d: 20.0 in

Demand	Capacity
Mu: 4.18 k-ft	fMn: 54.45 k-ft
Vu: 2.01 k	fVn: 14.73 k

Min. Capacity: 12.20 k-ft
Deflection: l/3352506

Masonry Column:

Demand	Capacity
Mu+: 6.3 k-ft	fMn: 11.6 k-ft
Mu-: 0.0 k-ft	fMn: 11.6 k-ft
Vu: 1.5 k	fVn: 33.7 k
Pu: 27.4 k	fPn: 129.0 k



Design Summary

Masonry Lintel

No. Grouted Courses:	3	
Flexural Reinforcing:	(2) #5	
hear Reinforcing (Not Req'd):	1-Leg	2-Legs
#3 @	-	-
#4 @	-	-
#5 @	-	-

Masonry Column

No. of Grouted Cells:	3
Long. Reinforcing:	(4) #5
Depth to bar(s):	5.31 in

Overall Status: **OK**

MASONRY WALL OPENING

CREECH DRP PH2 - N/S WALL: LINTEL DESIGN FOR ML3

Michael Baker INTERNATIONAL

General Design Input

Total Wall Height:	17.30 ft	Lintel Depth:	24.00 in	Vert DL:	1536 plf
Block Thickness:	8" CMU	No. of Long Bars:	(2)	Vert LL:	240 plf
Opening Height:	14.00 ft	Bar Size:	#6	Vert SL:	0 plf
Opening Width:	12.00 ft	No. of Courses:	1	Horz Wind:	25 psf
Roof Deck Height:	17.33 ft			Horz Quake:	33 psf
Cells Grouted @:	Solid	Column Width:	24.00 in	Solid Opening:	Y
Masonry f'm:	2000 psi	Depth to Bar(s):	3.81 in		
		No. of Bars:	(3)		
		Bar Size:	#6		

Analysis and Calculations

Masonry Lintel:

Arching action: No

Effective Span, le: 14.00 ft

Depth to Reinf, d: 20.0 in

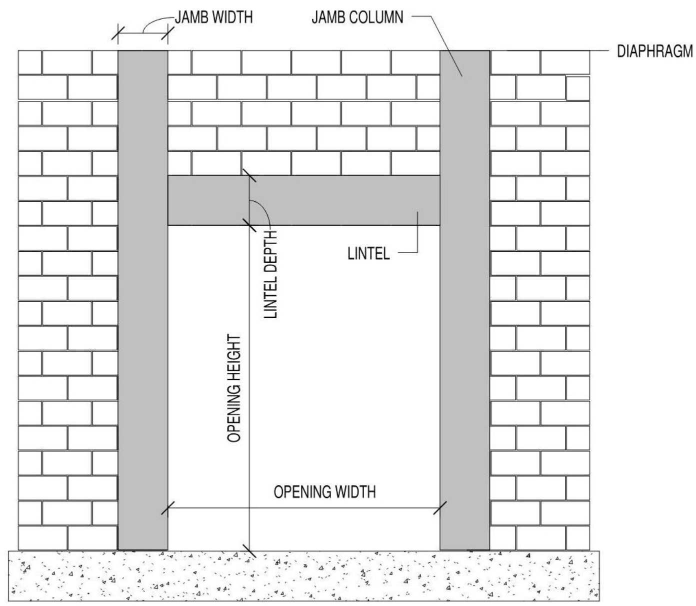
Demand	Capacity
Mu: 62.72 k-ft	fMn: 76.48 k-ft
Vu: 17.92 k	fVn: 17.92 k

Min. Capacity: 12.20 k-ft

Deflection: l/1126

Masonry Column:

Demand	Capacity
Mu+: 9.8 k-ft	fMn: 14.3 k-ft
Mu-: 0.0 k-ft	fMn: 14.3 k-ft
Vu: 2.3 k	fVn: 33.7 k
Pu: 18.8 k	fPn: 145.5 k



Design Summary

Masonry Lintel

No. Grouted Courses:	3	
Flexural Reinforcing:	(2) #6	
Shear Reinforcing (Req'd):	1-Leg	2-Legs
#3 @	-	12 in o.c.
#4 @	12 in o.c.	12 in o.c.
#5 @	12 in o.c.	12 in o.c.

Masonry Column

No. of Grouted Cells:	3
Long. Reinforcing:	(3) #6
Depth to bar(s):	3.81 in

Overall Status: **OK**

MASONRY WALL OPENING

CREECH DRP PH2 - LEVEL 2 OPENING: LINTEL DESIGN FOR ML1

Michael Baker INTERNATIONAL

General Design Input

Total Wall Height:	17.67 ft	Lintel Depth:	24.00 in	Vert DL:	300 plf
Block Thickness:	8" CMU	No. of Long Bars:	(2)	Vert LL:	0 plf
Opening Height:	14.00 ft	Bar Size:	#5	Vert SL:	710 plf
Opening Width:	12.00 ft	No. of Courses:	1	Horz Wind:	25 psf
Roof Deck Height:	17.67 ft			Horz Quake:	33 psf
Cells Grouted @:	Solid	Column Width:	24.00 in	Solid Opening:	Y
Masonry f'm:	2000 psi	Depth to Bar(s):	5.31 in		
		No. of Bars:	(2)		
		Bar Size:	#5		

Analysis and Calculations

Masonry Lintel:

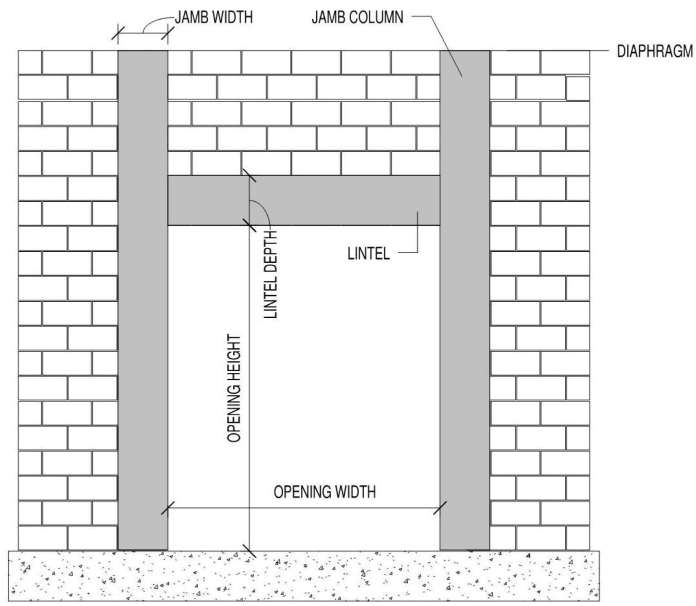
Arching action:	No
Effective Span, le:	14.00 ft
Depth to Reinf, d:	20.0 in

Demand	Capacity
Mu: 45.72 k-ft	fMn: 54.45 k-ft
Vu: 13.06 k	fVn: 14.73 k

Min. Capacity:	12.20 k-ft
Deflection:	l/1441

Masonry Column:

Demand	Capacity
Mu+: 10.2 k-ft	fMn: 11.6 k-ft
Mu-: 0.0 k-ft	fMn: 11.6 k-ft
Vu: 2.3 k	fVn: 33.5 k
Pu: 17.0 k	fPn: 124.8 k



Design Summary

Masonry Lintel

No. Grouted Courses:	3	
Flexural Reinforcing:	(2) #5	
hear Reinforcing (Not Req'd):	1-Leg	2-Legs
#3 @	-	-
#4 @	-	-
#5 @	-	-

Masonry Column

No. of Grouted Cells:	3
Long. Reinforcing:	(4) #5
Depth to bar(s):	5.31 in

Overall Status: **OK**

MASONRY WALL OPENING

CREECH DRP PH2 - E/W WALL: LINTEL DESIGN FOR ML1

Michael Baker INTERNATIONAL

General Design Input

Total Wall Height:	17.33 ft	Lintel Depth:	16.00 in	Vert DL:	2634 plf
Block Thickness:	8" CMU	No. of Long Bars:	(2)	Vert LL:	438 plf
Opening Height:	7.50 ft	Bar Size:	#5	Vert SL:	219 plf
Opening Width:	4.00 ft	No. of Courses:	1	Horz Wind:	25 psf
Roof Deck Height:	17.33 ft			Horz Quake:	33 psf
Cells Grouted @:	Solid	Column Width:	16.00 in	Solid Opening:	Y
Masonry f'm:	2000 psi	Depth to Bar(s):	3.81 in		
		No. of Bars:	(1)		
		Bar Size:	#5		

Analysis and Calculations

Masonry Lintel:

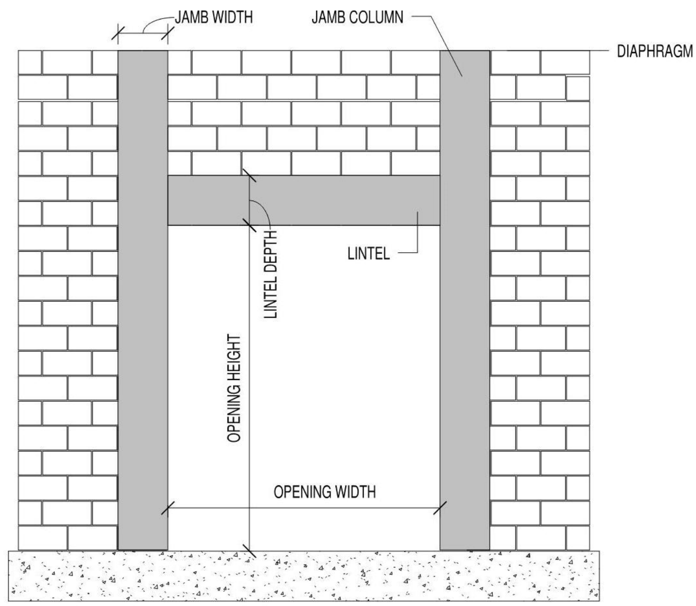
Arching action: Yes
Effective Span, le: 5.33 ft
Depth to Reinf, d: 12.0 in

Demand	Capacity
Mu: 1.19 k-ft	fMn: 31.45 k-ft
Vu: 0.90 k	fVn: 9.82 k

Min. Capacity: 5.42 k-ft
Deflection: l/20159470

Masonry Column:

Demand	Capacity
Mu+: 4.1 k-ft	fMn: 4.2 k-ft
Mu-: 0.0 k-ft	fMn: 4.2 k-ft
Vu: 0.9 k	fVn: 22.5 k
Pu: 12.6 k	fPn: 83.6 k



Design Summary

Masonry Lintel

No. Grouted Courses:	2	
Flexural Reinforcing:	(2) #5	
hear Reinforcing (Not Req'd):	1-Leg	2-Legs
#3 @	-	-
#4 @	-	-
#5 @	-	-

Masonry Column

No. of Grouted Cells: 2
Long. Reinforcing: (1) #5
Depth to bar(s): 3.81 in

Overall Status: **OK**

MASONRY WALL OPENING

CREECH DRP PH2 - E/W WALL: LINTEL DESIGN FOR ML2

Michael Baker INTERNATIONAL

General Design Input

Total Wall Height:	17.33 ft	Lintel Depth:	32.00 in	Vert DL:	2634 plf
Block Thickness:	8" CMU	No. of Long Bars:	(2)	Vert LL:	438 plf
Opening Height:	14.00 ft	Bar Size:	#6	Vert SL:	219 plf
Opening Width:	12.00 ft	No. of Courses:	1	Horz Wind:	25 psf
Roof Deck Height:	17.33 ft			Horz Quake:	33 psf
Cells Grouted @:	Solid	Column Width:	24.00 in	Solid Opening:	Y
Masonry f'm:	2000 psi	Depth to Bar(s):	5.31 in		
		No. of Bars:	(2)		
		Bar Size:	#5		

Analysis and Calculations

<u>Masonry Lintel:</u>

Arching action: No

Effective Span, le: 14.00 ft

Depth to Reinf, d: 28.0 in

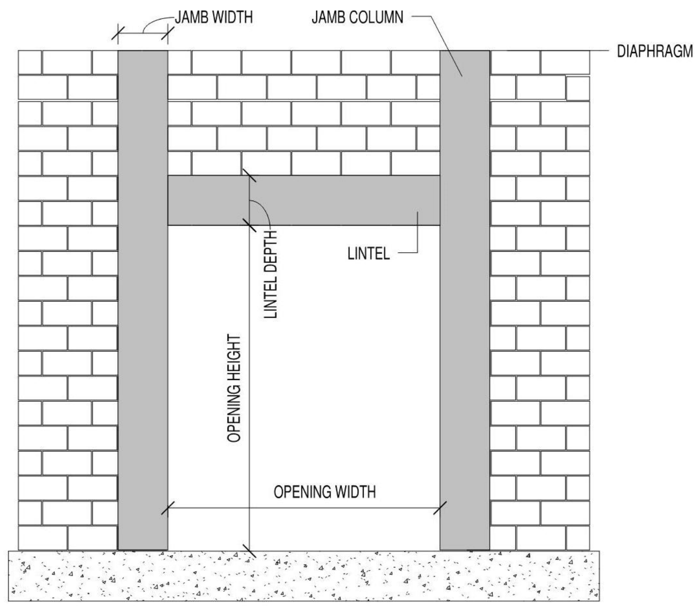
Demand	Capacity
Mu: 105.50 k-ft	fMn: 108.84 k-ft
Vu: 30.14 k	fVn: 30.14 k

Min. Capacity: 21.69 k-ft

Deflection: l/1349

<u>Masonry Column:</u>

Demand	Capacity
Mu+: 9.8 k-ft	fMn: 11.6 k-ft
Mu-: 0.0 k-ft	fMn: 11.6 k-ft
Vu: 2.3 k	fVn: 33.7 k
Pu: 30.1 k	fPn: 129.0 k



Design Summary

<u>Masonry Lintel</u>

No. Grouted Courses:	4	
Flexural Reinforcing:	(2) #6	
Shear Reinforcing (Req'd):	1-Leg	2-Legs
#3 @	-	16 in o.c.
#4 @	14 in o.c.	16 in o.c.
#5 @	16 in o.c.	16 in o.c.

<u>Masonry Column</u>

No. of Grouted Cells:	3
Long. Reinforcing:	(4) #5
Depth to bar(s):	5.31 in

Overall Status: **OK**

RAM Elements Michael Baker

SW1

NORTH Shear Wall Design

Current Date: 10/15/2025 4:35 PM
Units system: English
File name: C:\Users\allison.valencia\Michael Baker International\196483 Creech DRP Part 2 - General 1\05_Deliverables\07_Design Data\05_Struc\04 Lateral Design\Admin Building\North Shear Walls.msw

Design Results
Masonry wall

General Information

Global status : OK

Design code	:	TMS 402-22 ASD

Materials:

Material	:	CMU 2.0-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Full grouting
Masonry compression strength (F`m)	:	2 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	32 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1800 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC D
Response modification factor	:	1.00
Shear wall type	:	Special

Geometry

Total height	:	39.00 [ft]
Total length	:	79.30 [ft]
Foundation type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	17.33	7.63	0.14
2	17.67	7.63	0.14

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	2.70	0.00	3.30	7.30
Lower left	7.80	0.00	6.30	7.30
Lower left	19.60	0.00	6.30	7.30
Lower left	32.60	0.00	6.30	7.30
Lower left	41.70	0.00	3.30	7.30
Lower left	59.30	0.00	6.30	7.30
Lower left	73.30	0.00	3.30	7.30
Lower right	7.40	17.40	12.00	14.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
Wx	No	WIND	Wind in X
Wz	No	WIND	Wind in Z
EQx	No	EQ	Seismic in X
EQz	No	EQ	Seismic in Z
SM1	Yes		DL
DM1	Yes		DL
D1	Yes		1.4DL
D2	Yes		1.2DL+0.5Wx
D3	Yes		1.2DL+0.5Wz
D4	Yes		1.2DL+Wx
D5	Yes		1.2DL+Wz
D6	Yes		0.9DL+Wx
D7	Yes		0.9DL+Wz
D8	Yes		1.2DL+EQx
D9	Yes		1.2DL+EQz
D10	Yes		0.9DL+EQx
D11	Yes		0.9DL+EQz

Loads

In-plane seismic weight:

Load condition	Coefficient
EQx	0.12

Distributed loads:

Consider self weight : DL

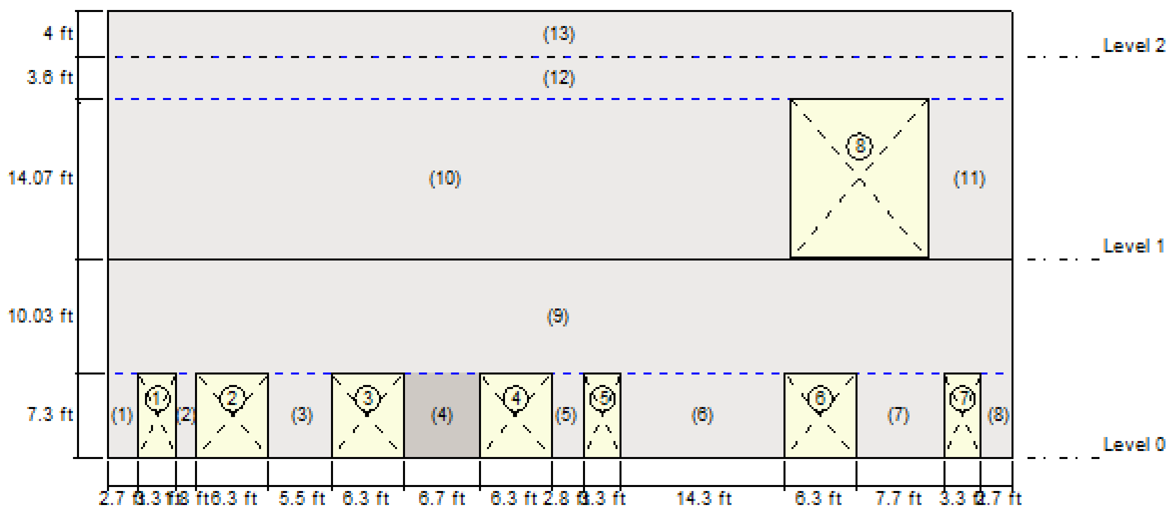
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.73	0.00
2	EQx	Horizontal	0.44	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wx	16.50
2	Wx	16.50
Parapet	Wx	16.50

Shear Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	2.70	7.30
	2	6.00	0.00	1.80	7.30
	3	14.10	0.00	5.50	7.30
	4	25.90	0.00	6.70	7.30
	5	38.90	0.00	2.80	7.30
	6	45.00	0.00	14.30	7.30
	7	65.60	0.00	7.70	7.30
	8	76.60	0.00	2.70	7.30
	9	0.00	7.30	79.30	10.03
1	10	0.00	17.33	59.30	14.07
	11	71.90	17.33	7.40	14.07
	12	0.00	31.40	79.30	3.60
2	13	0.00	35.00	79.30	4.00

Reinforcement

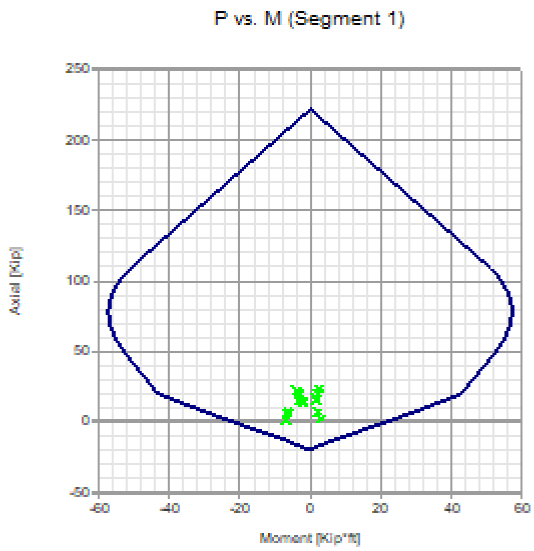
	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	2-#5	24.00	34.07	3-#4	32.00	21.80
2	1-#5	24.00	34.07	3-#4	32.00	21.80
3	3-#5	24.00	34.07	3-#4	32.00	21.80
4	4-#5	24.00	34.07	3-#4	32.00	21.80
5	2-#5	24.00	34.07	3-#4	32.00	21.80

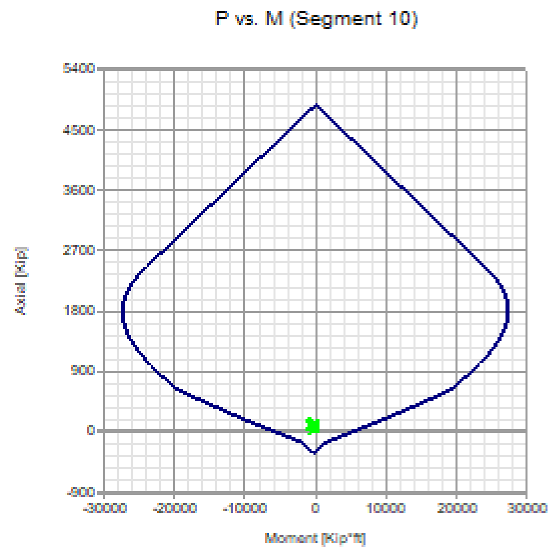
6	7-#5	24.00	34.07	3-#4	32.00	21.80
7	3-#5	24.00	34.07	3-#4	32.00	21.80
	1-#5	24.00	34.07	3-#4	32.00	21.80
8	2-#5	24.00	34.07	3-#4	32.00	21.80
9	2-#5	24.00	34.07	4-#4	32.00	21.80
	2-#5	24.00	34.07	4-#4	32.00	21.80
	1-#5	24.00	34.07	4-#4	32.00	21.80
	3-#5	24.00	34.07	4-#4	32.00	21.80
	3-#5	24.00	34.07	4-#4	32.00	21.80
	3-#5	24.00	34.07	4-#4	32.00	21.80
	4-#5	24.00	34.07	4-#4	32.00	21.80
	3-#5	24.00	34.07	4-#4	32.00	21.80
	2-#5	24.00	34.07	4-#4	32.00	21.80
	2-#5	24.00	34.07	4-#4	32.00	21.80
	7-#5	24.00	34.07	4-#4	32.00	21.80
	3-#5	24.00	34.07	4-#4	32.00	21.80
	3-#5	24.00	34.07	4-#4	32.00	21.80
	1-#5	24.00	34.07	4-#4	32.00	21.80
	2-#5	24.00	34.07	4-#4	32.00	21.80
	2-#5	24.00	34.07	4-#4	32.00	21.80
10	2-#5	24.00	34.07	6-#4	32.00	21.80
	2-#5	24.00	34.07	6-#4	32.00	21.80
	1-#5	24.00	34.07	6-#4	32.00	21.80
	3-#5	24.00	34.07	6-#4	32.00	21.80
	3-#5	24.00	34.07	6-#4	32.00	21.80
	3-#5	24.00	34.07	6-#4	32.00	21.80
	4-#5	24.00	34.07	6-#4	32.00	21.80
	3-#5	24.00	34.07	6-#4	32.00	21.80
	2-#5	24.00	34.07	6-#4	32.00	21.80
	2-#5	24.00	34.07	6-#4	32.00	21.80
	7-#5	24.00	34.07	6-#4	32.00	21.80
11	1-#5	24.00	34.07	6-#4	32.00	21.80
	2-#5	24.00	34.07	6-#4	32.00	21.80
	2-#5	24.00	34.07	6-#4	32.00	21.80
12	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	1-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	4-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	7-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	1-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
13	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	1-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	4-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	7-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	1-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80

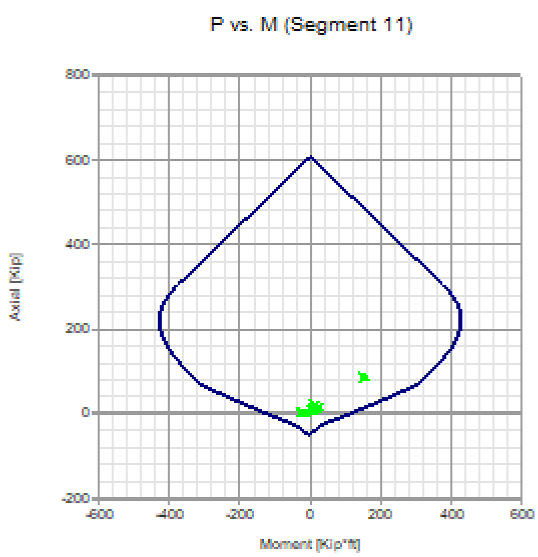
Combined axial flexure

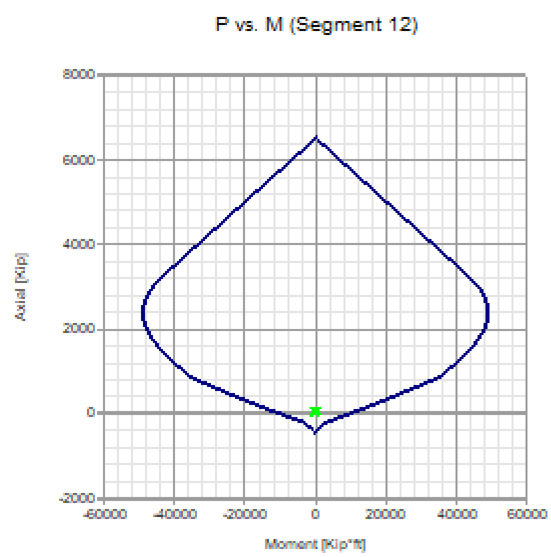
Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D10(Max)	0.91	-6.92	22.99	0.30
2	D10(Top)	4.53	2.41	4.70	0.51
3	D10(Bottom)	19.94	-35.56	107.33	0.33
4	D10(Bottom)	30.52	-48.86	169.15	0.29
5	D10(Bottom)	13.54	-9.07	38.12	0.24
6	D10(Bottom)	67.15	-158.93	715.97	0.22
7	D10(Bottom)	41.15	-52.07	236.22	0.22
8	D8(Bottom)	26.25	-7.05	45.29	0.16
9	D10(Bottom)	186.59	-1799.44	15916.68	0.11
10	D10(Max)	87.98	-623.15	7817.35	0.08
11	D10(Bottom)	86.35	154.90	326.56	0.47
12	D8(Bottom)	59.25	-352.09	11968.29	0.03
13	D8(Bottom)	25.12	-38.59	10890.41	0.00

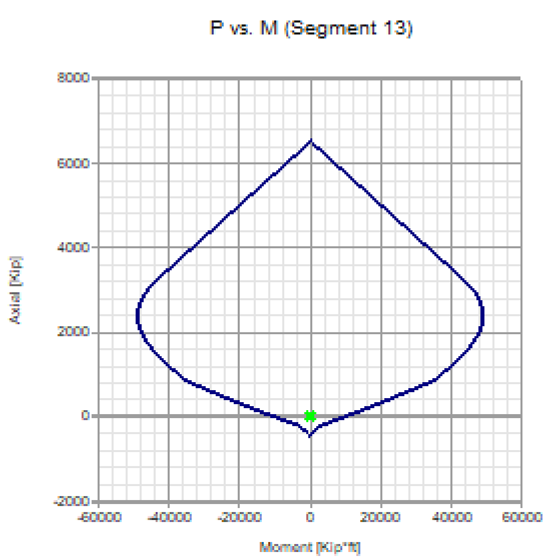
Interaction diagrams, P vs. M

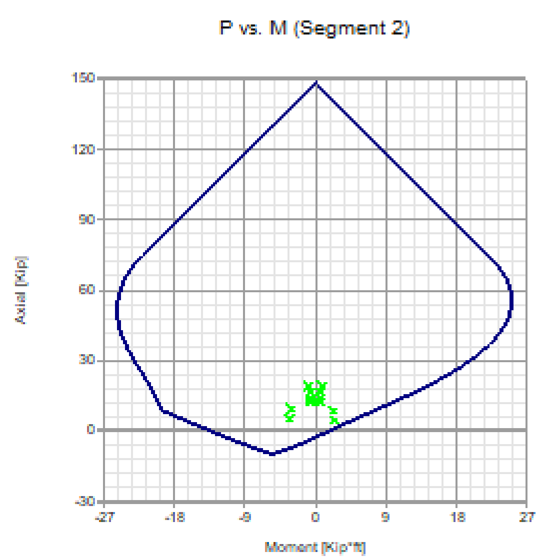

P vs. M (Segment 1)
Axial [Kip]
Moment [Kip*ft]
250
200
150
100
50
0
-50
-60
-40
-20
0
20
40
60


P vs. M (Segment 10)
Axial [Kip]
Moment [Kip*ft]
5400
4500
3600
2700
1800
900
0
-900
-30000
-20000
-10000
0
10000
20000
30000

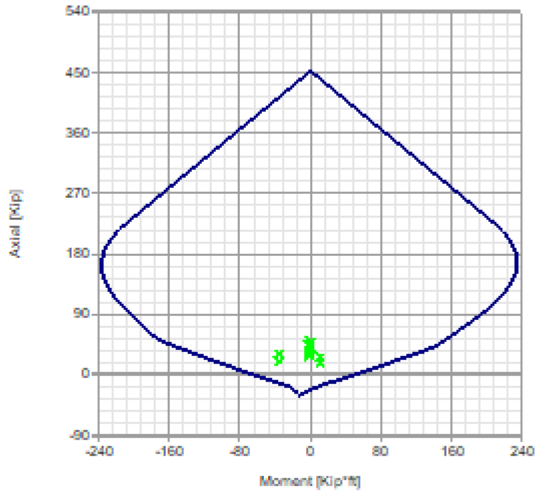

P vs. M (Segment 11)
Axial [Kip]
Moment [Kip*ft]
800
600
400
200
0
-200
-600
-400
-200
0
200
400
600


P vs. M (Segment 12)
Axial [Kip]
Moment [Kip*ft]
8000
6000
4000
2000
0
-2000
-60000
-40000
-20000
0
20000
40000
60000

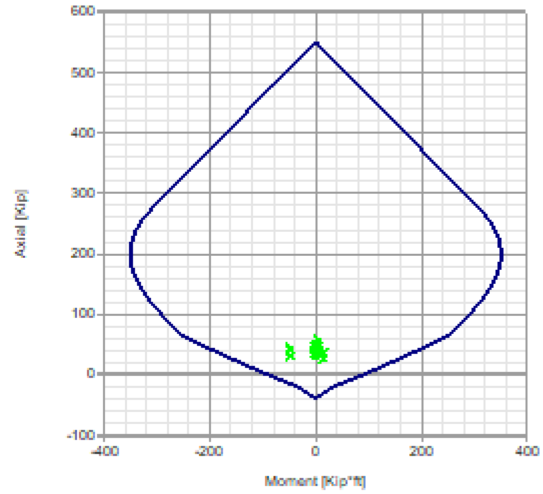

P vs. M (Segment 13)
Axial [Kip]
Moment [Kip*ft]
8000
6000
4000
2000
0
-2000
-60000
-40000
-20000
0
20000
40000
60000


P vs. M (Segment 2)
Axial [Kip]
Moment [Kip*ft]
150
120
90
60
30
0
-30
-27
-18
-9
0
9
18
27

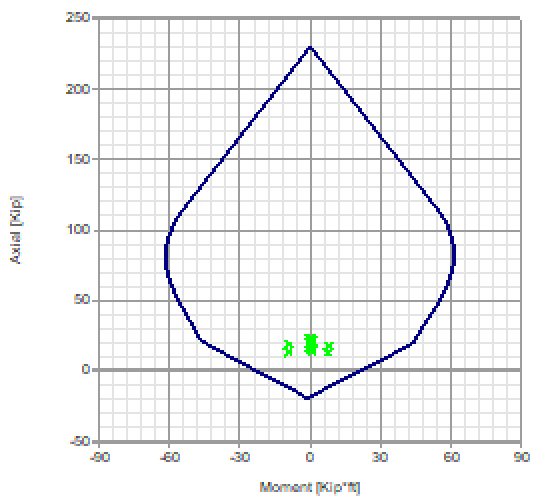
P vs. M (Segment 3)



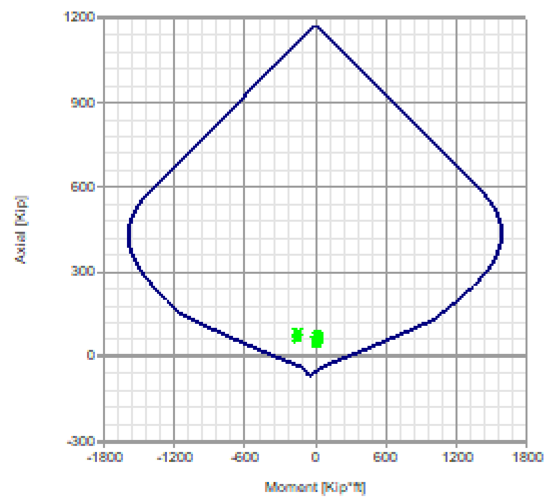
P vs. M (Segment 4)



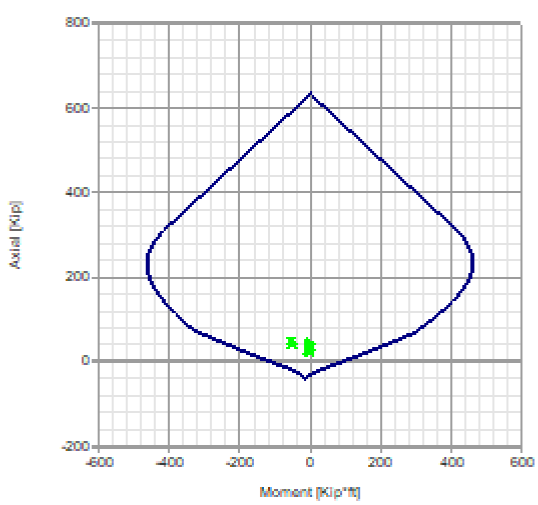
P vs. M (Segment 5)



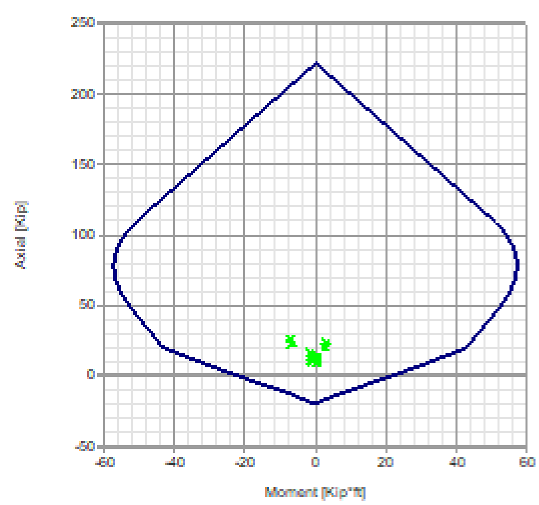
P vs. M (Segment 6)

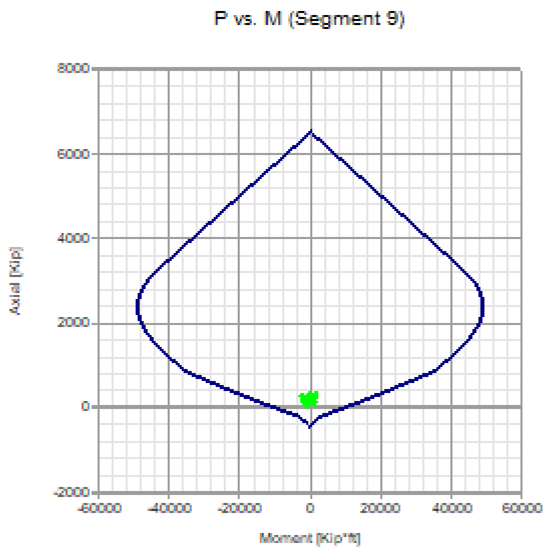


P vs. M (Segment 7)



P vs. M (Segment 8)





Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio
1	D1(Bottom)	23.38	80.64	0.29
2	D1(Max)	19.34	53.79	0.36
3	D1(Bottom)	49.77	164.37	0.30
4	D1(Max)	57.14	200.20	0.29
5	D1(Bottom)	23.16	83.63	0.28
6	D8(Max)	85.73	427.44	0.20
7	D8(Max)	50.21	230.14	0.22
8	D8(Max)	26.40	80.64	0.33
9	D1(Bottom)	304.40	2369.93	0.13
10	D1(Bottom)	143.39	1714.79	0.08
11	D8(Bottom)	86.90	213.89	0.41
12	D1(Bottom)	66.10	2293.12	0.03
13	D1(Max)	28.69	4243.06	0.01

Axial tension

Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio
1	DM1(Top)	0.00	32.00	0.00
2	DM1(Top)	0.00	32.00	0.00
3	DM1(Top)	0.00	32.00	0.00
4	DM1(Top)	0.00	32.00	0.00
5	DM1(Top)	0.00	32.00	0.00
6	DM1(Top)	0.00	32.00	0.00
7	DM1(Top)	0.00	32.00	0.00
8	DM1(Top)	0.00	32.00	0.00
9	DM1(Top)	0.00	32.00	0.00
10	DM1(Top)	0.00	32.00	0.00
11	DM1(Top)	0.00	32.00	0.00
12	DM1(Top)	0.00	32.00	0.00
13	DM1(Top)	0.00	32.00	0.00

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio
1	D10(Max)	0.036	0.079	0.46
2	D10(Max)	0.028	0.075	0.38
3	D10(Max)	0.075	0.098	0.76
4	D10(Max)	0.083	0.103	0.81
5	D10(Max)	0.050	0.095	0.53
6	D10(Max)	0.077	0.105	0.73
7	D10(Max)	0.066	0.104	0.64
8	D8(Max)	0.032	0.100	0.32
9	D10(Max)	0.034	0.102	0.33
10	D10(Bottom)	0.018	0.098	0.18
11	D10(Max)	0.009	0.069	0.13
12	D10(Bottom)	0.011	0.100	0.11
13	D8(Max)	0.001	0.101	0.01

Notes

* P = Axial load
* Pa = Allowable compressive force due to axial load.
* M = Moment at the section under consideration.
* Ma = Wall allowable moment due to axial force or lintel pure flexure allowable moment
* fa = Calculated compressive stress due to axial load only
* fb = Calculated compressive stress due to axial flexure only
* ft = Calculated axial tension
* Fa = Allowable compressive stress due to axial load only
* Fb = Allowable compressive stress due to axial flexure only
* fv = Calculated shear stress
* Fs = Allowable tensile or compressive stress
* Fv = Allowable shear stress
* ld = Embedment length
* As = Effective cross sectional area of reinforcement
* δs = Calculated deflection
* δmax = Maximum allowable deflection

RAM Elements Michael Baker

SW3 SOUTH Shear Wall Design

Current Date: 10/15/2025 4:39 PM
Units system: English
File name: C:\Users\allison.valencia\Michael Baker International\196483 Creech DRP Part 2 - General 1\05_Deliverables\07_Design Data\05_Struc\04 Lateral Design\Admin Building\South Shear Wall.msw

Design Results
Masonry wall

General Information

Global status : OK

Design code : TMS 402-22 ASD

Materials:

Material	:	CMU 2.0-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Full grouting
Masonry compression strength (F`m)	:	2 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	32 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1800 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC D
Response modification factor	:	1.00
Shear wall type	:	Special

Geometry

Total height	:	39.00 [ft]
Total length	:	79.30 [ft]
Foundation type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	17.33	7.63	0.14
2	17.67	7.63	0.14

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	4.00	0.00	3.30	7.30
Lower left	14.70	0.00	12.00	14.00
Lower left	31.30	0.00	3.30	7.30
Lower left	38.00	0.00	3.30	7.30
Lower left	56.00	0.00	3.30	7.30
Lower left	74.00	0.00	3.30	7.30

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
Wx	No	WIND	Wind in X
Wz	No	WIND	Wind in Z
EQx	No	EQ	Seismic in X
EQz	No	EQ	Seismic in Z
SM1	Yes		DL
DM1	Yes		DL
D1	Yes		1.4DL
D2	Yes		1.2DL+0.5Wx
D3	Yes		1.2DL+0.5Wz
D4	Yes		1.2DL+Wx
D5	Yes		1.2DL+Wz
D6	Yes		0.9DL+Wx
D7	Yes		0.9DL+Wz
D8	Yes		1.2DL+EQx
D9	Yes		1.2DL+EQz
D10	Yes		0.9DL+EQx
D11	Yes		0.9DL+EQz

Loads

In-plane seismic weight:

Load condition	Coefficient
EQx	0.12

Distributed loads:

Consider self weight : DL

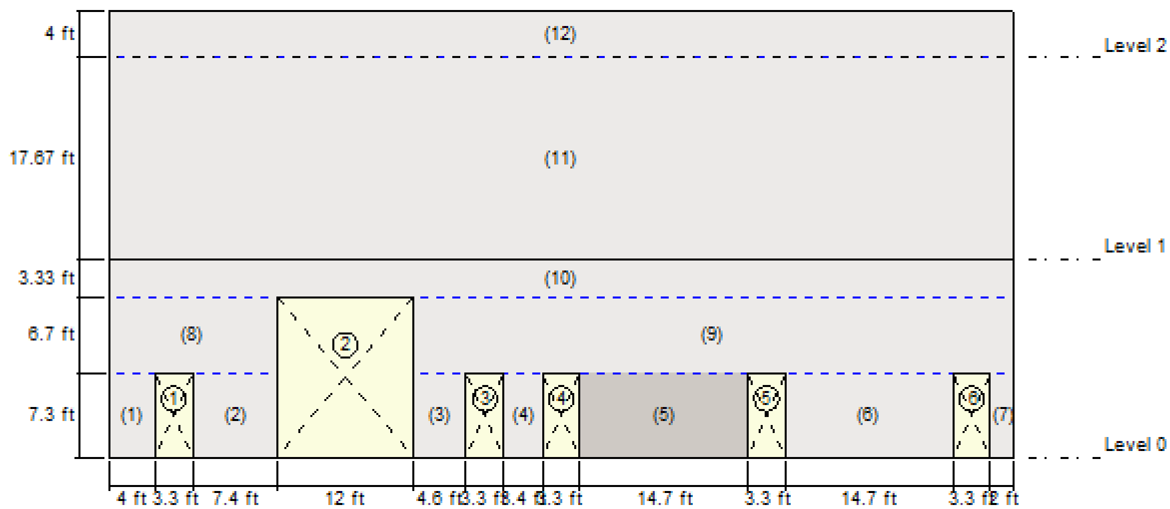
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.73	0.00
2	EQx	Horizontal	0.44	0.00

Out-of-plane loads:

Story	Condition	Magnitude [Kip/ft2]
1	Wx	16.50
2	Wx	16.50
Parapet	Wx	16.50

Shear Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	4.00	7.30
	2	7.30	0.00	7.40	7.30
	3	26.70	0.00	4.60	7.30
	4	34.60	0.00	3.40	7.30
	5	41.30	0.00	14.70	7.30
	6	59.30	0.00	14.70	7.30
	7	77.30	0.00	2.00	7.30
	8	0.00	7.30	14.70	6.70
	9	26.70	7.30	52.60	6.70
	10	0.00	14.00	79.30	3.33
1	11	0.00	17.33	79.30	17.67
2	12	0.00	35.00	79.30	4.00

Reinforcement

Segment	Vertical reinforcement			Horizontal reinforcement		
	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	2-#5	24.00	34.07	3-#4	32.00	21.80
2	4-#5	24.00	34.07	3-#4	32.00	21.80
3	3-#5	24.00	34.07	3-#4	32.00	21.80
4	2-#5	24.00	34.07	3-#4	32.00	21.80
5	8-#5	24.00	34.07	3-#4	32.00	21.80
6	8-#5	24.00	34.07	3-#4	32.00	21.80

7	1-#5	24.00	34.07	3-#4	32.00	21.80
8	2-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	4-#5	24.00	34.07	3-#4	32.00	21.80
9	3-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	8-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	8-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	1-#5	24.00	34.07	3-#4	32.00	21.80
10	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	4-#5	24.00	34.07	2-#4	32.00	21.80
	6-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	1-#5	24.00	34.07	2-#4	32.00	21.80
11	2-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	4-#5	24.00	34.07	7-#4	32.00	21.80
	6-#5	24.00	34.07	7-#4	32.00	21.80
	3-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	8-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	8-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	1-#5	24.00	34.07	7-#4	32.00	21.80
12	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	4-#5	24.00	34.07	2-#4	32.00	21.80
	6-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	1-#5	24.00	34.07	2-#4	32.00	21.80

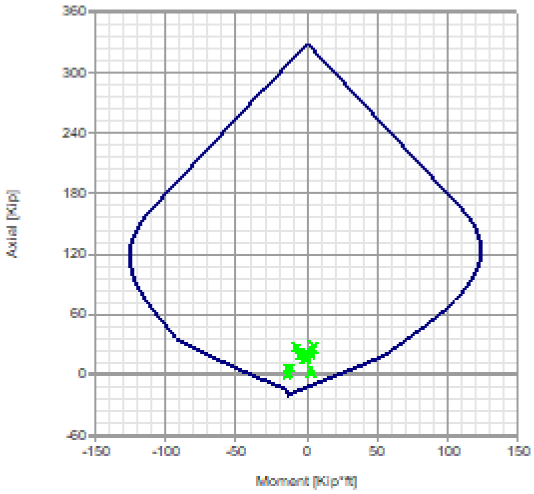
Combined axial flexure

Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D10(Max)	0.19	-14.10	39.29	0.36
2	D10(Bottom)	32.85	-58.86	200.86	0.29
3	D10(Max)	9.90	-25.64	66.68	0.38
4	D10(Bottom)	12.61	-13.66	47.07	0.29
5	D10(Bottom)	54.77	-144.14	676.16	0.21
6	D10(Bottom)	81.80	-183.33	824.80	0.22

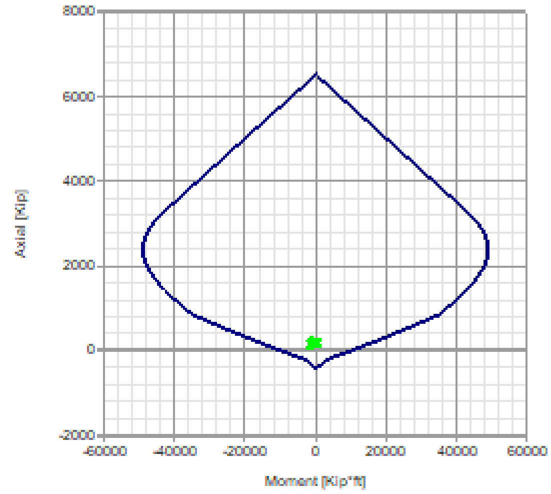
7	D8(Max)	21.89	2.99	18.01	0.17	
8	D10(Bottom)	23.33	-66.90	506.50	0.13	
9	D10(Bottom)	166.75	-951.23	8130.43	0.12	
10	D10(Top)	142.36	-1064.39	14326.80	0.07	
11	D10(Bottom)	119.41	-786.65	13618.07	0.06	
12	D10(Bottom)	18.57	-14.85	10459.61	0.00	

Interaction diagrams, P vs. M

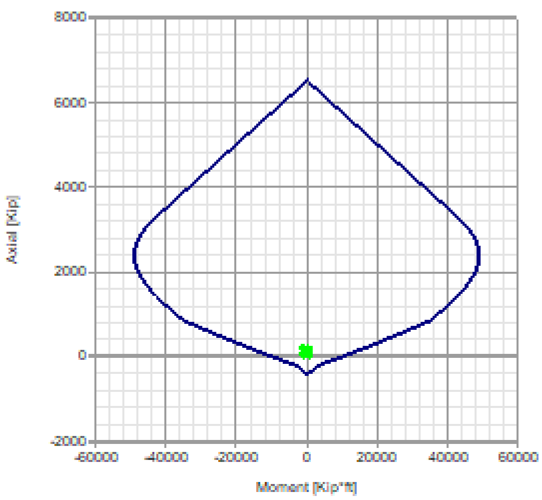
P vs. M (Segment 1)



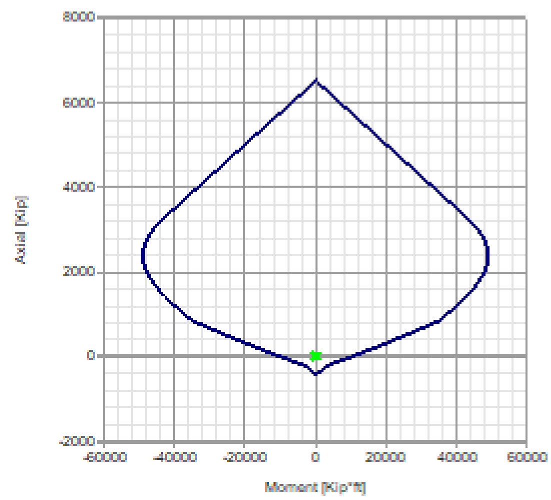
P vs. M (Segment 10)



P vs. M (Segment 11)

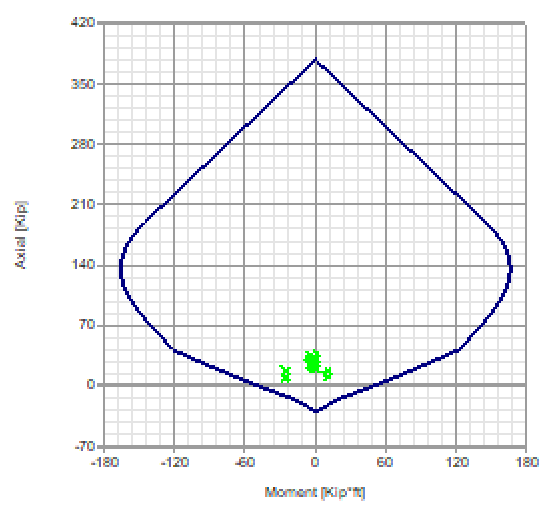


P vs. M (Segment 12)



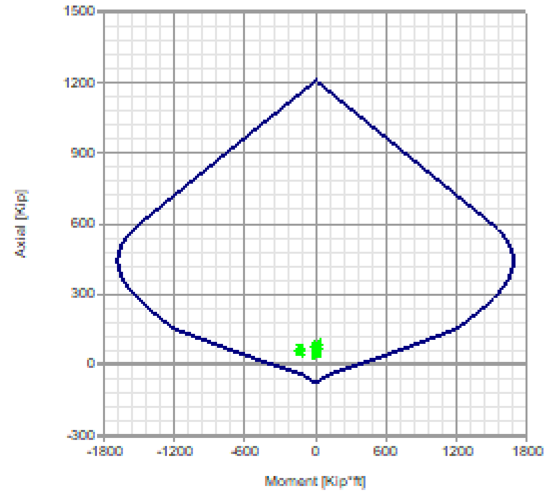
P vs. M (Segment 2)

P vs. M (Segment 3)

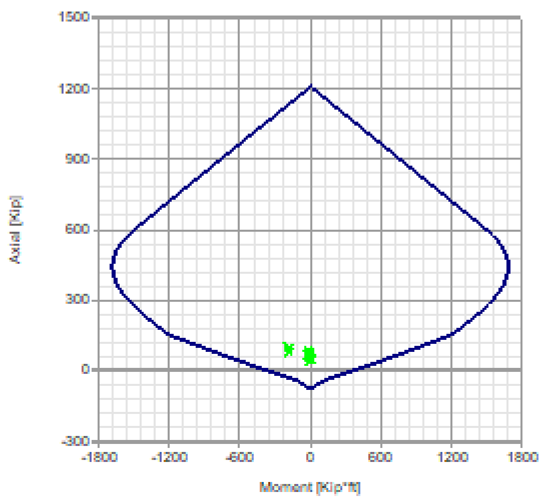


P vs. M (Segment 4)

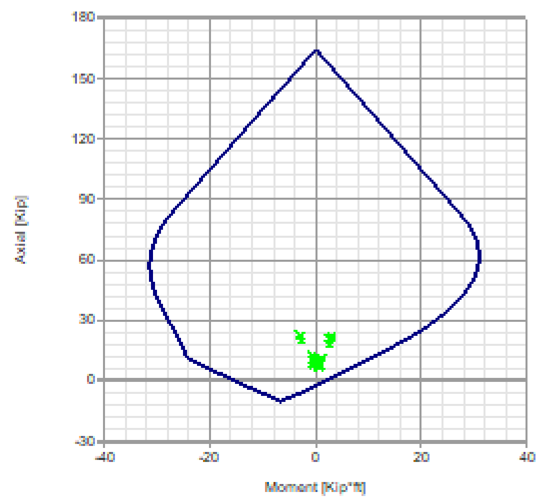
P vs. M (Segment 5)



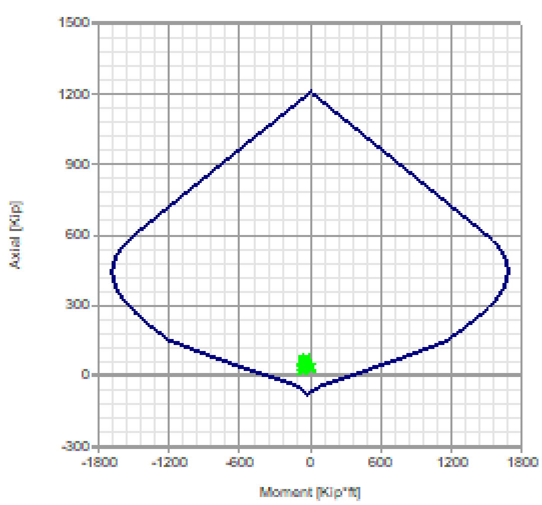
P vs. M (Segment 6)



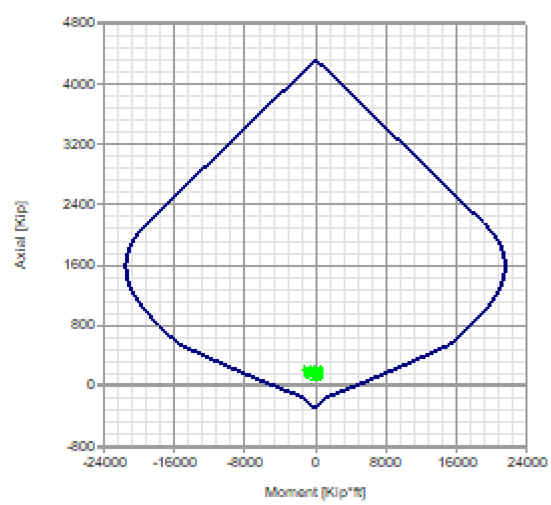
P vs. M (Segment 7)



P vs. M (Segment 8)



P vs. M (Segment 9)



Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio
1	D1(Bottom)	29.24	119.56	0.24
2	D1(Max)	52.99	221.16	0.24
3	D1(Bottom)	36.76	137.42	0.27
4	D1(Max)	26.73	101.60	0.26
5	D1(Bottom)	88.72	439.32	0.20
6	D8(Max)	99.76	439.32	0.23
7	D8(Bottom)	22.50	59.78	0.38
8	D1(Bottom)	73.84	439.32	0.17
9	D1(Bottom)	218.86	1571.84	0.14
10	D1(Bottom)	241.65	2369.84	0.10
11	D1(Bottom)	185.41	2293.02	0.08
12	D1(Max)	28.64	4242.88	0.01

Axial tension

Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio
1	DM1(Top)	0.00	32.00	0.00
2	DM1(Top)	0.00	32.00	0.00
3	DM1(Top)	0.00	32.00	0.00
4	DM1(Top)	0.00	32.00	0.00
5	DM1(Top)	0.00	32.00	0.00
6	DM1(Top)	0.00	32.00	0.00
7	DM1(Top)	0.00	32.00	0.00
8	DM1(Top)	0.00	32.00	0.00
9	DM1(Top)	0.00	32.00	0.00
10	DM1(Top)	0.00	32.00	0.00
11	DM1(Top)	0.00	32.00	0.00
12	DM1(Top)	0.00	32.00	0.00

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio
1	D10(Max)	0.032	0.078	0.42
2	D10(Max)	0.048	0.094	0.52
3	D10(Max)	0.044	0.083	0.53
4	D10(Max)	0.052	0.092	0.57
5	D10(Max)	0.072	0.104	0.70
6	D10(Max)	0.060	0.104	0.58
7	D8(Max)	0.028	0.105	0.26
8	D10(Max)	0.031	0.099	0.31
9	D10(Max)	0.042	0.104	0.40
10	D10(Max)	0.032	0.102	0.31
11	D10(Bottom)	0.015	0.099	0.15
12	D10(Max)	0.001	0.102	0.01

Notes

* P = Axial load
* Pa = Allowable compressive force due to axial load.
* M = Moment at the section under consideration.
* Ma = Wall allowable moment due to axial force or lintel pure flexure allowable moment
* fa = Calculated compressive stress due to axial load only
* fb = Calculated compressive stress due to axial flexure only
* ft = Calculated axial tension
* Fa = Allowable compressive stress due to axial load only
* Fb = Allowable compressive stress due to axial flexure only
* fv = Calculated shear stress
* Fs = Allowable tensile or compressive stress
* Fv = Allowable shear stress
* ld = Embedment length
* As = Effective cross sectional area of reinforcement
* δs = Calculated deflection
* δmax = Maximum allowable deflection

Current Date: 10/15/2025 4:41 PM
Units system: English
File name: C:\Users\allison.valencia\Michael Baker International\196483 Creech DRP Part 2 - General 1\05_Deliverables\07_Design Data\05_Struc\04 Lateral Design\Admin Building\East Shear Walls.msw

Design Results
Masonry wall

General Information

Global status : OK

Design code	:	TMS 402-22 ASD

Materials:

Material	:	CMU 2.0-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Full grouting
Masonry compression strength (F`m)	:	2 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	32 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1800 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC D
Response modification factor	:	1.00
Shear wall type	:	Special

Geometry

Total height	:	39.00 [ft]
Total length	:	121.17 [ft]
Foundation type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	17.33	7.63	0.14
2	17.67	7.63	0.14

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	12.00	0.00	3.30	7.40
Lower left	50.00	0.00	6.30	7.30
Lower left	89.70	0.00	3.30	7.40
Lower left	95.70	0.00	9.20	14.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
Wx	No	WIND	Wind in X
Wz	No	WIND	Wind in Z
EQx	No	EQ	Seismic in X
EQz	No	EQ	Seismic in Z
SM1	Yes		DL
DM1	Yes		DL
D1	Yes		1.4DL
D2	Yes		1.2DL+0.5Wx
D3	Yes		1.2DL+0.5Wz
D4	Yes		1.2DL+Wx
D5	Yes		1.2DL+Wz
D6	Yes		0.9DL+Wx
D7	Yes		0.9DL+Wz
D8	Yes		1.2DL+EQx
D9	Yes		1.2DL+EQz
D10	Yes		0.9DL+EQx
D11	Yes		0.9DL+EQz

Loads

In-plane seismic weight:

Load condition	Coefficient
EQx	0.12

Distributed loads:

Consider self weight : DL

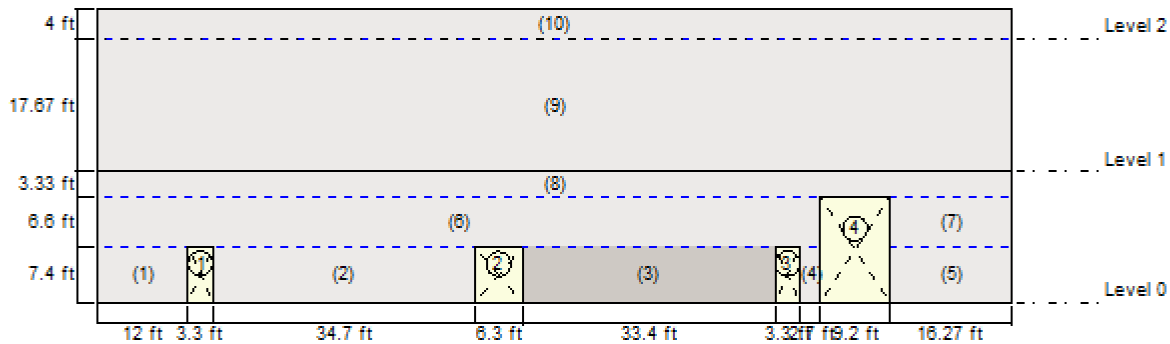
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.43	0.00
2	EQx	Horizontal	0.24	0.00

Out-of-plane seismic weight:

Load condition	Coefficient
EQx	0.12

Shear Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	12.00	7.40
	2	15.30	0.00	34.70	7.40
	3	56.30	0.00	33.40	7.40
	4	93.00	0.00	2.70	7.40
	5	104.90	0.00	16.27	7.40
	6	0.00	7.40	95.70	6.60
	7	104.90	7.40	16.27	6.60
	8	0.00	14.00	121.17	3.33
1	9	0.00	17.33	121.17	17.67
2	10	0.00	35.00	121.17	4.00

Reinforcement

Segment	Vertical reinforcement Bars	Vertical Spacing [in]	Vertical Ld [in]	Horizontal reinforcement Bars	Horizontal Spacing [in]	Horizontal Ld [in]
1	6-#5	24.00	34.07	3-#4	32.00	21.80
2	18-#5	24.00	34.07	3-#4	32.00	21.80
3	17-#5	24.00	34.07	3-#4	32.00	21.80
4	2-#5	24.00	34.07	3-#4	32.00	21.80
5	8-#5	24.00	34.07	3-#4	32.00	21.80
6	6-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	18-#5	24.00	34.07	3-#4	32.00	21.80
	3-#5	24.00	34.07	3-#4	32.00	21.80
	17-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
7	8-#5	24.00	34.07	3-#4	32.00	21.80
8	6-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	18-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	17-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	5-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80

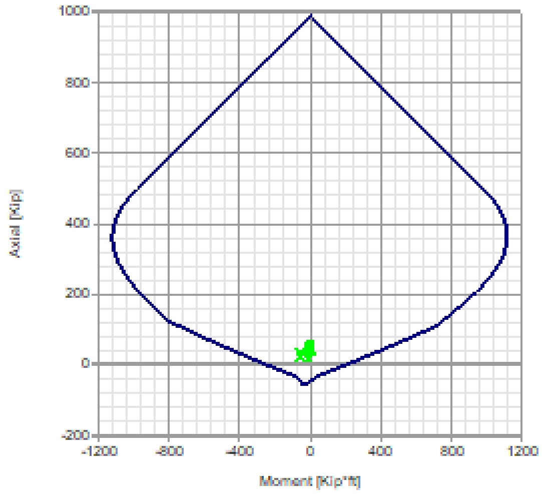
9	6-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	18-#5	24.00	34.07	7-#4	32.00	21.80
	3-#5	24.00	34.07	7-#4	32.00	21.80
	17-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	5-#5	24.00	34.07	7-#4	32.00	21.80
	8-#5	24.00	34.07	7-#4	32.00	21.80
10	6-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	18-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	17-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	5-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80

Combined axial flexure

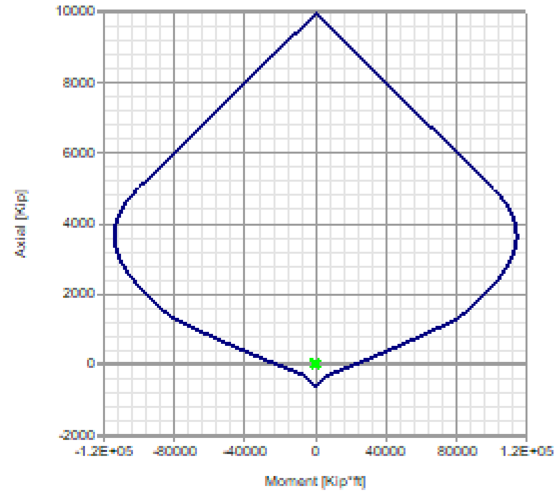
Segment	**Condition**	**P** [Kip]	**M** [Kip*ft]	**Ma** [Kip*ft]	**Ratio**
1	D10(Max)	21.19	-57.35	350.22	0.16
2	D10(Bottom)	106.39	-219.70	3287.47	0.07
3	D10(Bottom)	123.72	-163.40	3321.73	0.05
4	D10(Bottom)	15.05	-2.20	38.13	0.06
5	D10(Bottom)	70.41	-130.80	881.54	0.15
6	D10(Max)	216.54	-1385.34	21822.28	0.06
7	D8(Top)	58.09	60.41	738.93	0.08
8	D10(Bottom)	232.83	-1292.29	32926.67	0.04
9	D10(Bottom)	182.60	-807.74	30544.11	0.03
10	D8(Top)	12.37	11.24	22478.61	0.00

Interaction diagrams, P vs. M

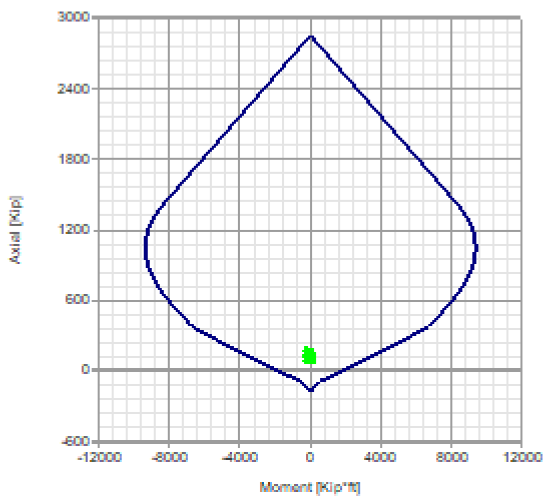
P vs. M (Segment 1)



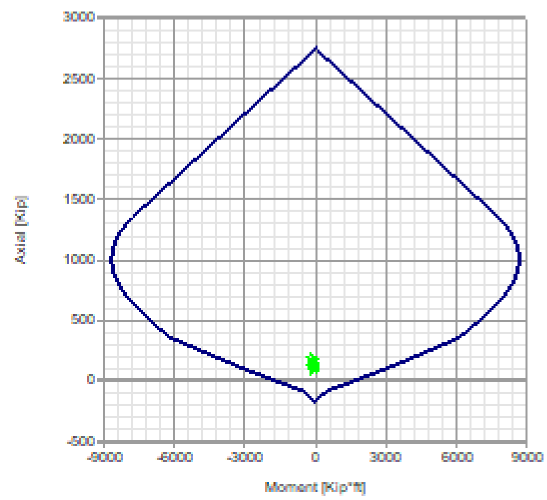
P vs. M (Segment 10)



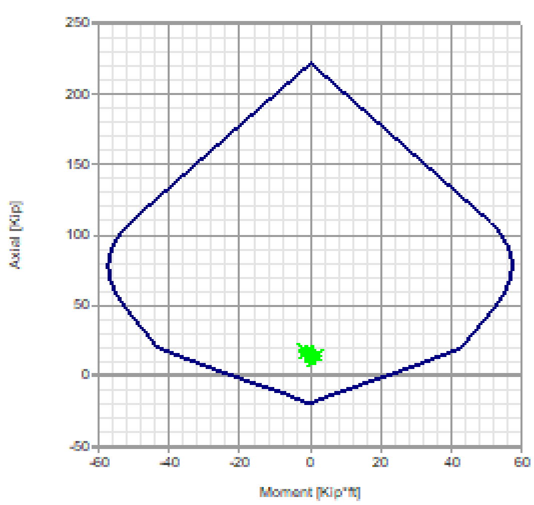
P vs. M (Segment 2)



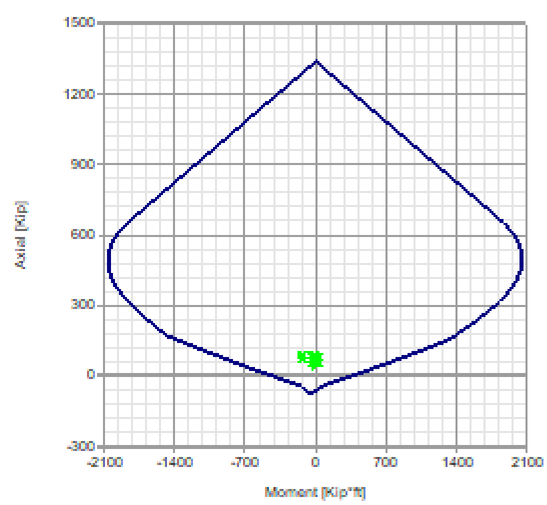
P vs. M (Segment 3)

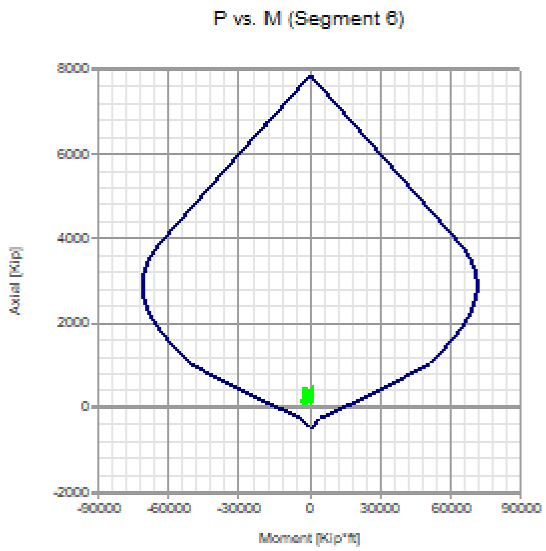


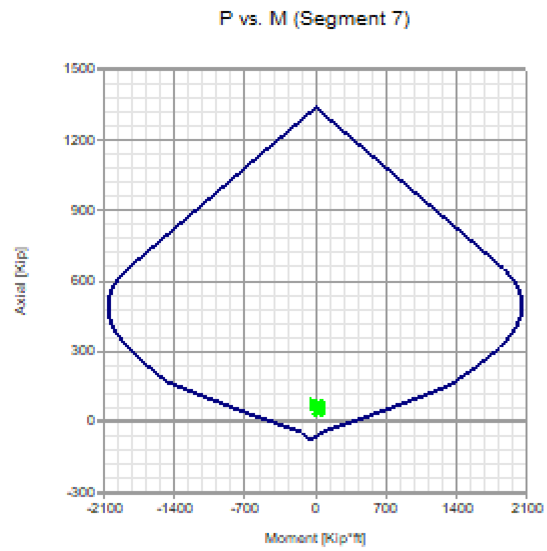
P vs. M (Segment 4)

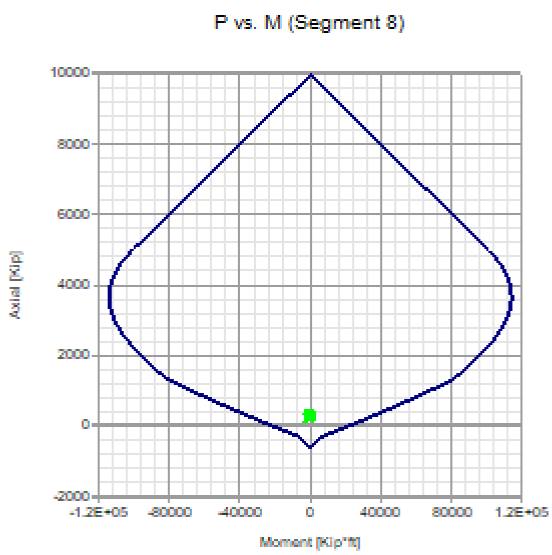


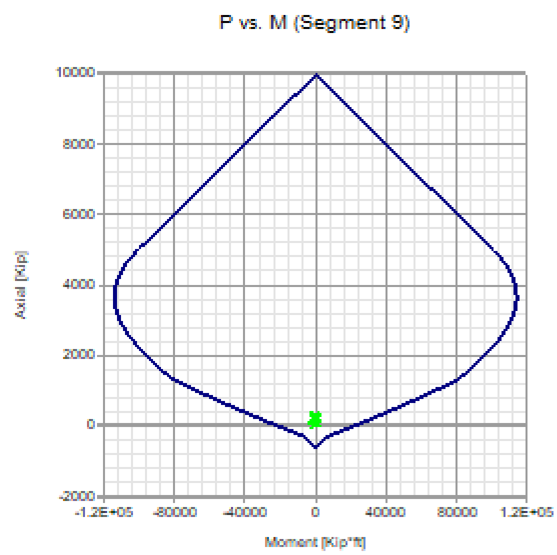
P vs. M (Segment 5)











Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio
1	D1(Bottom)	61.00	358.68	0.17
2	D1(Max)	175.39	1037.12	0.17
3	D1(Bottom)	183.77	998.29	0.18
4	D8(Max)	19.61	80.64	0.24
5	D8(Max)	89.17	486.32	0.18
6	D1(Bottom)	396.39	2860.25	0.14
7	D8(Bottom)	83.34	486.32	0.17
8	D1(Bottom)	360.66	3621.52	0.10
9	D1(Max)	283.58	3504.14	0.08
10	D1(Max)	43.87	6483.87	0.01

Axial tension

Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio
1	DM1(Top)	0.00	32.00	0.00
2	DM1(Top)	0.00	32.00	0.00
3	DM1(Top)	0.00	32.00	0.00
4	DM1(Top)	0.00	32.00	0.00
5	DM1(Top)	0.00	32.00	0.00
6	DM1(Top)	0.00	32.00	0.00
7	DM1(Top)	0.00	32.00	0.00
8	DM1(Top)	0.00	32.00	0.00
9	DM1(Top)	0.00	32.00	0.00
10	DM1(Top)	0.00	32.00	0.00

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio
1	D10(Max)	0.016	0.085	0.19
2	D10(Max)	0.028	0.104	0.27
3	D10(Bottom)	0.034	0.107	0.32
4	D10(Max)	0.029	0.111	0.26
5	D8(Bottom)	0.032	0.101	0.31
6	D10(Max)	0.023	0.102	0.22
7	D8(Max)	0.031	0.108	0.29
8	D10(Top)	0.020	0.104	0.19
9	D10(Max)	0.010	0.102	0.09
10	D10(Max)	0.000	0.102	0.00

Notes

* P = Axial load
* Pa = Allowable compressive force due to axial load.
* M = Moment at the section under consideration.
* Ma = Wall allowable moment due to axial force or lintel pure flexure allowable moment
* fa = Calculated compressive stress due to axial load only
* fb = Calculated compressive stress due to axial flexure only
* ft = Calculated axial tension
* Fa = Allowable compressive stress due to axial load only
* Fb = Allowable compressive stress due to axial flexure only
* fv = Calculated shear stress
* Fs = Allowable tensile or compressive stress
* Fv = Allowable shear stress
* ld = Embedment length
* As = Effective cross sectional area of reinforcement
* δs = Calculated deflection
* δmax = Maximum allowable deflection

Current Date: 10/15/2025 4:38 PM
Units system: English
File name: C:\Users\allison.valencia\Michael Baker International\196483 Creech DRP Part 2 - General 1\05_Deliverables\07_Design Data\05_Struc\04 Lateral Design\Admin Building\West Shear Walls.msw

Design Results
Masonry wall

General Information

Global status : OK

Design code	:	TMS 402-22 ASD

Materials:

Material	:	CMU 2.0-60
Mortar type	:	Port/Mort - M/S
Grouting type	:	Full grouting
Masonry compression strength (F`m)	:	2 [Kip/in2]
Steel tension strength (fy)	:	60 [Kip/in2]
Steel allowable tension strength (Fs)	:	32 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Masonry elasticity modulus (Em)	:	1800 [Kip/in2]
Masonry unit weight	:	0.135 [Kip/ft3]

Seismic data:

Seismic design category	:	SDC D
Response modification factor	:	1.00
Shear wall type	:	Special

Geometry

Total height	:	39.00 [ft]
Total length	:	121.17 [ft]
Foundation type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	None

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]	Effective unit weight [Kip/ft3]
1	17.33	7.63	0.14
2	17.67	7.63	0.14

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	12.00	0.00	3.30	7.40
Lower left	50.00	0.00	6.30	7.30
Lower left	89.70	0.00	3.30	7.40
Lower left	95.70	0.00	9.20	14.00

Load Conditions

ID	Comb.	Category	Description
DL	No	DL	Dead Load
Wx	No	WIND	Wind in X
Wz	No	WIND	Wind in Z
EQx	No	EQ	Seismic in X
EQz	No	EQ	Seismic in Z
SM1	Yes		DL
DM1	Yes		DL
D1	Yes		1.4DL
D2	Yes		1.2DL+0.5Wx
D3	Yes		1.2DL+0.5Wz
D4	Yes		1.2DL+Wx
D5	Yes		1.2DL+Wz
D6	Yes		0.9DL+Wx
D7	Yes		0.9DL+Wz
D8	Yes		1.2DL+EQx
D9	Yes		1.2DL+EQz
D10	Yes		0.9DL+EQx
D11	Yes		0.9DL+EQz

Loads

In-plane seismic weight:

Load condition	Coefficient
EQx	0.12

Distributed loads:

Consider self weight : DL

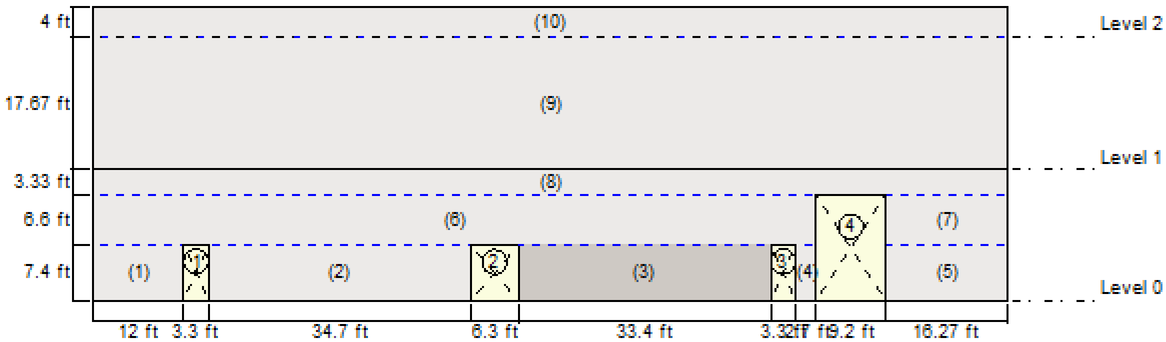
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.43	0.00
2	EQx	Horizontal	0.24	0.00

Out-of-plane seismic weight:

Load condition	Coefficient
EQx	0.12

Shear Wall Design

Status : OK



Geometry

Level	Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
0	1	0.00	0.00	12.00	7.40
	2	15.30	0.00	34.70	7.40
	3	56.30	0.00	33.40	7.40
	4	93.00	0.00	2.70	7.40
	5	104.90	0.00	16.27	7.40
	6	0.00	7.40	95.70	6.60
	7	104.90	7.40	16.27	6.60
	8	0.00	14.00	121.17	3.33
1	9	0.00	17.33	121.17	17.67
2	10	0.00	35.00	121.17	4.00

Reinforcement

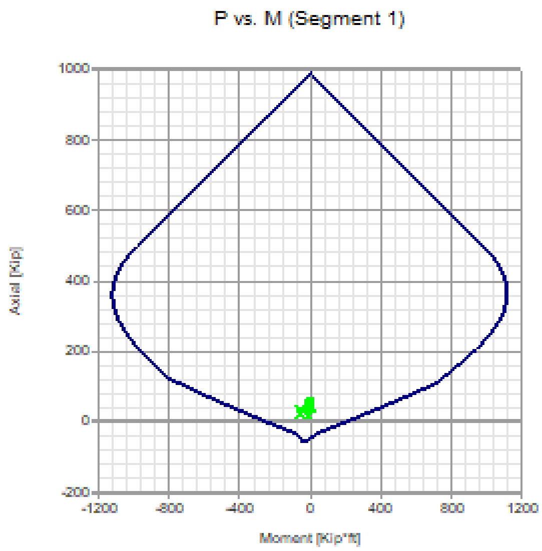
Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	6-#5	24.00	34.07	3-#4	32.00	21.80
2	18-#5	24.00	34.07	3-#4	32.00	21.80
3	17-#5	24.00	34.07	3-#4	32.00	21.80
4	2-#5	24.00	34.07	3-#4	32.00	21.80
5	8-#5	24.00	34.07	3-#4	32.00	21.80
6	6-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	18-#5	24.00	34.07	3-#4	32.00	21.80
	3-#5	24.00	34.07	3-#4	32.00	21.80
	17-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
	2-#5	24.00	34.07	3-#4	32.00	21.80
7	8-#5	24.00	34.07	3-#4	32.00	21.80
8	6-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	18-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	17-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	5-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80

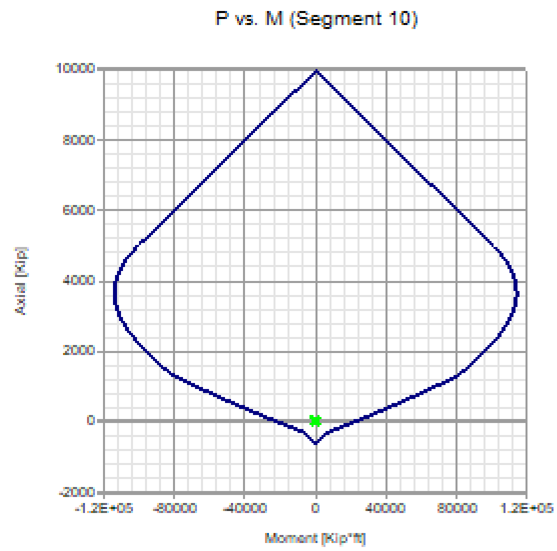
9	6-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	18-#5	24.00	34.07	7-#4	32.00	21.80
	3-#5	24.00	34.07	7-#4	32.00	21.80
	17-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	5-#5	24.00	34.07	7-#4	32.00	21.80
	8-#5	24.00	34.07	7-#4	32.00	21.80
10	6-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	18-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	17-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	5-#5	24.00	34.07	2-#4	32.00	21.80
	8-#5	24.00	34.07	2-#4	32.00	21.80

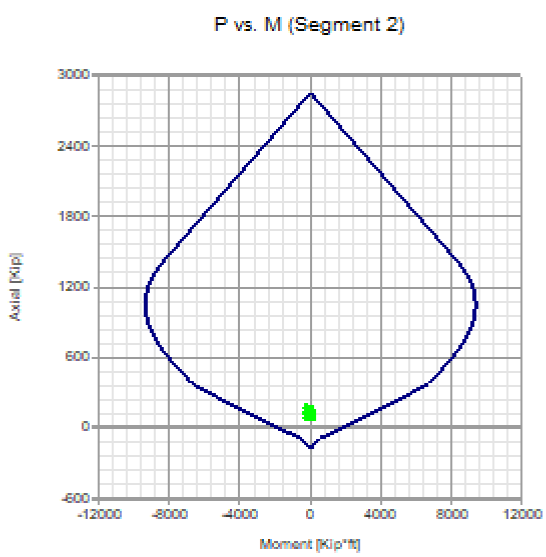
Combined axial flexure

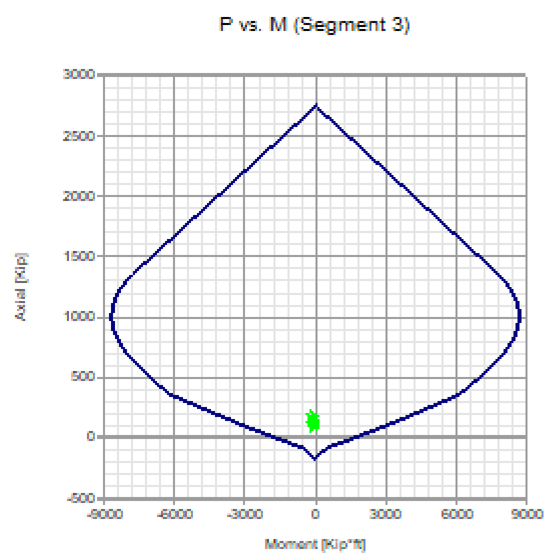
Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D10(Max)	21.19	-57.35	350.22	0.16
2	D10(Bottom)	106.39	-219.70	3287.47	0.07
3	D10(Bottom)	123.72	-163.40	3321.73	0.05
4	D10(Bottom)	15.05	-2.20	38.13	0.06
5	D10(Bottom)	70.41	-130.80	881.54	0.15
6	D10(Max)	216.54	-1385.34	21822.28	0.06
7	D8(Top)	58.09	60.41	738.93	0.08
8	D10(Bottom)	232.83	-1292.29	32926.67	0.04
9	D10(Bottom)	182.60	-807.74	30544.11	0.03
10	D8(Top)	12.37	11.24	22478.61	0.00

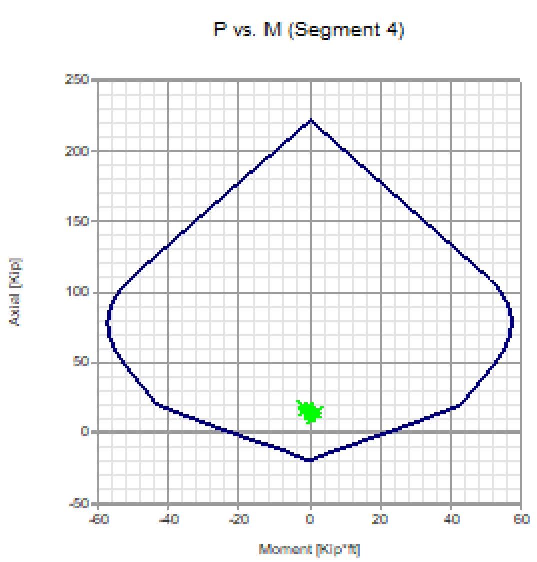
Interaction diagrams, P vs. M

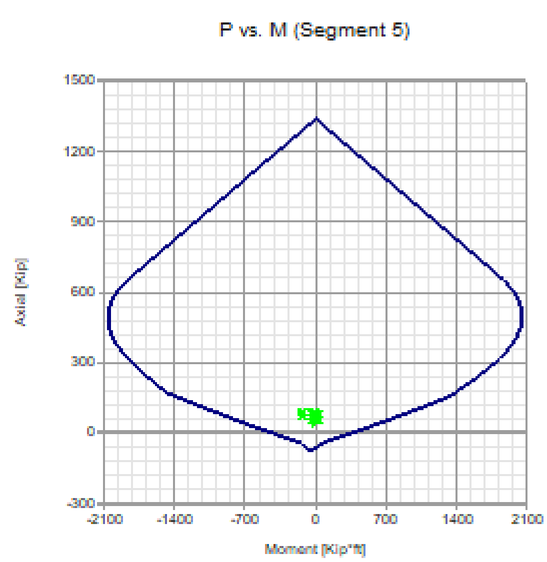

P vs. M (Segment 1)
Axial [Kip]
Moment [Kip*ft]
1000
800
600
400
200
0
-200
-1200
-800
-400
0
400
800
1200

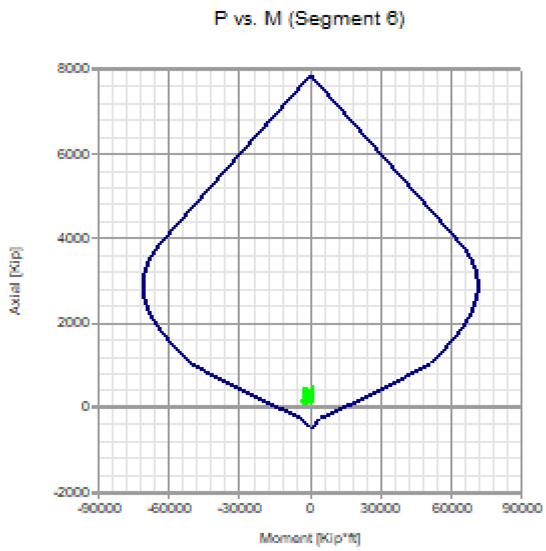

P vs. M (Segment 10)
Axial [Kip]
Moment [Kip*ft]
10000
8000
6000
4000
2000
0
-2000
-1.2E+05
-80000
-40000
0
40000
80000
1.2E+05

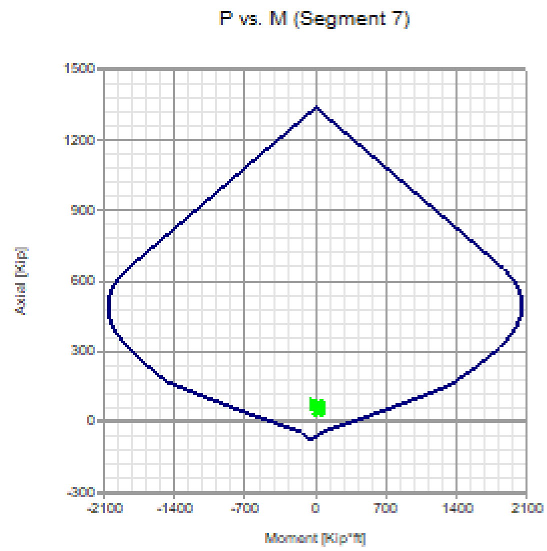

P vs. M (Segment 2)
Axial [Kip]
Moment [Kip*ft]
3000
2400
1800
1200
600
0
-600
-12000
-8000
-4000
0
4000
8000
12000

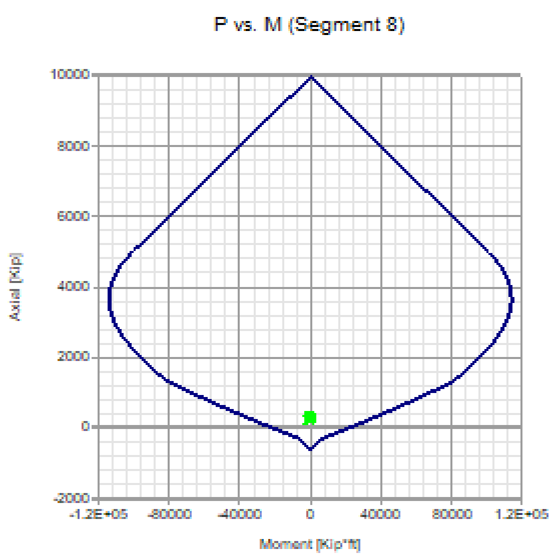

P vs. M (Segment 3)
Axial [Kip]
Moment [Kip*ft]
3000
2500
2000
1500
1000
500
0
-500
-9000
-6000
-3000
0
3000
6000
9000

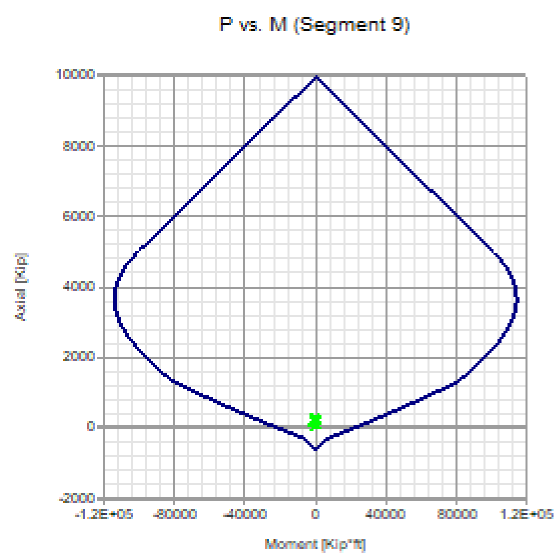

P vs. M (Segment 4)
Axial [Kip]
Moment [Kip*ft]
250
200
150
100
50
0
-50
-60
-40
-20
0
20
40
60


P vs. M (Segment 5)
Axial [Kip]
Moment [Kip*ft]
1500
1200
900
600
300
0
-300
-2100
-1400
-700
0
700
1400
2100









Axial compression

Segment	Condition	P [Kip]	Pa [Kip]	Ratio
1	D1(Bottom)	61.00	358.68	0.17
2	D1(Max)	175.39	1037.12	0.17
3	D1(Bottom)	183.77	998.29	0.18
4	D8(Max)	19.61	80.64	0.24
5	D8(Max)	89.17	486.32	0.18
6	D1(Bottom)	396.39	2860.25	0.14
7	D8(Bottom)	83.34	486.32	0.17
8	D1(Bottom)	360.66	3621.52	0.10
9	D1(Max)	283.58	3504.14	0.08
10	D1(Max)	43.87	6483.87	0.01

Axial tension

Segment	Condition	ft [Kip/in2]	Fs [Kip/in2]	Ratio
1	DM1(Top)	0.00	32.00	0.00
2	DM1(Top)	0.00	32.00	0.00
3	DM1(Top)	0.00	32.00	0.00
4	DM1(Top)	0.00	32.00	0.00
5	DM1(Top)	0.00	32.00	0.00
6	DM1(Top)	0.00	32.00	0.00
7	DM1(Top)	0.00	32.00	0.00
8	DM1(Top)	0.00	32.00	0.00
9	DM1(Top)	0.00	32.00	0.00
10	DM1(Top)	0.00	32.00	0.00

Shear

Segment	Condition	fv [Kip/in2]	Fv [Kip/in2]	Ratio
1	D10(Max)	0.016	0.085	0.19
2	D10(Max)	0.028	0.104	0.27
3	D10(Bottom)	0.034	0.107	0.32
4	D10(Max)	0.029	0.111	0.26
5	D8(Bottom)	0.032	0.101	0.31
6	D10(Max)	0.023	0.102	0.22
7	D8(Max)	0.031	0.108	0.29
8	D10(Top)	0.020	0.104	0.19
9	D10(Max)	0.010	0.102	0.09
10	D10(Max)	0.000	0.102	0.00

Notes

* P = Axial load
* Pa = Allowable compressive force due to axial load.
* M = Moment at the section under consideration.
* Ma = Wall allowable moment due to axial force or lintel pure flexure allowable moment
* fa = Calculated compressive stress due to axial load only
* fb = Calculated compressive stress due to axial flexure only
* ft = Calculated axial tension
* Fa = Allowable compressive stress due to axial load only
* Fb = Allowable compressive stress due to axial flexure only
* fv = Calculated shear stress
* Fs = Allowable tensile or compressive stress
* Fv = Allowable shear stress
* ld = Embedment length
* As = Effective cross sectional area of reinforcement
* δs = Calculated deflection
* δmax = Maximum allowable deflection

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Creech DRP PH 2	Date:	10/16/2025
Fastening point:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**HY 100 + threaded rod 5.8 5/8**
Item number:	385428 HAS 5.8 5/8"x8" (element) / 2078494 HIT-HY 100 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 100 + threaded rod 5.8 with 6 in nominal embedment depth per IAPMO ER-547 , Hammer drilled installation per MPII,
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	5.8
Evaluation Service Report:	ER-547
Issued I Valid:	6/2/2025 | 5/31/2026
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-19
Stand-off installation:	
Profile:	
Base material:	uncracked Grout-filled CMU, f=2000, f = 1,999 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Solid Head Joint: yes; open ended unit: yes
Temp. short/long:	32 °F / 32 °F
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Face installation, Drill hole: Hammer drilled, Installation condition: Dry

Wall Anchorage Calculation

$$F_p = 0.4 S_{DS} k_a I_e W_p \qquad (12.11\text{-}1)$$

Second floor Anchor

Ie = 1.25

ka = 1.0

Sds = 0.527

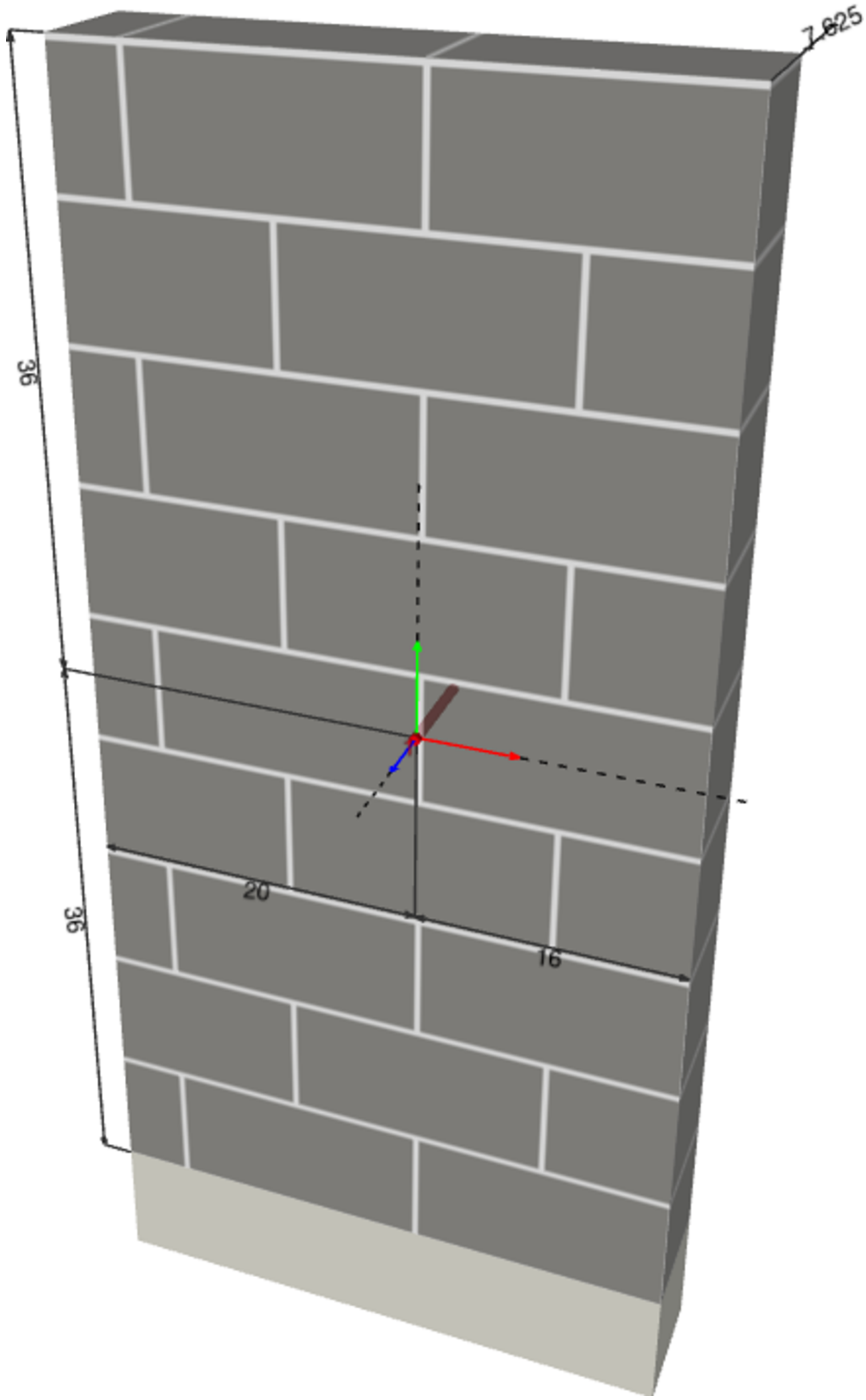
Wp = 128psf x 17.33 x 24/12 = 4436.5lbs

Fp = 1170lbs

Fp min = 0.2 x 1.07 x 1.25 x 4436.5 = 1186.6lbs (USE)

Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Creech DRP PH 2	Date:	10/16/2025
Fastening point:			

Geometry [in.]



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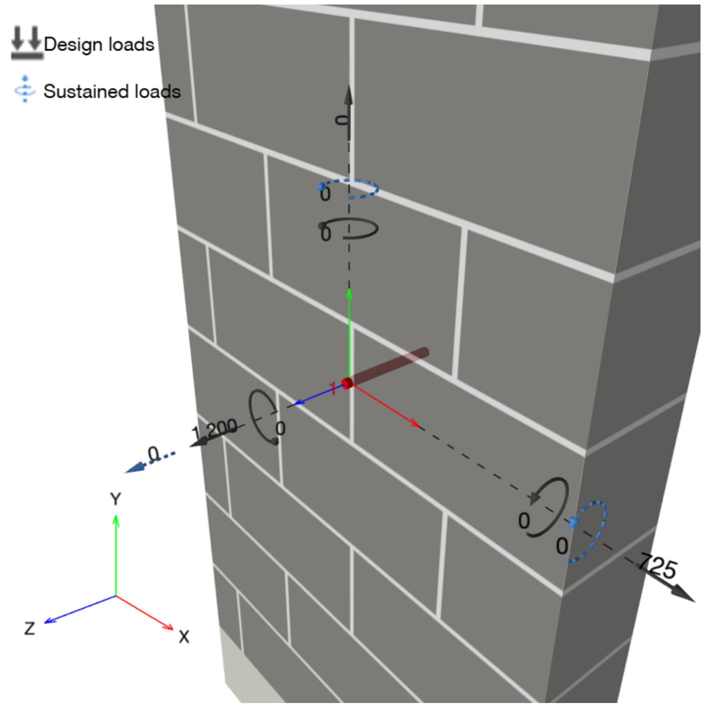
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 1,200; V_x = 725; V_y = 0;$ $M_x = 0; M_y = 0; M_z = 0;$ $N_{sus} = 0; M_{x,sus} = 0; M_{y,sus} = 0;$	no	40

2 Load case/Resulting anchor forces

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,200	725	725	0

3 Tension load

	Load [lb]	Capacity [lb]	Utilization β_N [%]	Status
Steel	1,200	10,650	12	OK
Bond	1,200	3,014	40	OK
Masonry breakout	1,200	7,260	17	OK

3.1 Steel strength

N_{sa} = ESR value refer to ICC-ES ER-547

$\phi\ N_{sa} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

N_{sa} [lb]	ϕ	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
16,384	0.650	10,650	1,200

3.2 Bond strength

$N_{ma} = \left(\frac{A_{Na}}{A_{Na0}}\right) \cdot \psi_{ed,Na} \cdot N_{ba,m}$ ACI 318-19 Eq. (17.6.5.1a)

$\phi\ N_{ma} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-19 Table D.4.1.1

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.00$ ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{uncr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]	α_{sat}	α_{top}
0.625	6.000	16.000	465	1.000	1.000

λ_a
1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
4.046	65.48	65.48	1.000

$N_{ba,m}$ [lb]
5,480

Results

N_{ma} [lb]	ϕ	$\phi\ N_{ma}$ [lb]	$\phi\ N_{ua}$ [lb]
5,480	0.550	3,014	1,200

3.3 Masonry breakout strength

$N_{mb} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$	ACI 318-19 Eq. (17.6.2.1a)
$\phi\ N_{mb} \geq N_{ua}$	AC58 Table 3.2 + ACI 318-19 Table 17.5.2
A_{Nm} see ACI 318-19, Section 17.4.2.1, Fig. R 17.4.2.1(b)	
$A_{Nm0} = 9 \cdot h_{ef}^2$	ACI 318-19 Eq. (17.6.2.2.1)
$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \cdot h_{ef}} \leq 1.0$	ACI 318-19 Eq. (17.6.2.4.1b)
$N_{b,m} = k_m \cdot \lambda_a \cdot \sqrt{f'_m} \cdot h_{ef}^{1.5}$	ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N,m}$	k_m
6.000	16.000	1.000	17

λ_a	f'_m [psi]
1.000	1,999

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ed,N,m}$	$N_{b,m}$ [lb]
324.00	324.00	1.000	11,170

Results

N_{mb} [lb]	ϕ	$\phi\ N_{mb}$ [lb]	N_{ua} [lb]
11,170	0.650	7,260	1,200

4 Shear load

	Load [lb]	Capacity [lb]	Utilization β_V [%]	Status
Steel	725	5,898	13	OK
Pryout bond	725	7,672	10	OK
Masonry crushing strength	725	4,034	18	OK
Masonry breakout in direction x+	725	13,257	6	OK

4.1 Steel strength

V_{sa} = ESR value refer to ICC-ES ER-547

$\phi\ V_{sa} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

V_{sa} [lb]	ϕ	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
9,830	0.600	5,898	725

4.2 Pryout strength (Bond strength controls)

$V_{mpg} = \min(k_{cp} \cdot N_{mag}, k_{cp} \cdot N_{mbg})$

$\phi\ V_{mpg} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e_N'}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{uncr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

k_{cp}	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]
2	0.625	6.000	16.000	465

α_{sat}	α_{top}	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a
1.000	1.000	0.000	0.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$	$\psi_{ec1,Na}$	$\psi_{ec2,Na}$
4.046	65.48	65.48	1.000	1.000	1.000

$\psi_{cp,Na}$	$N_{ba,m}$ [lb]
1.000	5,480

Results

V_{mpg} [lb]	ϕ	$\phi\ V_{mpg}$ [lb]	V_{ua} [lb]
10,960	0.700	7,672	725

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4.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot \left(f'_m \cdot A_{se,V}\right)^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,v}$ [in.^{2}]
1,999	0.23

Results

V_{mc} [lb]	ϕ	$\phi\ V_{mc}$ [lb]	V_{ua} [lb]
8,068	0.500	4,034	725

4.4 Masonry breakout strength x+

$V_{mb} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \psi_{m,V}\ \psi_{h,V,m}\ \psi_{parallel,V}\ V_{b,m}$ ACI 318-19 Eq. (17.7.2.1a)

$\phi\ V_{mb} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Vm} see ACI 318-19 , Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vm0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V,m} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V,m} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_{b,m} = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\ \sqrt{f'_m}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
5.000	0.625	16.000	36.000	366.00	1,152.00	1,999

Calculations

$\psi_{ed,V,m}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{m,V}$	$\psi_{h,V,m}$
1.000	1.000	0.000	1.400	1.774

Results

$V_{b,m}$ [lb]	ϕ	$\phi\ V_{mbg}$ [lb]	V_{ua} [lb]
23,999	0.700	13,257	725

5 Required verifications under combined tension and shear forces

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.398	0.180	5/3	28	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

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6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

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7 Installation data

Anchor plate, steel: -

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Drilled in hammer mode

Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HY 100 + threaded rod 5.8 5/8

Item number: 385428 HAS 5.8 5/8"x8" (element) / 2078494 HIT-HY 100 (adhesive)

Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 100 + threaded rod 5.8 with 6 in nominal embedment depth per IAPMO ER-547 , Hammer drilled installation per MPII

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	20.000	16.000	36.000	36.000

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8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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Specifier's comments:

1 Input data

Anchor type and diameter:	**HY 100 + threaded rod 5.8 5/8**
Item number:	385428 HAS 5.8 5/8"x8" (element) / 2078494 HIT-HY 100 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 100 + threaded rod 5.8 with 6 in nominal embedment depth per IAPMO ER-547 , Hammer drilled installation per MPII,
Effective embedment depth:	h_{ef} = 6.000 in.
Material:	5.8
Evaluation Service Report:	ER-547
Issued I Valid:	6/2/2025 | 5/31/2026
Proof:	Design Method LRFD (AC58) Masonry + ACI 318-19
Stand-off installation:	
Profile:	
Base material:	uncracked Grout-filled CMU, f=2000, f = 1,999 psi, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Solid Head Joint: yes; open ended unit: yes
Temp. short/long:	32 °F / 32 °F
	Joints: vertical: 0.375 in.; horizontal: 0.375 in.
Installation:	Face installation, Drill hole: Hammer drilled, Installation condition: Dry

Wall Anchorage Calculation

$$F_p = 0.4 S_{DS} k_a I_e W_p \qquad (12.11\text{-}1)$$

Roof Anchor
Ie = 1.25
ka = 1.0 + 7/100 = 1.07
Sds = 0.527
Wp = 128psf x 17.33/2 x 48/12 = 4436.5 lbs

Fp = 1250lbs

Fp min = 0.2 x 1.07 x 1.25 x 4436.5 = 1186.6lbs

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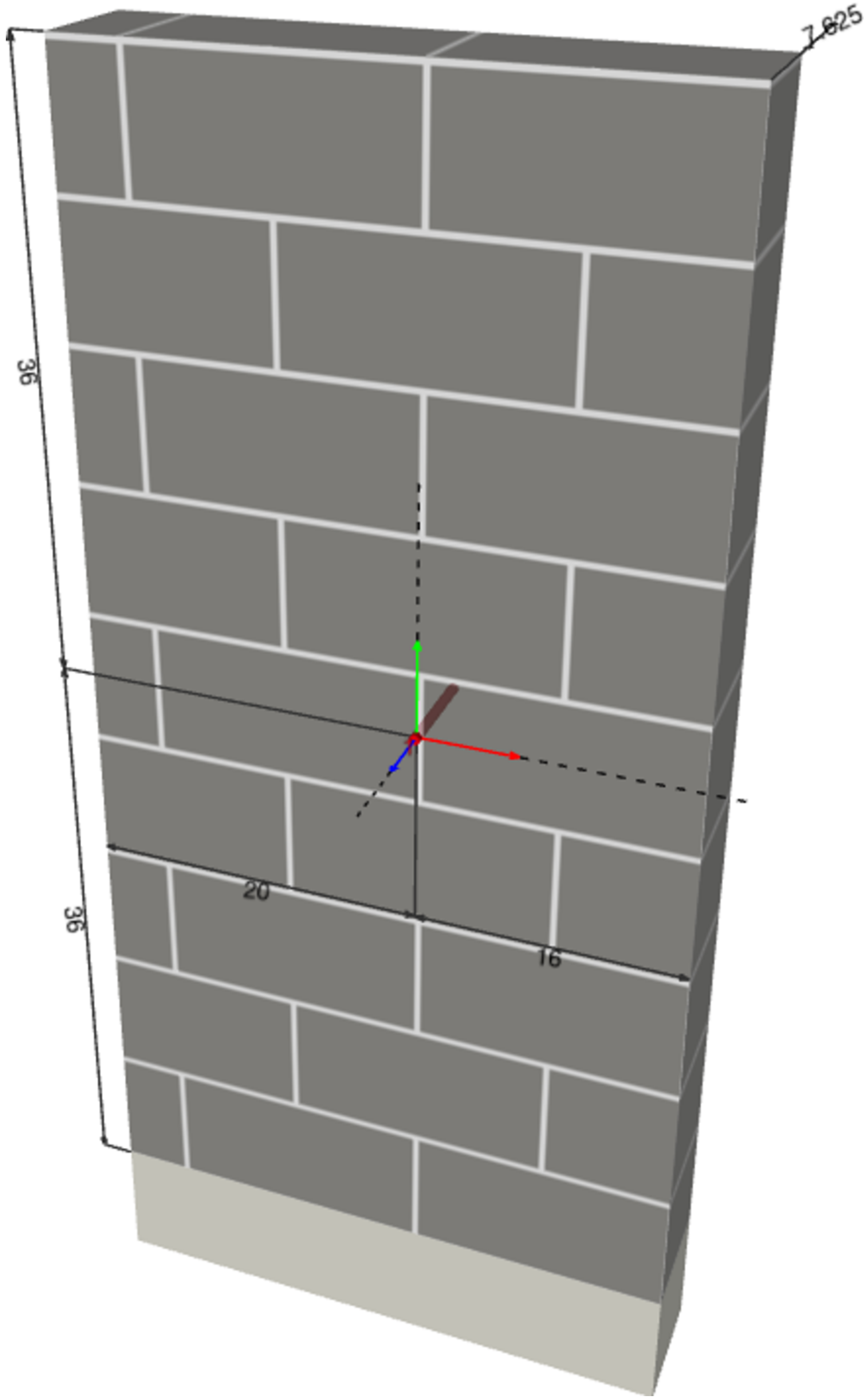
Geometry [in.]



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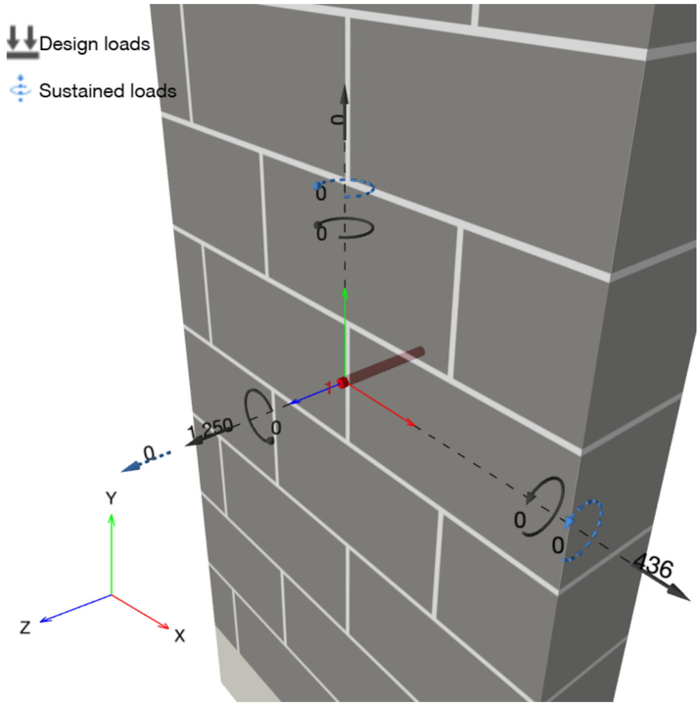
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 1{,}250$; $V_x = 436$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$; $N_{sus} = 0$; $M_{x,sus} = 0$; $M_{y,sus} = 0$;	no	42

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,250	436	436	0

3 Tension load

	Load [lb]	Capacity [lb]	Utilization β_N [%]	Status
Steel	1,250	10,650	12	OK
Bond	1,250	3,014	42	OK
Masonry breakout	1,250	7,260	18	OK

3.1 Steel strength

N_{sa} = ESR value refer to ICC-ES ER-547

$\phi\ N_{sa} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

N_{sa} [lb]	ϕ	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
16,384	0.650	10,650	1,250

3.2 Bond strength

$N_{ma} = \left(\frac{A_{Na}}{A_{Na0}}\right) \cdot \psi_{ed,Na} \cdot N_{ba,m}$ ACI 318-19 Eq. (17.6.5.1a)

$\phi\ N_{ma} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-19 Table D.4.1.1

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.00$ ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{uncr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]	α_{sat}	α_{top}
0.625	6.000	16.000	465	1.000	1.000

λ_a
1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
4.046	65.48	65.48	1.000

$N_{ba,m}$ [lb]
5,480

Results

N_{ma} [lb]	ϕ	$\phi\ N_{ma}$ [lb]	$\phi\ N_{ua}$ [lb]
5,480	0.550	3,014	1,250

3.3 Masonry breakout strength

$N_{mb} = \frac{A_{Nm}}{A_{Nm0}} \cdot \psi_{ed,N,m} \cdot \psi_{c,N,m} \cdot N_{b,m}$ ACI 318-19 Eq. (17.6.2.1a)

$\phi\ N_{mb} \geq N_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Nm} see ACI 318-19, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nm0} = 9 \cdot h_{ef}^2$ ACI 318-19 Eq. (17.6.2.2.1)

$\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \cdot h_{ef}} \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$N_{b,m} = k_m \cdot \lambda_a \cdot \sqrt{f'_m} \cdot h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N,m}$	k_m
6.000	16.000	1.000	17

λ_a	f'_m [psi]
1.000	1,999

Calculations

A_{Nm} [in.^{2}]	A_{Nm0} [in.^{2}]	$\psi_{ed,N,m}$	$N_{b,m}$ [lb]
324.00	324.00	1.000	11,170

Results

N_{mb} [lb]	ϕ	$\phi\ N_{mb}$ [lb]	N_{ua} [lb]
11,170	0.650	7,260	1,250

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4 Shear load

	Load [lb]	Capacity [lb]	Utilization β_V [%]	Status
Steel	436	5,898	8	OK
Pryout bond	436	7,672	6	OK
Masonry crushing strength	436	4,034	11	OK
Masonry breakout in direction x+	436	13,257	4	OK

4.1 Steel strength

V_{sa} = ESR value refer to ICC-ES ER-547

$\phi\ V_{sa} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

V_{sa} [lb]	ϕ	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
9,830	0.600	5,898	436

4.2 Pryout strength (Bond strength controls)

$V_{mpg} = \min(k_{cp} \cdot N_{mag}, k_{cp} \cdot N_{mbg})$

$\phi\ V_{mpg} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr,m}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$N_{ba,m} = \lambda_a \cdot \tau_{uncr,m} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

k_{cp}	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\tau_{uncr,m}$ [psi]
2	0.625	6.000	16.000	465

α_{sat}	α_{top}	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	λ_a
1.000	1.000	0.000	0.000	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$	$\psi_{ec1,Na}$	$\psi_{ec2,Na}$
4.046	65.48	65.48	1.000	1.000	1.000

$\psi_{cp,Na}$	$N_{ba,m}$ [lb]
1.000	5,480

Results

V_{mpg} [lb]	ϕ	$\phi\ V_{mpg}$ [lb]	V_{ua} [lb]
10,960	0.700	7,672	436

4.3 Masonry crushing strength

$\phi\ V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot \left(f'_m \cdot A_{se,V}\right)^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,v}$ [in.^{2}]
1,999	0.23

Results

V_{mc} [lb]	ϕ	$\phi\ V_{mc}$ [lb]	V_{ua} [lb]
8,068	0.500	4,034	436

4.4 Masonry breakout strength x+

$V_{mb} = \left(\frac{A_{Vm}}{A_{Vm0}}\right) \psi_{m,V}\ \psi_{h,V,m}\ \psi_{parallel,V}\ V_{b,m}$ ACI 318-19 Eq. (17.7.2.1a)

$\phi\ V_{mb} \geq V_{ua}$ AC58 Table 3.2 + ACI 318-19 Table 17.5.2

A_{Vm} see ACI 318-19 , Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vm0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ed,V,m} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V,m} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_{b,m} = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_m}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

l_e [in.]	d_a [in.]	c_{a1} [in.]	c_{a2} [in.]	A_{Vm} [in.^{2}]	A_{Vm0} [in.^{2}]	f'_m [psi]
5.000	0.625	16.000	36.000	366.00	1,152.00	1,999

Calculations

$\psi_{ed,V,m}$	$\psi_{parallel,V}$	$e_{c,V}$ [in.]	$\psi_{m,V}$	$\psi_{h,V,m}$
1.000	1.000	0.000	1.400	1.774

Results

$V_{b,m}$ [lb]	ϕ	$\phi\ V_{mbg}$ [lb]	V_{ua} [lb]
23,999	0.700	13,257	436

5 Required verifications under combined tension and shear forces

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.415	0.108	5/3	26	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Creech DRP PH 2	Date:	10/16/2025
Fastening point:			

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry should be built according to industry standards.

Fastening meets the design criteria!

7 Installation data

Anchor plate, steel: -

Profile: -

Hole diameter in the fixture: -

Plate thickness (input): -

Drilling method: Drilled in hammer mode

Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: HY 100 + threaded rod 5.8 5/8

Item number: 385428 HAS 5.8 5/8"x8" (element) / 2078494 HIT-HY 100 (adhesive)

Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 6.000 in.

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 100 + threaded rod 5.8 with 6 in nominal embedment depth per IAPMO ER-547 , Hammer drilled installation per MPII

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	20.000	16.000	36.000	36.000

8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Foundation Calculations

<u>FoundationReactions</u>

FoundationReactions, First-order linear, Local SupportAxis System

Supports

Source	Support	Coordinates [ft, in] X	Y	Z	Reference	Size	Result rotation [°]	Loadcase	Reactions F_x [kip]	F_y [kip]	F_z [kip]	M_x [kip ft]	M_y [kip ft]	M_z [kip ft]
SC A.1/#487	SUP A.1/#487	14′ 8″	2′ 6″	100′ 0″	SC A.1/#487	HSS 6x6x1/4	0.0000	1 Self weight - excluding slabs	0.0	0.0	1.4	0.0	0.0	-0.0
								2 Slab self weight	0.0	0.0	6.3	0.0	0.0	0.0
								3 Dead	0.0	0.0	13.0	0.0	0.0	0.0
								4 Live	0.0	0.0	13.0	0.0	0.0	-0.0
SC A.1/P	SUP A.1/P	30′ 8″	2′ 6″	100′ 0″	SC A.1/P	HSS 6x6x1/4	0.0000	1 Self weight - excluding slabs	-0.0	0.0	1.4	0.0	0.0	-0.0
								2 Slab self weight	-0.0	0.0	5.9	0.0	0.0	0.0
								3 Dead	-0.0	0.0	12.2	0.0	0.0	0.0
								4 Live	-0.0	0.0	12.2	0.0	0.0	-0.0
SC A.9/4.1	SUP A.9/4.1	3′ 8″	32′ 2″	100′ 0″	SC A.9/4.1	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	-0.0	8.6	0.0	0.0	-0.0
								2 Slab self weight	-0.0	-0.0	19.4	0.0	0.0	0.0
								3 Dead	0.1	-0.0	51.6	0.0	0.0	0.0
								4 Live	-0.0	0.0	45.2	0.0	0.0	-0.0
								5 Roof Live	0.1	-0.0	9.5	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.1	0.0	-11.2	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	-0.0	0.0	-3.5	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	-0.1	0.0	-10.0	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	-0.0	-0.0	-3.5	0.0	0.0	-0.0
								27 Minimum Snow Load	0.1	-0.0	10.0	0.0	0.0	-0.0
								28 Balanced Snow Load	0.1	-0.0	8.2	0.0	0.0	0.0
								30 Drift Snow Load 1	0.1	-0.0	10.8	0.0	0.0	-0.0
								31 Seismic Dir1	0.6	-0.0	-0.0	0.0	0.0	0.0
								32 Seismic Dir2	-0.0	0.3	0.0	0.0	0.0	-0.0
SC A.9/5	SUP A.9/5	42′ 0″	32′ 2″	100′ 0″	SC A.9/5	HSS 10x10x1/2	0.0000	1 Self weight - excluding slabs	-0.0	-0.0	14.9	0.0	0.0	-0.0
								2 Slab self weight	0.0	0.0	39.1	0.0	0.0	0.0
								3 Dead	-0.1	-0.0	110.9	0.0	0.0	0.0
								4 Live	0.0	-0.0	81.9	0.0	0.0	-0.0
								5 Roof Live	-0.1	-0.0	24.6	0.0	0.0	-0.0

Source	Support	Coordinates [ft, in]			Reference	Size	Result rotation [°]	Loadcase	Reactions					
		X	Y	Z					F_x [kip]	F_y [kip]	F_z [kip]	M_x [kip ft]	M_y [kip ft]	M_z [kip ft]
SC A.9/5	SUP A.9/5	42' 0"	32' 2"	100' 0"	SC A.9/5	HSS 10x10x1/2	0.0000	11 Wind +X',GCpi 0.18,-Cp	0.1	-0.0	-28.8	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	0.0	-0.0	-9.0	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	0.1	-0.0	-28.5	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	0.0	-0.0	-9.0	0.0	0.0	-0.0
								27 Minimum Snow Load	-0.1	-0.0	25.8	0.0	0.0	-0.0
								28 Balanced Snow Load	-0.1	-0.0	21.1	0.0	0.0	-0.0
								30 Drift Snow Load 1	-0.1	-0.0	22.0	0.0	0.0	-0.0
								31 Seismic Dir1	1.9	0.0	-1.2	0.0	0.0	0.0
								32 Seismic Dir2	-0.0	0.7	0.0	0.0	0.0	-0.0
SC A.9/5.9	SUP A.9/5.9	80' 4"	32' 2"	100' 0"	SC A.9/5.9	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	-0.0	6.4	0.0	0.0	-0.0
								2 Slab self weight	-0.0	0.0	15.7	0.0	0.0	0.0
								3 Dead	0.1	0.0	41.7	0.0	0.0	0.0
								4 Live	-0.0	0.0	32.1	0.0	0.0	-0.0
								5 Roof Live	0.1	0.0	7.6	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.1	-0.0	-9.0	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	-0.0	-0.0	-2.8	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	-0.1	-0.0	-9.0	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	-0.0	-0.0	-2.8	0.0	0.0	-0.0
								27 Minimum Snow Load	0.1	0.0	8.0	0.0	0.0	-0.0
								28 Balanced Snow Load	0.1	0.0	6.6	0.0	0.0	0.0
								30 Drift Snow Load 1	0.1	0.0	8.5	0.0	0.0	-0.0
								31 Seismic Dir1	0.6	0.0	1.2	0.0	0.0	0.0
								32 Seismic Dir2	-0.0	0.3	-0.0	0.0	0.0	-0.0
SC C/4.1	SUP C/4.1	3' 8"	60' 9"	100' 0"	SC C/4.1	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	-0.0	8.6	0.0	0.0	-0.0

Source	Support	Coordinates [ft, in]			Reference	Size	Result rotation [°]	Loadcase	Reactions					
		X	Y	Z					F_x [kip]	F_y [kip]	F_z [kip]	M_x [kip ft]	M_y [kip ft]	M_z [kip ft]
SC C/4.1	SUP C/4.1	3′ 8″	60′ 9″	100′ 0″	SC C/4.1	HSS 8x8x1/2	0.0000	2 Slab self weight	-0.0	0.0	18.9	0.0	0.0	0.0
								3 Dead	0.0	-0.0	51.2	0.0	0.0	0.0
								4 Live	-0.0	0.0	62.2	0.0	0.0	-0.0
								5 Roof Live	0.0	-0.0	10.0	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.0	0.0	-10.6	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	-0.0	0.0	-3.7	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	-0.0	0.0	-10.5	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	-0.0	-0.0	-3.7	0.0	0.0	-0.0
								27 Minimum Snow Load	0.0	-0.0	10.5	0.0	0.0	-0.0
								28 Balanced Snow Load	0.0	-0.0	8.6	0.0	0.0	0.0
								30 Drift Snow Load 1	0.0	-0.0	11.2	0.0	0.0	-0.0
								31 Seismic Dir1	1.1	-0.0	0.0	0.0	0.0	0.0
								32 Seismic Dir2	-0.0	0.4	0.0	0.0	0.0	-0.0
SC C/4.9	SUP C/4.9	39′ 8″	60′ 9″	100′ 0″	SC C/4.9	HSS 10x10x1/2	0.0000	1 Self weight - excluding slabs	0.0	-0.0	16.9	0.0	0.0	-0.0
								2 Slab self weight	-0.0	-0.0	46.5	0.0	0.0	0.0
								3 Dead	0.0	-0.0	125.5	0.0	0.0	0.0
								4 Live	-0.0	-0.0	111.0	0.0	0.0	-0.0
								5 Roof Live	0.0	-0.0	24.1	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.0	-0.0	-25.6	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	-0.0	-0.0	-8.8	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	-0.0	0.0	-27.9	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	-0.0	0.0	-8.8	0.0	0.0	-0.0
								27 Minimum Snow Load	0.0	-0.0	25.3	0.0	0.0	-0.0
								28 Balanced Snow Load	0.0	-0.0	20.6	0.0	0.0	-0.0
								30 Drift Snow Load 1	0.0	-0.0	21.7	0.0	0.0	-0.0
								31 Seismic Dir1	2.3	0.0	-0.0	0.0	0.0	0.0

Source	Support	Coordinates [ft, in]			Reference	Size	Result rotation [°]	Loadcase	Reactions					
		X	Y	Z					F_x [kip]	F_y [kip]	F_z [kip]	M_x [kip ft]	M_y [kip ft]	M_z [kip ft]
SC C/4.9	SUP C/4.9	39' 8"	60' 9"	100' 0"	SC C/4.9	HSS 10x10x1/2	0.0000	32 Seismic Dir2	-0.0	0.7	0.0	0.0	0.0	-0.0
SC C/5.9	SUP C/5.9	80' 4"	60' 9"	100' 0"	SC C/5.9	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	-0.0	9.3	0.0	0.0	-0.0
								2 Slab self weight	-0.0	0.0	22.3	0.0	0.0	0.0
								3 Dead	0.0	0.0	60.1	0.0	0.0	0.0
								4 Live	-0.0	0.0	45.8	0.0	0.0	-0.0
								5 Roof Live	0.0	0.0	11.4	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.0	-0.0	-12.0	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	-0.0	0.0	-4.2	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	-0.0	-0.0	-13.4	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	-0.0	-0.0	-4.2	0.0	0.0	-0.0
								27 Minimum Snow Load	0.0	0.0	12.0	0.0	0.0	-0.0
								28 Balanced Snow Load	0.0	0.0	9.8	0.0	0.0	0.0
								30 Drift Snow Load 1	0.0	0.0	12.5	0.0	0.0	-0.0
								31 Seismic Dir1	1.1	-0.0	0.0	0.0	0.0	0.0
								32 Seismic Dir2	-0.0	0.4	0.0	0.0	0.0	-0.0
SC D.4/4.1	SUP D.4/4.1	3' 8"	97' 4"	100' 0"	SC D.4/4.1	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	0.0	5.8	0.0	0.0	-0.0
								2 Slab self weight	0.0	0.0	12.0	0.0	0.0	0.0
								3 Dead	0.0	0.0	32.5	0.0	0.0	0.0
								4 Live	0.0	0.0	46.3	0.0	0.0	-0.0
								5 Roof Live	0.0	0.0	6.3	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	0.0	0.0	-5.4	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	0.0	0.0	-2.3	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	0.0	0.0	-6.3	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	0.0	0.0	-2.3	0.0	0.0	-0.0
								27 Minimum Snow Load	0.0	0.0	6.6	0.0	0.0	-0.0
								28 Balanced Snow Load	0.0	0.0	5.4	0.0	0.0	0.0

Source	Support	Coordinates [ft, in] X	Y	Z	Reference	Size	Result rotation [°]	Loadcase	Reactions F_x [kip]	F_y [kip]	F_z [kip]	M_x [kip ft]	M_y [kip ft]	M_z [kip ft]
SC D.4/4.1	SUP D.4/4.1	3′ 8″	97′ 4″	100′ 0″	SC D.4/4.1	HSS 8x8x1/2	0.0000	30 Drift Snow Load 1	0.0	0.0	7.5	0.0	0.0	-0.0
SC D.4/4.7	SUP D.4/4.7	29′ 8″	97′ 4″	100′ 0″	SC D.4/4.7	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	-0.0	6.3	0.0	0.0	-0.0
								2 Slab self weight	0.0	-0.0	13.5	0.0	0.0	0.0
								3 Dead	0.0	-0.0	37.4	0.0	0.0	0.0
								4 Live	-0.0	-0.0	52.0	0.0	0.0	-0.0
								5 Roof Live	0.0	-0.0	7.8	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.0	0.0	-6.7	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	-0.0	-0.0	-2.9	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	0.0	0.0	-8.7	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	0.0	0.0	-2.9	0.0	0.0	-0.0
								27 Minimum Snow Load	0.0	-0.0	8.2	0.0	0.0	-0.0
								28 Balanced Snow Load	0.0	-0.0	6.7	0.0	0.0	0.0
								30 Drift Snow Load 1	0.0	-0.0	7.4	0.0	0.0	-0.0
								31 Seismic Dir1	0.1	-0.0	0.0	0.0	0.0	0.0
SC D.4/5	SUP D.4/5	42′ 0″	97′ 4″	100′ 0″	SC D.4/5	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	-0.0	8.8	0.0	0.0	-0.0
								2 Slab self weight	-0.0	-0.0	20.2	0.0	0.0	0.0
								3 Dead	0.0	-0.0	54.5	0.0	0.0	0.0
								4 Live	-0.0	-0.0	41.5	0.0	0.0	-0.0
								5 Roof Live	0.0	-0.0	10.4	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.0	-0.0	-9.0	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	0.0	-0.0	-3.8	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	0.0	0.0	-12.2	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	0.0	0.0	-3.8	0.0	0.0	-0.0
								27 Minimum Snow Load	0.0	-0.0	10.9	0.0	0.0	-0.0
								28 Balanced Snow Load	0.0	-0.0	8.9	0.0	0.0	0.0
								30 Drift Snow Load 1	0.0	-0.0	9.4	0.0	0.0	-0.0

Source	Support	Coordinates [ft, in]			Reference	Size	Result rotation [°]	Loadcase	Reactions					
		X	Y	Z					F_x [kip]	F_y [kip]	F_z [kip]	M_x [kip ft]	M_y [kip ft]	M_z [kip ft]
SC D.4/5	SUP D.4/5	42′ 0″	97′ 4″	100′ 0″	SC D.4/5	HSS 8x8x1/2	0.0000	31 Seismic Dir1	0.2	0.0	0.0	0.0	0.0	0.0
								32 Seismic Dir2	-0.0	0.4	0.0	0.0	0.0	-0.0
SC D.4/5.9	SUP D.4/5.9	80′ 4″	97′ 4″	100′ 0″	SC D.4/5.9	HSS 8x8x1/2	0.0000	1 Self weight - excluding slabs	0.0	0.0	8.7	0.0	0.0	-0.0
								2 Slab self weight	-0.0	0.0	19.5	0.0	0.0	0.0
								3 Dead	0.0	0.0	52.4	0.0	0.0	0.0
								4 Live	-0.0	0.0	40.0	0.0	0.0	-0.0
								5 Roof Live	0.0	0.0	10.0	0.0	0.0	-0.0
								11 Wind +X',GCpi 0.18,-Cp	-0.0	-0.0	-8.6	0.0	0.0	-0.0
								12 Wind +X',GCpi 0.18,+Cp	0.0	0.0	-3.7	0.0	0.0	-0.0
								14 Wind +Y',GCpi 0.18,-Cp	0.0	-0.0	-11.7	0.0	0.0	-0.0
								15 Wind +Y',GCpi 0.18,+Cp	0.0	-0.0	-3.7	0.0	0.0	-0.0
								27 Minimum Snow Load	0.0	0.0	10.5	0.0	0.0	-0.0
								28 Balanced Snow Load	0.0	0.0	8.6	0.0	0.0	0.0
								30 Drift Snow Load 1	0.0	0.0	11.1	0.0	0.0	-0.0
								31 Seismic Dir1	0.2	-0.0	0.0	0.0	0.0	0.0
								32 Seismic Dir2	-0.0	0.3	0.0	0.0	0.0	-0.0

Wall Supports

No results available.

COLUMN D.4 4.1

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.847

Description	**Unit**	**Applied**	**Resisting**	**FoS**	**Result**
Uplift verification	kips	91.5			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.542	3	0.847	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	68.8	142.6	0.483	Pass
Moment, positive, y-direction	kip_ft	68.8	142.6	0.483	Pass
Shear, one-way, x-direction	kips	35.7	49.0	0.729	Pass
Shear, one-way, y-direction	kips	35.7	49.0	0.729	Pass
Shear, two-way, Col 1	psi	145.107	201.246	0.721	Pass
Min.area of reinf, bot., x-direction	in^2	1.814	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.8		Pass
Min.area of reinf, bot., y-direction	in^2	1.814	3.080		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.8		Pass

Pad footing details

Length of footing; L_x = **6** ft

Width of footing; L_y = **6** ft

Footing area; $A = L_x \times L_y$ = **36** ft^2

Depth of footing; h = **14** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3

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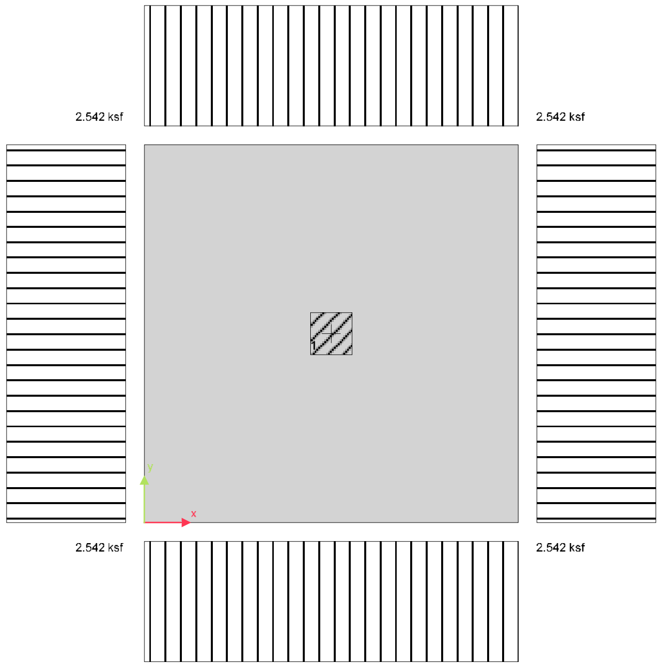

Column no.1 details

Length of column; l_{x1} = **8.00** in

Width of column; l_{y1} = **8.00** in

position in x-axis; x_1 = **36.00** in

position in y-axis; y_1 = **36.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **175** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **32.5** kips

Live load in z; F_{Lz1} = **46.3** kips

Live roof load in z; F_{Lrz1} = **6.3** kips

Snow load in z; F_{Sz1} = **7.6** kips

Wind load in z; F_{Wz1} = **-6.3** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.419)

1.0D + 1.0L (0.847)
1.0D + 1.0Lr (0.477)
1.0D + 0.7S (0.468)
1.0D + 1.0R (0.419)
1.0D + 0.75L + 0.75Lr (0.784)
1.0D + 0.75L + 0.525S (0.777)
1.0D + 0.75L + 0.75R (0.740)
1.0D + 0.6W (0.384)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.449)
1.0D + 0.75L + 0.75Lr + 0.45W (0.758)
1.0D + 0.75L + 0.525S + 0.45W (0.751)
1.0D + 0.75L + 0.75R + 0.45W (0.714)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.770)
0.6D + 0.6W (0.216)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.220)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **91.5** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **274.5** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **274.5** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **91.5** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in
Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.542** ksf
$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.542** ksf
$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.542** ksf
$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.542** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.542** ksf
Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.542** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} =$ **3** ksf
$q_{max} / q_{allow} =$ **0.847**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; f'_c = **4500** psi

Yield strength of reinforcement; f_y = **60000** psi

Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**

Cover to top of footing; c_{nom_t} = **3** in

Cover to side of footing; c_{nom_s} = **3** in

Cover to bottom of footing; c_{nom_b} = **3** in

Concrete type; Normal weight

Concrete modification factor; λ = **1.00**

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.299)

1.2D + 1.6L + 0.5Lr (0.729)

1.2D + 1.6L + 0.3S (0.723)

1.2D + 1.6L + 0.5R (0.721)

1.2D + 1.0L + 1.6Lr (0.721)

1.2D + 1.0L + 1.0S (0.721)

1.2D + 1.0L + 1.6R (0.721)

1.2D + 1.6Lr + 0.5W (0.721)

1.2D + 1.0S + 0.5W (0.721)

1.2D + 1.6R + 0.5W (0.721)

1.2D + 1.0L + 0.5Lr + 1.0W (0.721)

1.2D + 1.0L + 0.3S + 1.0W (0.721)

1.2D + 1.0L + 0.5R + 1.0W (0.721)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.721)

0.9D + 1.0W (0.721)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.721)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1}$ = **131.5** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1)$ = **394.4** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **394.4** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.652}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.652}$ ksf

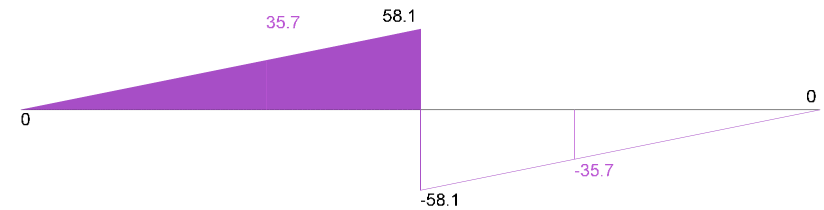
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.652}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.652}$ ksf

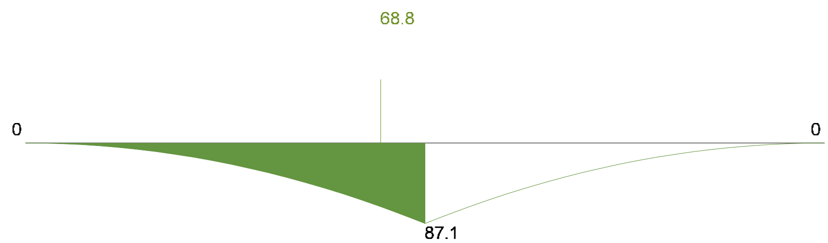
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.652}$ ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.652}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} = \mathbf{68.837}$ kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} = \mathbf{3.08}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{1.814}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{10.625}$ in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.671}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.83}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{0.813}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.03619}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{158.458}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{142.612}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.483}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **35.693** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **10.625** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00403**

Shear strength reduction factor; ϕ_v = **0.75**

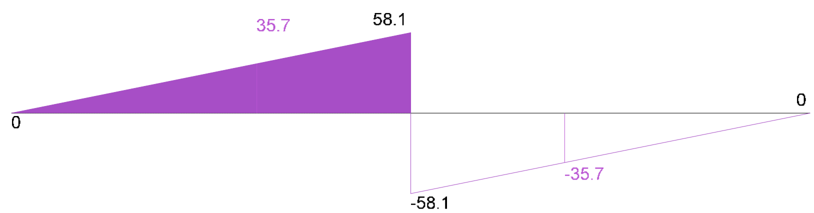
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v)$ = **65.311** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **48.983** kips

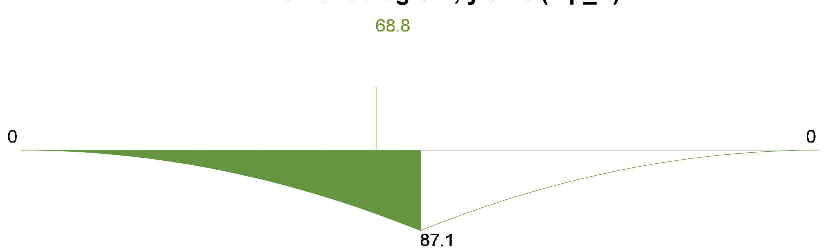
$V_{u.x} / \phi V_n$ = **0.729**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **68.837** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **1.814** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **9.875** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.671** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.813** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03342**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **146.908** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n$ = **132.217** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.521**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

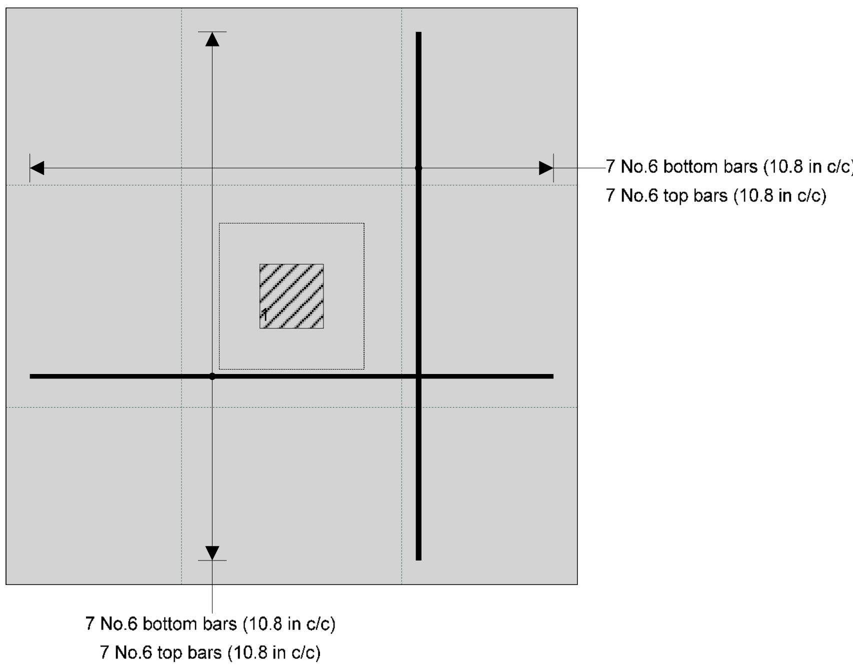
Ultimate shear force;	$V_{u.y}$ = **35.693** kips
Depth to reinforcement;	$d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2)$ = **9.875** in
Size effect factor (22.5.5.1.3);	λ_s = **1**
Ratio of longitudinal reinforcement;	$\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v)$ = **0.00433**
Shear strength reduction factor;	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v)$ = **62.2** kips
Design shear capacity;	$\phi V_n = \phi_v \times V_n$ = **46.65** kips
	$V_{u.y} / \phi V_n$ = **0.765**

PASS - Design shear capacity exceeds ultimate shear load

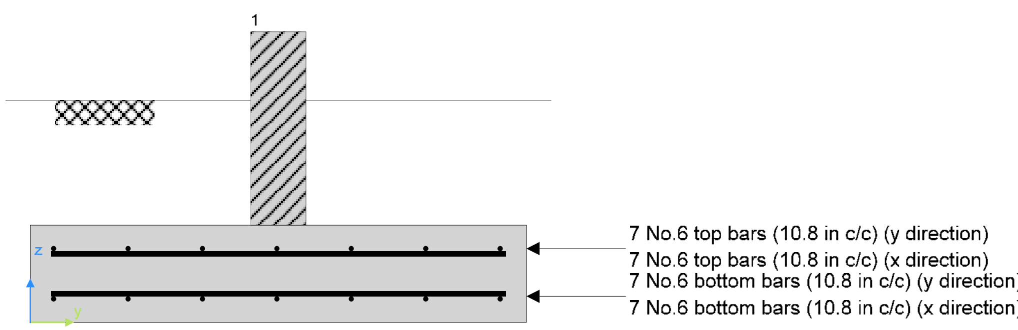
Two-way shear design at column 1

Depth to reinforcement;	d_{v2} = **10.25** in
Shear perimeter length (22.6.4);	l_{xp} = **18.250** in
Shear perimeter width (22.6.4);	l_{yp} = **18.250** in
Shear perimeter (22.6.4);	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **73.000** in
Shear area;	$A_p = l_{x,perim} \times l_{y,perim}$ = **333.062** in^2
Surcharge loaded area;	$A_{sur} = A_p - l_{x1} \times l_{y1}$ = **269.062** in^2
Ultimate bearing pressure at center of shear area;	$q_{up.avg}$ = **3.652** ksf
Ultimate shear load;	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **108.577** kips
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **145.107** psi
Column geometry factor (Table 22.6.5.2);	$\beta = l_{y1} / l_{x1}$ = **1.00**
Column location factor (22.6.5.3);	α_s =**40**
Size effect factor (22.5.5.1.3);	λ_s = **1**
Concrete shear strength (22.6.5.2);	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **402.492** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **510.926** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **268.328** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **268.328** psi
Shear strength reduction factor;	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2);	$v_n = v_{cp}$ = **268.328** psi
Design shear stress capacity (8.5.1.1(d));	$\phi v_n = \phi_v \times v_n$ = **201.246** psi
	$v_{ug} / \phi v_n$ = **0.721**

PASS - Design shear stress capacity exceeds ultimate shear stress load



7 No.6 bottom bars (10.8 in c/c)
7 No.6 top bars (10.8 in c/c)
7 No.6 bottom bars (10.8 in c/c)
7 No.6 top bars (10.8 in c/c)



1
z
y
7 No.6 top bars (10.8 in c/c) (y direction)
7 No.6 top bars (10.8 in c/c) (x direction)
7 No.6 bottom bars (10.8 in c/c) (y direction)
7 No.6 bottom bars (10.8 in c/c) (x direction)

;

COLUMN D.4 4.7

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.933

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	100.8			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.8	3	0.933	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	77.0	142.6	0.540	Pass
Moment, positive, y-direction	kip_ft	77.0	142.6	0.540	Pass
Shear, one-way, x-direction	kips	39.9	46.7	0.856	Pass
Shear, one-way, y-direction	kips	39.9	46.7	0.856	Pass
Shear, two-way, Col 1	psi	162.391	201.246	0.807	Pass
Min.area of reinf, bot., x-direction	in^2	1.814	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.8		Pass
Min.area of reinf, bot., y-direction	in^2	1.814	3.080		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.8		Pass

Pad footing details

Length of footing; L_x = **6** ft

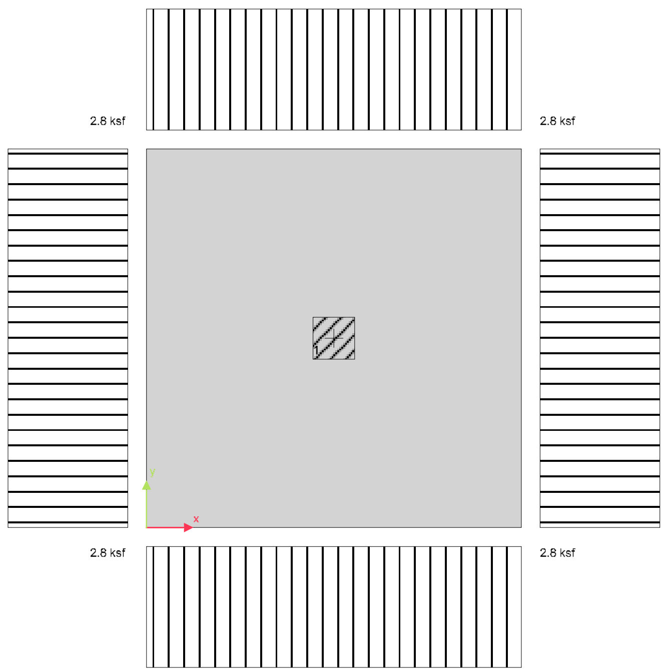
Width of footing; L_y = **6** ft

Footing area; $A = L_x \times L_y$ = **36** ft^2

Depth of footing; h = **14** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column; l_{x1} = **8.00** in

Width of column; l_{y1} = **8.00** in

position in x-axis; x_1 = **36.00** in

position in y-axis; y_1 = **36.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **175** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **36.9** kips

Live load in z; F_{Lz1} = **51.2** kips

Live roof load in z; F_{Lrz1} = **7.7** kips

Snow load in z; F_{Sz1} = **8.1** kips

Wind load in z; F_{Wz1} = **-8.6** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.459)

1.0D + 1.0L (0.933)

1.0D + 1.0Lr (0.531)

1.0D + 0.7S (0.512)

1.0D + 1.0R (0.459)

1.0D + 0.75L + 0.75Lr (0.868)

1.0D + 0.75L + 0.525S (0.854)

1.0D + 0.75L + 0.75R (0.815)

1.0D + 0.6W (0.411)

$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.493)

1.0D + 0.75L + 0.75Lr + 0.45W (0.832)

1.0D + 0.75L + 0.525S + 0.45W (0.818)

1.0D + 0.75L + 0.75R + 0.45W (0.779)

$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.848)

0.6D + 0.6W (0.228)

$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.242)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **100.8** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **302.4** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **302.4** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **100.8** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.8** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.8** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.8** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.8** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.8** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.8** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} = \mathbf{3}$ ksf

$q_{max} / q_{allow} = \mathbf{0.933}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; $f'_c = \mathbf{4500}$ psi

Yield strength of reinforcement; $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2); $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to top of footing; $c_{nom\_t} = \mathbf{3}$ in

Cover to side of footing; $c_{nom\_s} = \mathbf{3}$ in

Cover to bottom of footing; $c_{nom\_b} = \mathbf{3}$ in

Concrete type; Normal weight

Concrete modification factor; $\lambda = \mathbf{1.00}$

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.340)

1.2D + 1.6L + 0.5Lr (0.856)

1.2D + 1.6L + 0.3S (0.847)

1.2D + 1.6L + 0.5R (0.831)

1.2D + 1.0L + 1.6Lr (0.807)

1.2D + 1.0L + 1.0S (0.807)

1.2D + 1.0L + 1.6R (0.807)

1.2D + 1.6Lr + 0.5W (0.807)

1.2D + 1.0S + 0.5W (0.807)

1.2D + 1.6R + 0.5W (0.807)

1.2D + 1.0L + 0.5Lr + 1.0W (0.807)

1.2D + 1.0L + 0.3S + 1.0W (0.807)

1.2D + 1.0L + 0.5R + 1.0W (0.807)

$(1.2 + 0.2 \times S_{DS})D + 1.0L + 0.15S + 1.0E$ (0.807)

0.9D + 1.0W (0.807)

$(0.9 - 0.2 \times S_{DS})D + 1.0E$ (0.807)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} = \mathbf{145.3}$ kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1)$ = **435.9** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **435.9** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.036** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.036** ksf

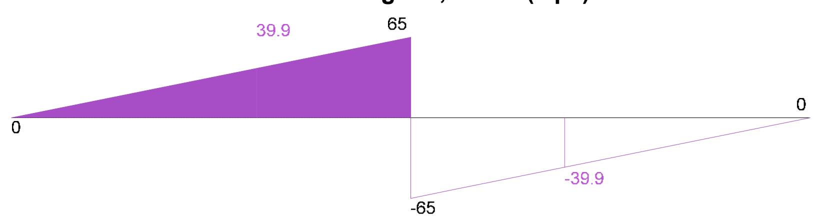
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.036** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.036** ksf

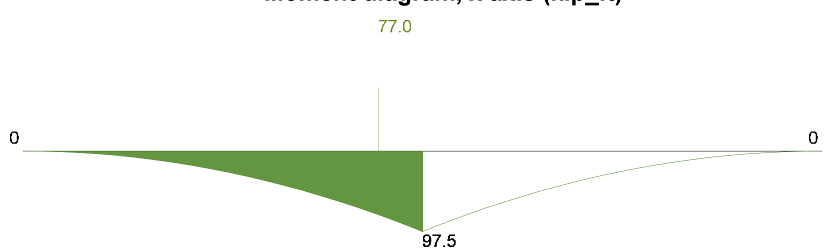
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **4.036** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **4.036** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max}$ = **77.029** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h$ = **1.814** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **10.625** in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.671** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.813** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.03619}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{158.458}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{142.612}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.540}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x} = \mathbf{39.941}$ kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2) = \mathbf{10.625}$ in

Size effect factor (22.5.5.1.3); $\lambda_s = \mathbf{1}$

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v) = \mathbf{0.00403}$

Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

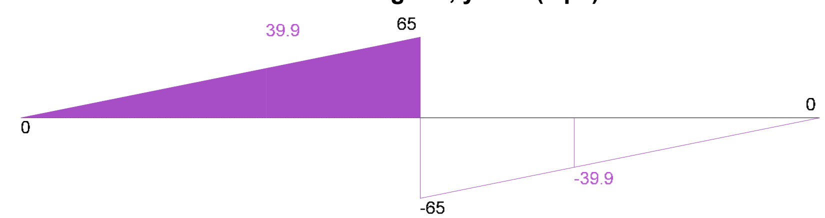
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v) = \mathbf{65.311}$ kips

Design shear capacity; $\phi V_n = \phi_v \times V_n = \mathbf{48.983}$ kips

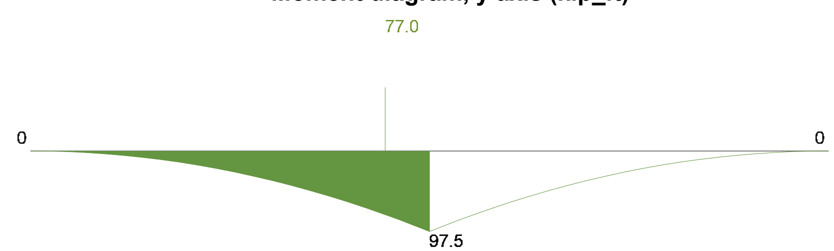
$V_{u.x} / \phi V_n = \mathbf{0.815}$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max} = \mathbf{77.029}$ kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} = \mathbf{3.08}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h = \mathbf{1.814}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;	$d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 =$ **9.875** in
Depth of compression block;	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) =$ **0.671** in
Neutral axis factor;	$\beta_1 =$ **0.83**
Depth to neutral axis;	$c = a / \beta_1 =$ **0.813** in
Strain in tensile reinforcement;	$\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03342**
Minimum tensile strain(8.3.3.1);	$\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **146.908** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n =$ **132.217** kip_ft
	$M_{u.y.max} / \phi M_n =$ **0.583**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;	$V_{u.y} =$ **39.941** kips
Depth to reinforcement;	$d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **9.875** in
Size effect factor (22.5.5.1.3);	$\lambda_s =$ **1**
Ratio of longitudinal reinforcement;	$\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00433**
Shear strength reduction factor;	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v) =$ **62.2** kips
Design shear capacity;	$\phi V_n = \phi_v \times V_n =$ **46.65** kips
	$V_{u.y} / \phi V_n =$ **0.856**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;	$d_{v2} =$ **10.25** in
Shear perimeter length (22.6.4);	$l_{xp} =$ **18.250** in
Shear perimeter width (22.6.4);	$l_{yp} =$ **18.250** in
Shear perimeter (22.6.4);	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **73.000** in
Shear area;	$A_p = l_{x,perim} \times l_{y,perim} =$ **333.062** in^2
Surcharge loaded area;	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **269.062** in^2
Ultimate bearing pressure at center of shear area;	$q_{up.avg} =$ **4.036** ksf
Ultimate shear load;	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **121.509** kips
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi}) =$ **162.391** psi
Column geometry factor (Table 22.6.5.2);	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (22.6.5.3);	$\alpha_s =$ **40**
Size effect factor (22.5.5.1.3);	$\lambda_s =$ **1**
Concrete shear strength (22.6.5.2);	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **402.492** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **510.926** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **268.328** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi
Shear strength reduction factor;	$\phi_v =$ **0.75**

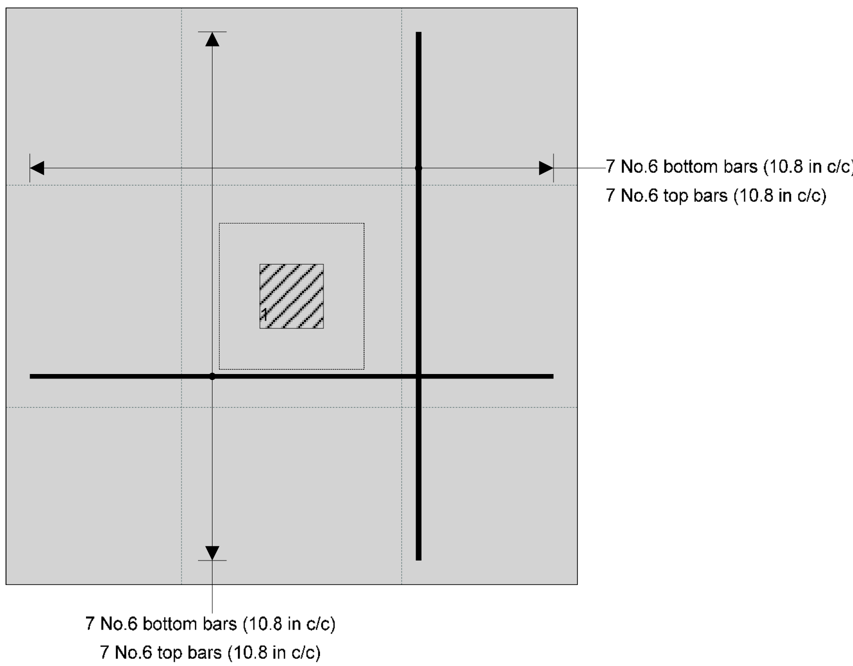
Tekla Tedds Michael Baker Internation 7090 S Union Park Ave Midvale, UT 84107	Project Creech DRP PH 2				Job Ref.	
	Section Column D.4 - 4.7				Sheet no./rev. 17	
	Calc. by ACV	Date 10/16/2025	Chk'd by	Date	App'd by	Date

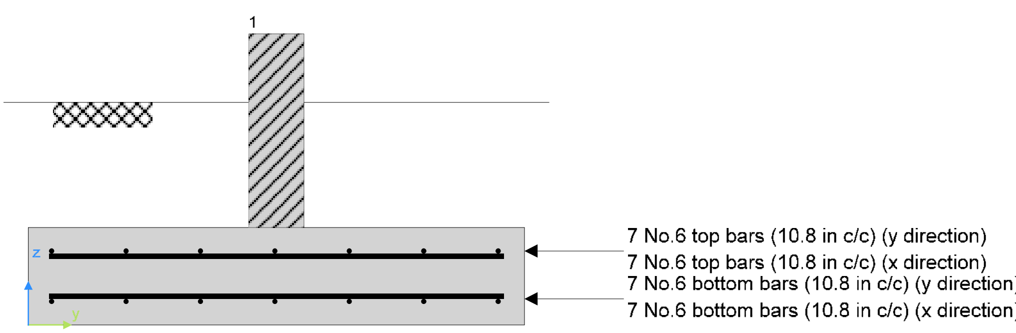
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} = \mathbf{268.328}$ psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n = \mathbf{201.246}$ psi

$v_{ug} / \phi v_n = \mathbf{0.807}$

PASS - Design shear stress capacity exceeds ultimate shear stress load





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COLUMN D.4 5.0

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.774

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	113.8			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.323	3	0.774	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	97.4	171.0	0.569	Pass
Moment, positive, y-direction	kip_ft	97.4	171.0	0.569	Pass
Shear, one-way, x-direction	kips	42.3	60.9	0.694	Pass
Shear, one-way, y-direction	kips	42.3	58.5	0.723	Pass
Shear, two-way, Col 1	psi	128.899	201.246	0.641	Pass
Min.area of reinf, bot., x-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	12.8		Pass
Min.area of reinf, bot., y-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	12.8		Pass

Pad footing details

Length of footing; L_x = **7** ft

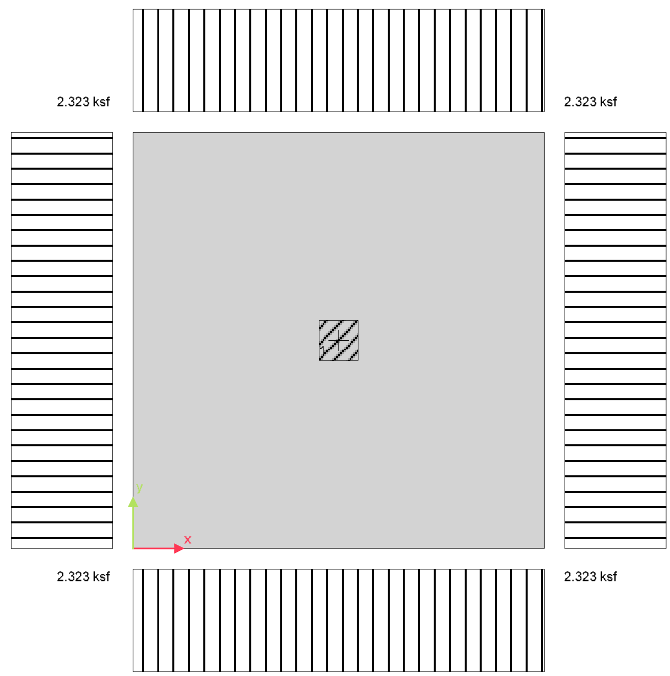
Width of footing; L_y = **7** ft

Footing area; $A = L_x \times L_y$ = **49** ft^2

Depth of footing; h = **16** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column; l_{x1} = **8.00** in

Width of column; l_{y1} = **8.00** in

position in x-axis; x_1 = **42.00** in

position in y-axis; y_1 = **42.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **200** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **54.1** kips

Live load in z; F_{Lz1} = **41.2** kips

Live roof load in z; F_{Lrz1} = **10.3** kips

Snow load in z; F_{Sz1} = **10.9** kips

Wind load in z; F_{Wz1} = **-12.1** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.494)

1.0D + 1.0L (0.774)

1.0D + 1.0Lr (0.564)

1.0D + 0.7S (0.546)

1.0D + 1.0R (0.494)

1.0D + 0.75L + 0.75Lr (0.757)

1.0D + 0.75L + 0.525S (0.743)

1.0D + 0.75L + 0.75R (0.704)

1.0D + 0.6W (0.445)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.531)

1.0D + 0.75L + 0.75Lr + 0.45W (0.720)

1.0D + 0.75L + 0.525S + 0.45W (0.706)

1.0D + 0.75L + 0.75R + 0.45W (0.667)

$(1.0 + 0.105 \times S_{DS})$D + 0.75L + 0.1S + 0.525E (0.739)

0.6D + 0.6W (0.247)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.260)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **113.8** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **398.4** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **398.4** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **113.84** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.323** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.323** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.323** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.323** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.323** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.323** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} = \mathbf{3}$ ksf

$q_{max} / q_{allow} = \mathbf{0.774}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; $f'_c = \mathbf{4500}$ psi

Yield strength of reinforcement; $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2); $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to top of footing; $c_{nom\_t} = \mathbf{3}$ in

Cover to side of footing; $c_{nom\_s} = \mathbf{3}$ in

Cover to bottom of footing; $c_{nom\_b} = \mathbf{3}$ in

Concrete type; Normal weight

Concrete modification factor; $\lambda = \mathbf{1.00}$

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.402)

1.2D + 1.6L + 0.5Lr (0.723)

1.2D + 1.6L + 0.3S (0.713)

1.2D + 1.6L + 0.5R (0.696)

1.2D + 1.0L + 1.6Lr (0.652)

1.2D + 1.0L + 1.0S (0.641)

1.2D + 1.0L + 1.6R (0.641)

1.2D + 1.6Lr + 0.5W (0.641)

1.2D + 1.0S + 0.5W (0.641)

1.2D + 1.6R + 0.5W (0.641)

1.2D + 1.0L + 0.5Lr + 1.0W (0.641)

1.2D + 1.0L + 0.3S + 1.0W (0.641)

1.2D + 1.0L + 0.5R + 1.0W (0.641)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.641)

0.9D + 1.0W (0.641)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.641)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} = \mathbf{158.2}$ kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1)$ = **553.8** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **553.8** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.229** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.229** ksf

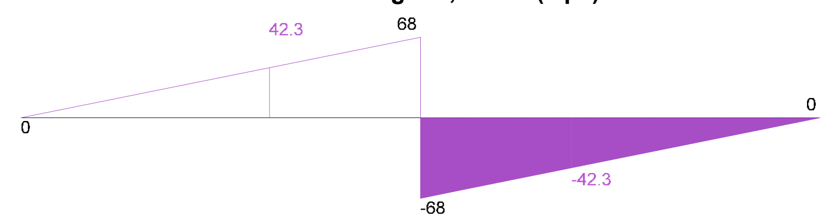
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.229** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.229** ksf

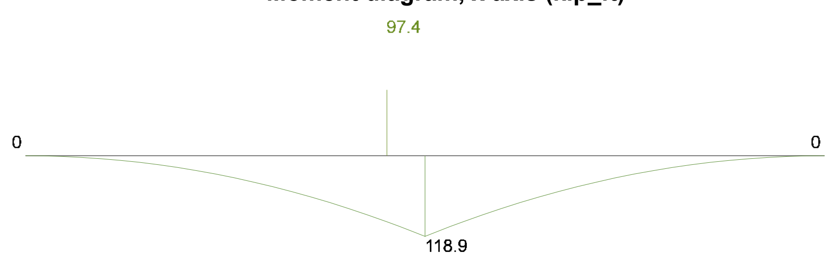
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.229** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.229** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max}$ = **97.357** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h$ = **2.419** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **12.625** in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.575** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05133}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{189.996}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{170.997}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.569}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x} = \mathbf{42.271}$ kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2) = \mathbf{12.625}$ in

Size effect factor (22.5.5.1.3); $\lambda_s = \mathbf{1}$

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v) = \mathbf{0.0029}$

Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

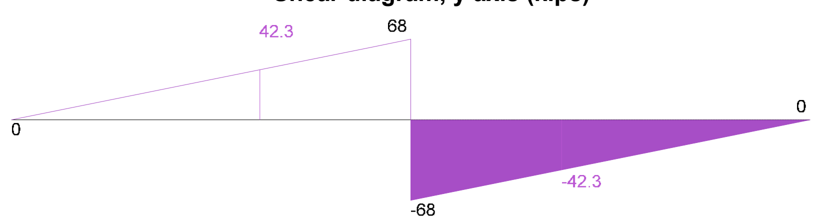
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v) = \mathbf{81.2}$ kips

Design shear capacity; $\phi V_n = \phi_v \times V_n = \mathbf{60.9}$ kips

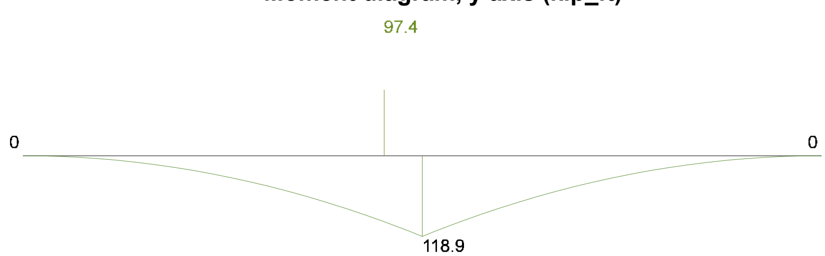
$V_{u.x} / \phi V_n = \mathbf{0.694}$

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max} = \mathbf{97.357}$ kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} = \mathbf{3.08}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h = \mathbf{2.419}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;	$d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 =$ **11.875** in
Depth of compression block;	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) =$ **0.575** in
Neutral axis factor;	$\beta_1 =$ **0.83**
Depth to neutral axis;	$c = a / \beta_1 =$ **0.697** in
Strain in tensile reinforcement;	$\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.04810**
Minimum tensile strain(8.3.3.1);	$\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **178.446** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n =$ **160.602** kip_ft
	$M_{u.y.max} / \phi M_n =$ **0.606**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;	$V_{u.y} =$ **42.271** kips
Depth to reinforcement;	$d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **11.875** in
Size effect factor (22.5.5.1.3);	$\lambda_s =$ **1**
Ratio of longitudinal reinforcement;	$\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00309**
Shear strength reduction factor;	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v) =$ **77.951** kips
Design shear capacity;	$\phi V_n = \phi_v \times V_n =$ **58.463** kips
	$V_{u.y} / \phi V_n =$ **0.723**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

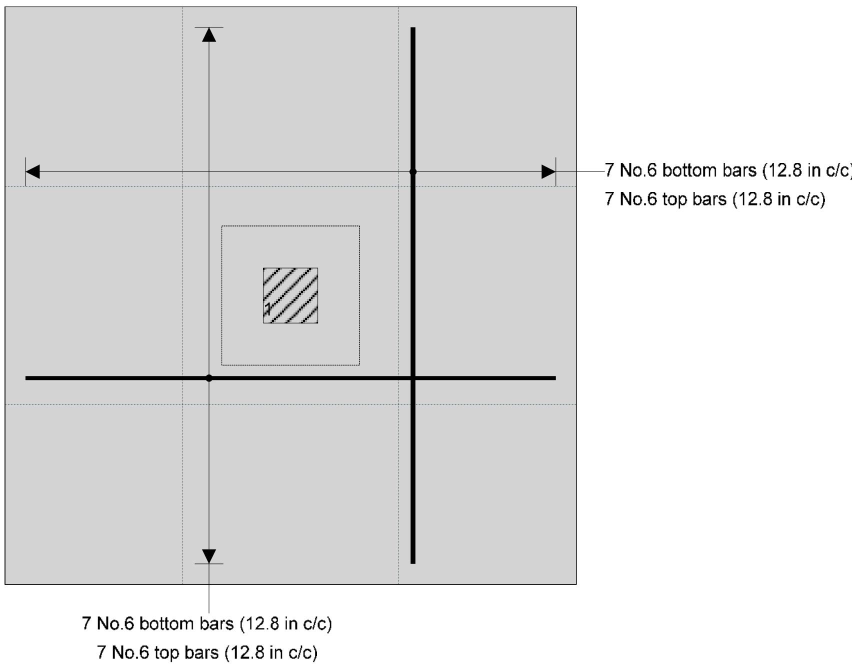
Depth to reinforcement;	$d_{v2} =$ **12.25** in
Shear perimeter length (22.6.4);	$l_{xp} =$ **20.250** in
Shear perimeter width (22.6.4);	$l_{yp} =$ **20.250** in
Shear perimeter (22.6.4);	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **81.000** in
Shear area;	$A_p = l_{x,perim} \times l_{y,perim} =$ **410.063** in^2
Surcharge loaded area;	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **346.063** in^2
Ultimate bearing pressure at center of shear area;	$q_{up.avg} =$ **3.229** ksf
Ultimate shear load;	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **127.900** kips
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi) =$ **128.899** psi
Column geometry factor (Table 22.6.5.2);	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (22.6.5.3);	$\alpha_s =$ **40**
Size effect factor (22.5.5.1.3);	$\lambda_s =$ **1**
Concrete shear strength (22.6.5.2);	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **402.492** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **539.969** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **268.328** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi
Shear strength reduction factor;	$\phi_v =$ **0.75**

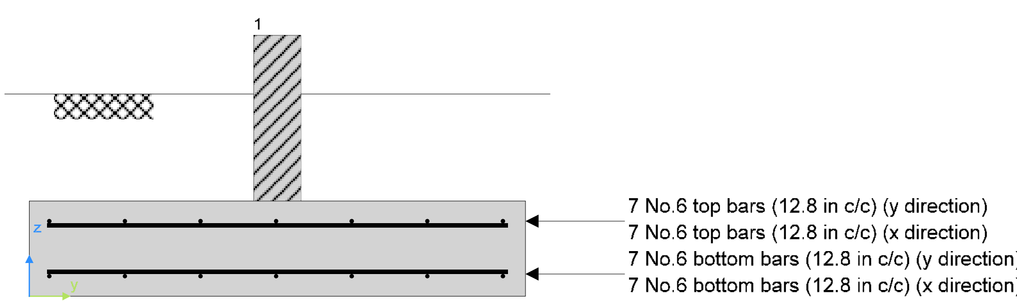
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **268.328** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.641**

PASS - Design shear stress capacity exceeds ultimate shear stress load





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COLUMN D.4 5.9

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.761

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	111.8			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.282	3	0.761	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	94.1	160.6	0.586	Pass
Moment, positive, y-direction	kip_ft	94.1	171.0	0.550	Pass
Shear, one-way, x-direction	kips	40.9	58.5	0.699	Pass
Shear, one-way, y-direction	kips	40.9	58.5	0.699	Pass
Shear, two-way, Col 1	psi	124.580	201.246	0.628	Pass
Min.area of reinf, bot., x-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	12.8		Pass
Min.area of reinf, bot., y-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	12.8		Pass

Pad footing details

Length of footing; L_x = **7** ft

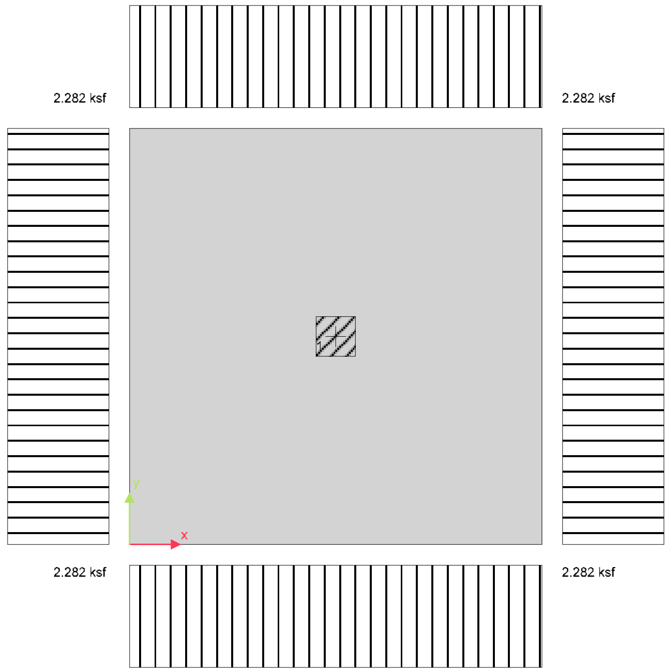
Width of footing; L_y = **7** ft

Footing area; $A = L_x \times L_y$ = **49** ft^2

Depth of footing; h = **16** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column; l_{x1} = **8.00** in

Width of column; l_{y1} = **8.00** in

position in x-axis; x_1 = **42.00** in

position in y-axis; y_1 = **42.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **200** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **53.0** kips

Live load in z; F_{Lz1} = **40.3** kips

Live roof load in z; F_{Lrz1} = **10.5** kips

Snow load in z; F_{Sz1} = **11.2** kips

Wind load in z; F_{Wz1} = **-11.8** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.487)

1.0D + 1.0L (0.761)

1.0D + 1.0Lr (0.558)
1.0D + 0.7S (0.540)
1.0D + 1.0R (0.487)
1.0D + 0.75L + 0.75Lr (0.746)
1.0D + 0.75L + 0.525S (0.732)
1.0D + 0.75L + 0.75R (0.692)
1.0D + 0.6W (0.439)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.523)
1.0D + 0.75L + 0.75Lr + 0.45W (0.710)
1.0D + 0.75L + 0.525S + 0.45W (0.696)
1.0D + 0.75L + 0.75R + 0.45W (0.656)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.727)
0.6D + 0.6W (0.244)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.256)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **111.8** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **391.4** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **391.4** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **111.84** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.282** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.282** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.282** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.282** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.282** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.282** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} =$ **3** ksf

$q_{max} / q_{allow} =$ **0.761**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; f'_c = **4500** psi

Yield strength of reinforcement; f_y = **60000** psi

Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**

Cover to top of footing; c_{nom_t} = **3** in

Cover to side of footing; c_{nom_s} = **3** in

Cover to bottom of footing; c_{nom_b} = **3** in

Concrete type; Normal weight

Concrete modification factor; λ = **1.00**

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.378)

1.2D + 1.6L + 0.5Lr (0.681)

1.2D + 1.6L + 0.3S (0.699)

1.2D + 1.6L + 0.5R (0.681)

1.2D + 1.0L + 1.6Lr (0.628)

1.2D + 1.0L + 1.0S (0.628)

1.2D + 1.0L + 1.6R (0.628)

1.2D + 1.6Lr + 0.5W (0.628)

1.2D + 1.0S + 0.5W (0.628)

1.2D + 1.6R + 0.5W (0.628)

1.2D + 1.0L + 0.5Lr + 1.0W (0.628)

1.2D + 1.0L + 0.3S + 1.0W (0.628)

1.2D + 1.0L + 0.5R + 1.0W (0.628)

$(1.2 + 0.2 \times S_{DS})D$ + 1.0L + 0.15S + 1.0E (0.628)

0.9D + 1.0W (0.628)

$(0.9 - 0.2 \times S_{DS})D$ + 1.0E (0.628)

Combination 3 results: 1.2D + 1.6L + 0.3S

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **153.7** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_S \times (F_{Sz1} \times x_1)$ = **537.9** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **537.9** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.136}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.136}$ ksf

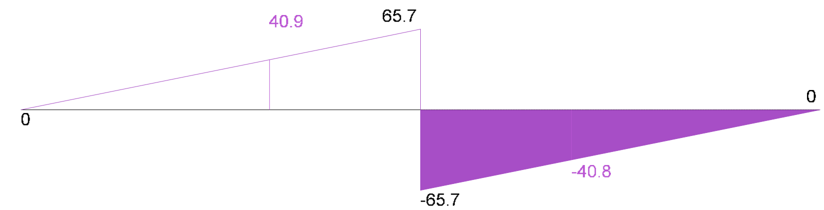
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.136}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.136}$ ksf

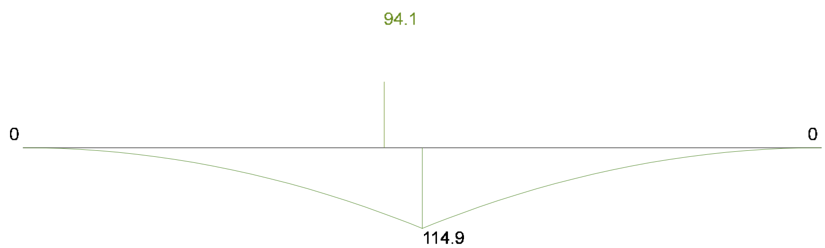
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.136}$ ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.136}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} = \mathbf{94.097}$ kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} = \mathbf{3.08}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{2.419}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{12.625}$ in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.575}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.83}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{0.697}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05133}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{189.996}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{170.997}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.550}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **40.856** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **12.625** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.0029**

Shear strength reduction factor; ϕ_v = **0.75**

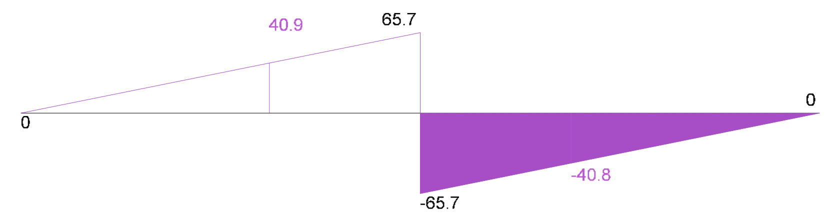
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v)$ = **81.2** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **60.9** kips

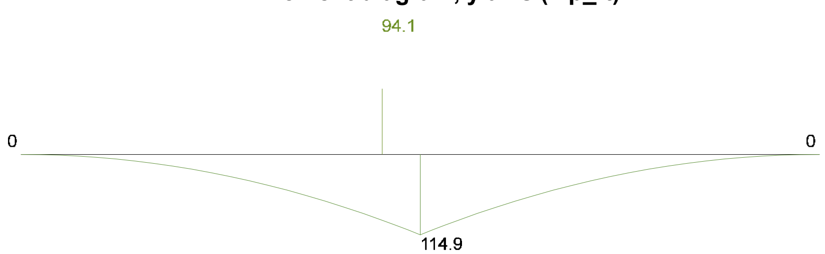
$V_{u.x} / \phi V_n$ = **0.671**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **94.097** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **2.419** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **11.875** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.575** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04810**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **178.446** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **160.602** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.586**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} =$ **40.856** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **11.875** in

Size effect factor (22.5.5.1.3); $\lambda_s = $ **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00309**

Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v) =$ **77.951** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **58.463** kips

$V_{u.y} / \phi V_n =$ **0.699**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} =$ **12.25** in

Shear perimeter length (22.6.4); $l_{xp} =$ **20.250** in

Shear perimeter width (22.6.4); $l_{yp} =$ **20.250** in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **81.000** in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} =$ **410.063** in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **346.063** in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} =$ **3.136** ksf

Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **123.615** kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) =$ **124.580** psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3); $\alpha_s =$ **40**

Size effect factor (22.5.5.1.3); $\lambda_s = $ **1**

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **402.492** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **539.969** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **268.328** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi

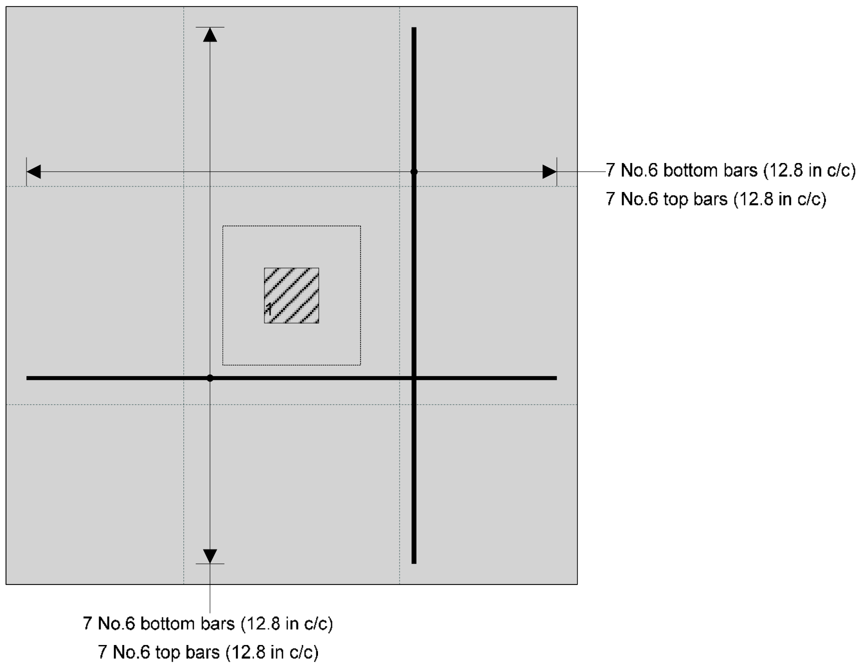
Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **268.328** psi

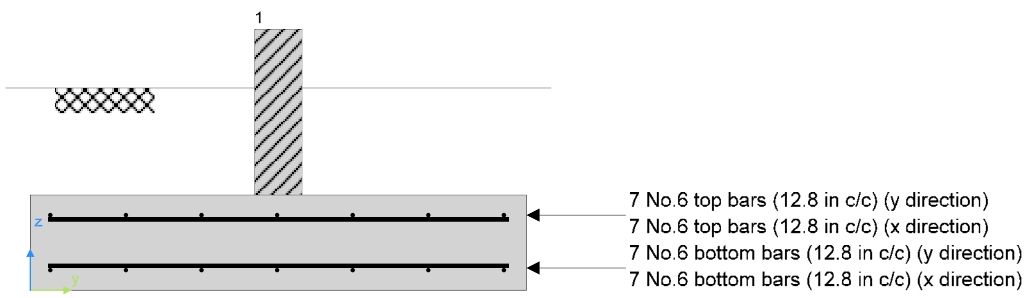
Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.619**

PASS - Design shear stress capacity exceeds ultimate shear stress load



7 No.6 bottom bars (12.8 in c/c)
7 No.6 top bars (12.8 in c/c)
7 No.6 bottom bars (12.8 in c/c)
7 No.6 top bars (12.8 in c/c)



1
7 No.6 top bars (12.8 in c/c) (y direction)
7 No.6 top bars (12.8 in c/c) (x direction)
7 No.6 bottom bars (12.8 in c/c) (y direction)
7 No.6 bottom bars (12.8 in c/c) (x direction)

;

COLUMN C 4

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.823

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	157.9			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.468	3	0.823	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	154.0	241.5	0.638	Pass
Moment, positive, y-direction	kip_ft	154.0	254.9	0.604	Pass
Shear, one-way, x-direction	kips	57.5	79.8	0.720	Pass
Shear, one-way, y-direction	kips	57.5	77.1	0.746	Pass
Shear, two-way, Col 1	psi	136.663	201.246	0.698	Pass
Min.area of reinf, bot., x-direction	in^2	3.110	3.960		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	11.1		Pass
Min.area of reinf, bot., y-direction	in^2	3.110	3.960		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	11.1		Pass

Pad footing details

Length of footing; L_x = **8** ft

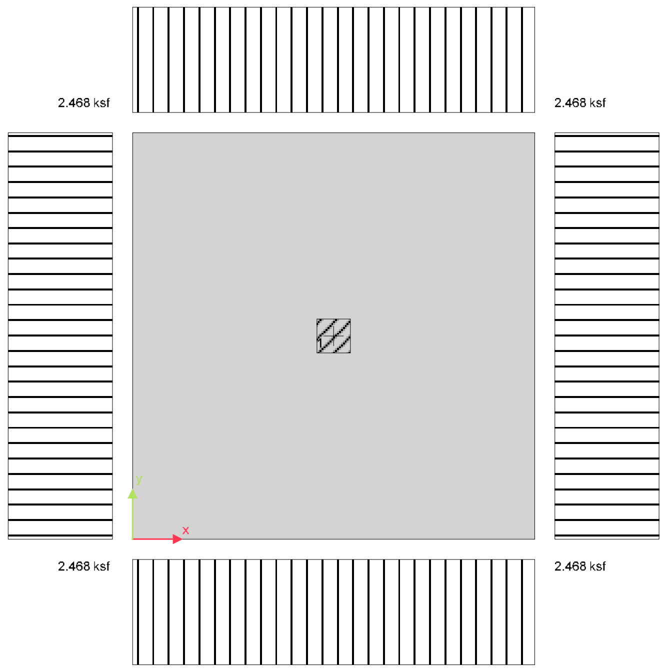
Width of footing; L_y = **8** ft

Footing area; $A = L_x \times L_y$ = **64** ft^2

Depth of footing; h = **18** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column;	l_{x1} = **8.00** in
Width of column;	l_{y1} = **8.00** in
position in x-axis;	x_1 = **48.00** in
position in y-axis;	y_1 = **48.00** in

Soil properties

Gross allowable bearing pressure;	q_{allow_Gross} = **3** ksf;
Density of soil;	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction;	ϕ_b = **30.0** deg
Design base friction angle;	δ_{bb} = **30.0** deg
Coefficient of base friction;	$\tan(\delta_{bb})$ = **0.577**
Self weight;	$F_{swt} = h \times \gamma_{conc}$ = **225** psf
Soil weight;	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z;	F_{Dz1} = **70.0** kips
Live load in z;	F_{Lz1} = **62.1** kips
Live roof load in z;	F_{Lrz1} = **10.0** kips
Snow load in z;	F_{Sz1} = **11.4** kips
Wind load in z;	F_{Wz1} = **-10.6** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.499)

1.0D + 1.0L (0.823)

1.0D + 1.0Lr (0.551)

1.0D + 0.7S (0.541)

1.0D + 1.0R (0.499)

1.0D + 0.75L + 0.75Lr (0.781)

1.0D + 0.75L + 0.525S (0.773)

1.0D + 0.75L + 0.75R (0.742)

1.0D + 0.6W (0.466)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.536)

1.0D + 0.75L + 0.75Lr + 0.45W (0.756)

1.0D + 0.75L + 0.525S + 0.45W (0.748)

1.0D + 0.75L + 0.75R + 0.45W (0.717)

$(1.0 + 0.105 \times S_{DS})$D + 0.75L + 0.1S + 0.525E (0.775)

0.6D + 0.6W (0.266)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.263)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **157.9** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **631.8** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **631.8** kip_ft

Uplift verification

Vertical force; F_{dz} = **157.94** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.468** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.468** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.468** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.468** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **2.468** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **2.468** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross}$ = **3** ksf

q_{max} / q_{allow} = **0.823**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; f'_c = **4500** psi

Yield strength of reinforcement; f_y = **60000** psi

Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**

Cover to top of footing; c_{nom_t} = **3** in

Cover to side of footing; c_{nom_s} = **3** in

Cover to bottom of footing; c_{nom_b} = **3** in

Concrete type; Normal weight

Concrete modification factor; λ = **1.00**

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.399)
1.2D + 1.6L + 0.5Lr (0.740)
1.2D + 1.6L + 0.3S (0.734)
1.2D + 1.6L + 0.5R (0.746)
1.2D + 1.0L + 1.6Lr (0.698)
1.2D + 1.0L + 1.0S (0.698)
1.2D + 1.0L + 1.6R (0.698)
1.2D + 1.6Lr + 0.5W (0.698)
1.2D + 1.0S + 0.5W (0.698)
1.2D + 1.6R + 0.5W (0.698)
1.2D + 1.0L + 0.5Lr + 1.0W (0.698)
1.2D + 1.0L + 0.3S + 1.0W (0.698)
1.2D + 1.0L + 0.5R + 1.0W (0.698)
$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.698)
0.9D + 1.0W (0.698)
$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.698)

Combination 4 results: 1.2D + 1.6L + 0.5R

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **214.4** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **857.5** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **857.5** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.349}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.349}$ ksf

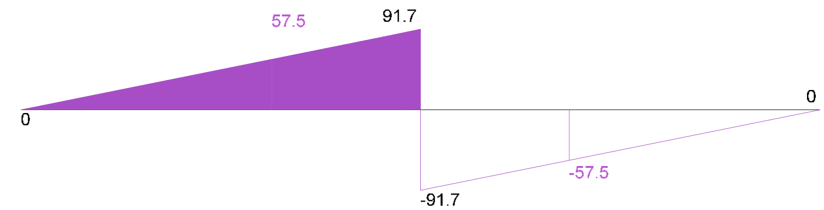
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.349}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.349}$ ksf

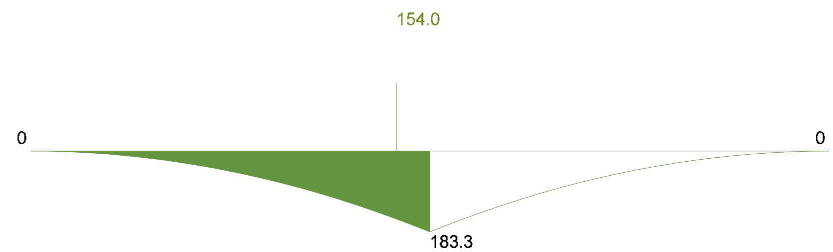
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.349}$ ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.349}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} = \mathbf{154.02}$ kip_ft

Tension reinforcement provided; 9 No.6 bottom bars (11.1 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} = \mathbf{3.96}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{3.11}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{14.625}$ in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.647}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.83}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{0.784}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05294}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{283.169}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{254.852}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.604}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **57.516** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **14.625** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00282**

Shear strength reduction factor; ϕ_v = **0.75**

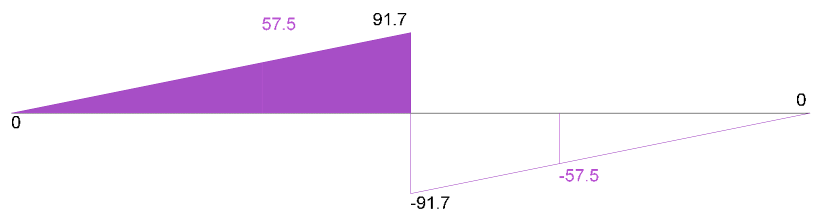
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v,\ 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v)$ = **106.457** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **79.842** kips

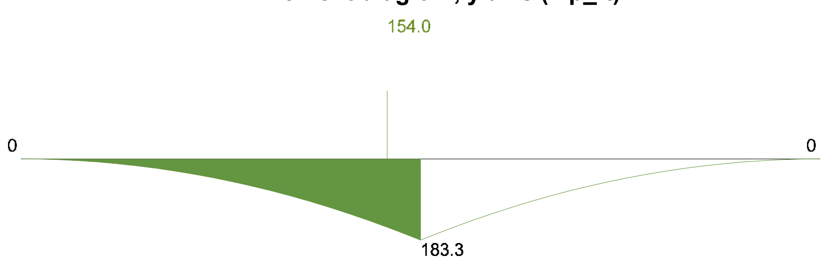
$V_{u.x} / \phi V_n$ = **0.720**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **154.02** kip_ft

Tension reinforcement provided; 9 No.6 bottom bars (11.1 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **3.96** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **3.11** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **13.875** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.647** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.784** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.05007**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = \mathbf{268.319}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{241.487}$ kip_ft

$M_{u.y.max} / \phi M_n = \mathbf{0.638}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} = \mathbf{57.516}$ kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) = \mathbf{13.875}$ in

Size effect factor (22.5.5.1.3); $\lambda_s = \mathbf{1}$

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) = \mathbf{0.00297}$

Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v) = \mathbf{102.785}$ kips

Design shear capacity; $\phi V_n = \phi_v \times V_n = \mathbf{77.089}$ kips

$V_{u.y} / \phi V_n = \mathbf{0.746}$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} = \mathbf{14.25}$ in

Shear perimeter length (22.6.4); $l_{xp} = \mathbf{22.250}$ in

Shear perimeter width (22.6.4); $l_{yp} = \mathbf{22.250}$ in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{89.000}$ in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} = \mathbf{495.062}$ in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{431.062}$ in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} = \mathbf{3.349}$ ksf

Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{173.323}$ kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{136.663}$ psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} = \mathbf{1.00}$

Column location factor (22.6.5.3); $\alpha_s = \mathbf{40}$

Size effect factor (22.5.5.1.3); $\lambda_s = \mathbf{1}$

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{402.492}$ psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{563.791}$ psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{268.328}$ psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{268.328}$ psi

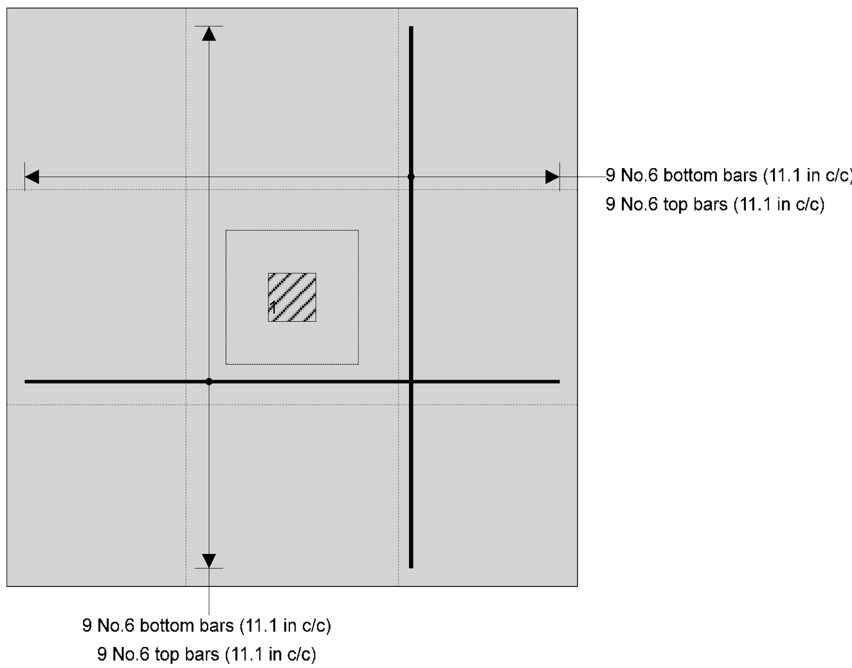
Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} = \mathbf{268.328}$ psi

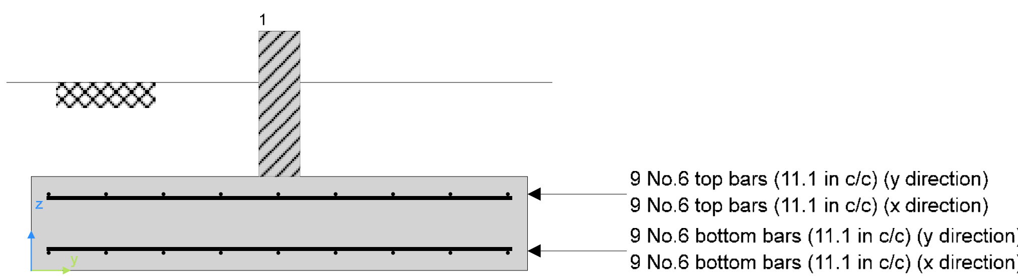
Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n = \mathbf{201.246}$ psi

$v_{ug} / \phi v_n = \mathbf{0.679}$

PASS - Design shear stress capacity exceeds ultimate shear stress load



9 No.6 bottom bars (11.1 in c/c)
9 No.6 top bars (11.1 in c/c)
9 No.6 bottom bars (11.1 in c/c)
9 No.6 top bars (11.1 in c/c)



1
9 No.6 top bars (11.1 in c/c) (y direction)
9 No.6 top bars (11.1 in c/c) (x direction)
9 No.6 bottom bars (11.1 in c/c) (y direction)
9 No.6 bottom bars (11.1 in c/c) (x direction)

;

COLUMN C 4.9

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.943

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	282.9			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.829	3	0.943	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	350.2	464.0	0.755	Pass
Moment, positive, y-direction	kip_ft	350.2	481.9	0.727	Pass
Shear, one-way, x-direction	kips	97.6	125.1	0.780	Pass
Shear, one-way, y-direction	kips	97.6	128.2	0.761	Pass
Shear, two-way, Col 1	psi	127.352	201.246	0.641	Pass
Min.area of reinf, bot., x-direction	in^2	5.184	5.280		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.2		Pass
Min.area of reinf, bot., y-direction	in^2	5.184	5.280		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.2		Pass

Pad footing details

Length of footing; L_x = **10** ft

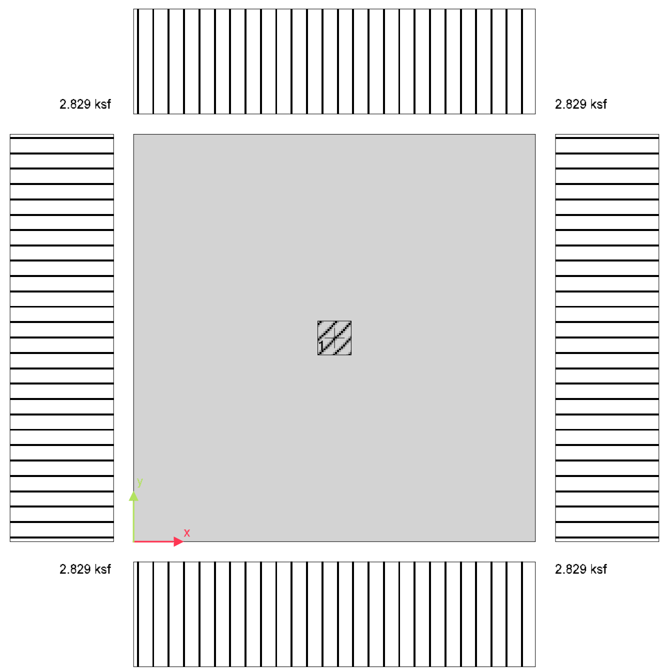
Width of footing; L_y = **10** ft

Footing area; $A = L_x \times L_y$ = **100** ft^2

Depth of footing; h = **24** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column;	l_{x1} = **10.00** in
Width of column;	l_{y1} = **10.00** in
position in x-axis;	x_1 = **60.00** in
position in y-axis;	y_1 = **60.00** in

Soil properties

Gross allowable bearing pressure;	q_{allow_Gross} = **3** ksf;
Density of soil;	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction;	ϕ_b = **30.0** deg
Design base friction angle;	δ_{bb} = **30.0** deg
Coefficient of base friction;	$\tan(\delta_{bb})$ = **0.577**
Self weight;	$F_{swt} = h \times \gamma_{conc}$ = **300** psf
Soil weight;	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z;	F_{Dz1} = **125.0** kips
Live load in z;	F_{Lz1} = **110.0** kips
Live roof load in z;	F_{Lrz1} = **23.9** kips
Snow load in z;	F_{Sz1} = **25.1** kips
Wind load in z;	F_{Wz1} = **-27.7** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.576)

1.0D + 1.0L (0.943)

1.0D + 1.0Lr (0.656)
1.0D + 0.7S (0.635)
1.0D + 1.0R (0.576)
1.0D + 0.75L + 0.75Lr (0.911)
1.0D + 0.75L + 0.525S (0.895)
1.0D + 0.75L + 0.75R (0.851)
1.0D + 0.6W (0.521)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.619)
1.0D + 0.75L + 0.75Lr + 0.45W (0.869)
1.0D + 0.75L + 0.525S + 0.45W (0.854)
1.0D + 0.75L + 0.75R + 0.45W (0.810)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.892)
0.6D + 0.6W (0.290)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.303)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **282.9** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **1414.4** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **1414.4** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **282.875** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.829** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.829** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.829** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.829** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.829** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.829** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} =$ **3** ksf

$q_{max} / q_{allow} =$ **0.943**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; f'_c = **4500** psi
Yield strength of reinforcement; f_y = **60000** psi
Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**
Cover to top of footing; c_{nom_t} = **3** in
Cover to side of footing; c_{nom_s} = **3** in
Cover to bottom of footing; c_{nom_b} = **3** in
Concrete type; Normal weight
Concrete modification factor; λ = **1.00**
Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.409)
1.2D + 1.6L + 0.5Lr (0.771)
1.2D + 1.6L + 0.3S (0.780)
1.2D + 1.6L + 0.5R (0.744)
1.2D + 1.0L + 1.6Lr (0.697)
1.2D + 1.0L + 1.0S (0.667)
1.2D + 1.0L + 1.6R (0.641)
1.2D + 1.6Lr + 0.5W (0.641)
1.2D + 1.0S + 0.5W (0.641)
1.2D + 1.6R + 0.5W (0.641)
1.2D + 1.0L + 0.5Lr + 1.0W (0.641)
1.2D + 1.0L + 0.3S + 1.0W (0.641)
1.2D + 1.0L + 0.5R + 1.0W (0.641)
$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.641)
0.9D + 1.0W (0.641)
$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.641)

Combination 3 results: 1.2D + 1.6L + 0.3S

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **391.0** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_S \times (F_{Sz1} \times x_1)$ = **1954.9** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **1954.9** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.91}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.91}$ ksf

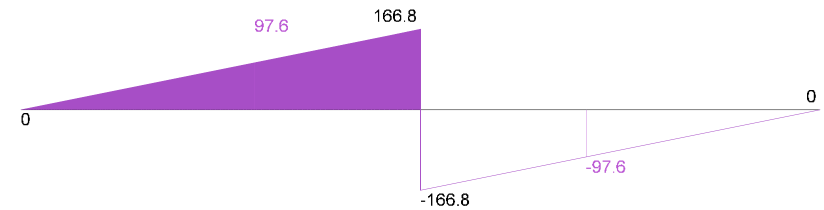
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.91}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.91}$ ksf

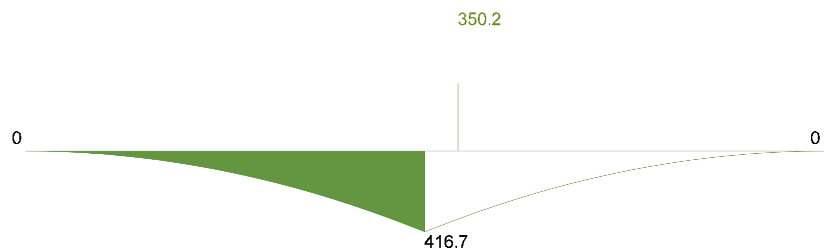
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.91}$ ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.91}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} = \mathbf{350.165}$ kip_ft

Tension reinforcement provided; 12 No.6 bottom bars (10.2 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} = \mathbf{5.28}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{5.184}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{20.625}$ in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.690}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.83}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{0.837}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.07096}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{535.389}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{481.85}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.727}$

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PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **97.583** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **20.625** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00213**

Shear strength reduction factor; ϕ_v = **0.75**

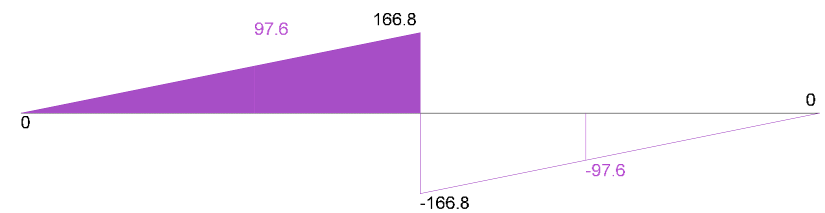
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v)$ = **170.985** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **128.239** kips

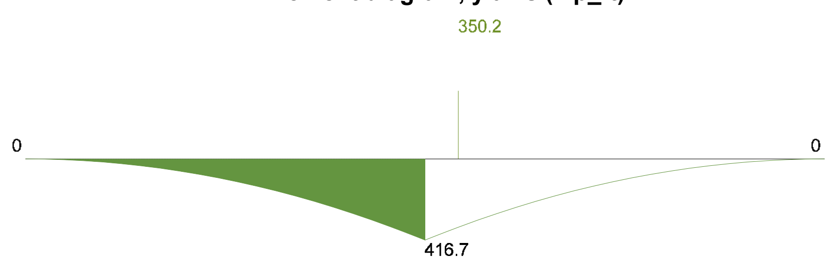
$V_{u.x} / \phi V_n$ = **0.761**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **350.165** kip_ft

Tension reinforcement provided; 12 No.6 bottom bars (10.2 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **5.28** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **5.184** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **19.875** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.690** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.837** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.06827**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **515.589** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **464.03** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.755**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} =$ **97.583** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **19.875** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00221**

Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v) =$ **166.814** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **125.111** kips

$V_{u.y} / \phi V_n =$ **0.780**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} =$ **20.25** in

Shear perimeter length (22.6.4); $l_{xp} =$ **30.250** in

Shear perimeter width (22.6.4); $l_{yp} =$ **30.250** in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **121.000** in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} =$ **915.062** in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **815.062** in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} =$ **3.910** ksf

Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **312.045** kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) =$ **127.352** psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3); $\alpha_s =$ **40**

Size effect factor (22.5.5.1.3); λ_s = **1**

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **402.492** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **583.226** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **268.328** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi

Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **268.328** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.633**

PASS - Design shear stress capacity exceeds ultimate shear stress load

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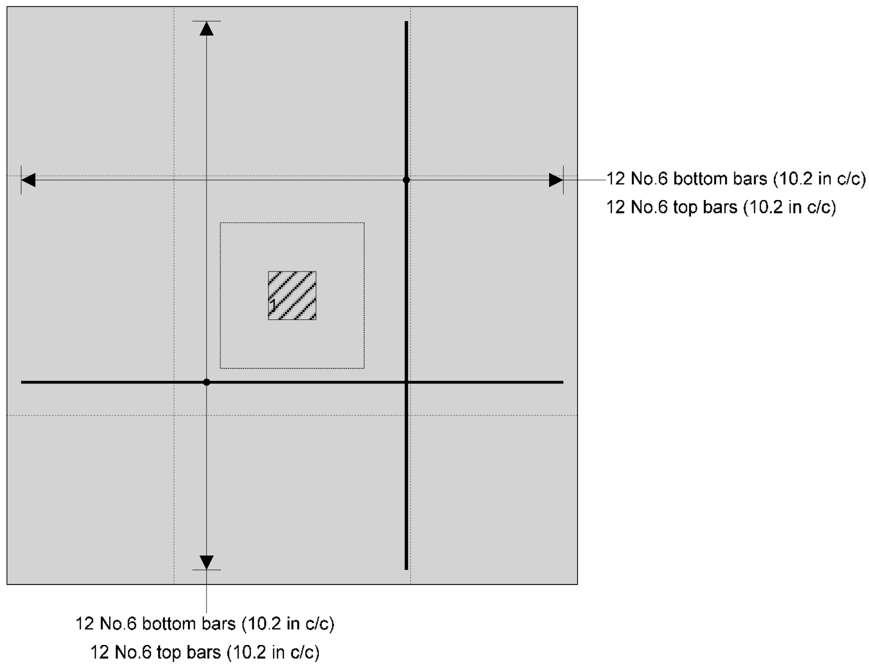

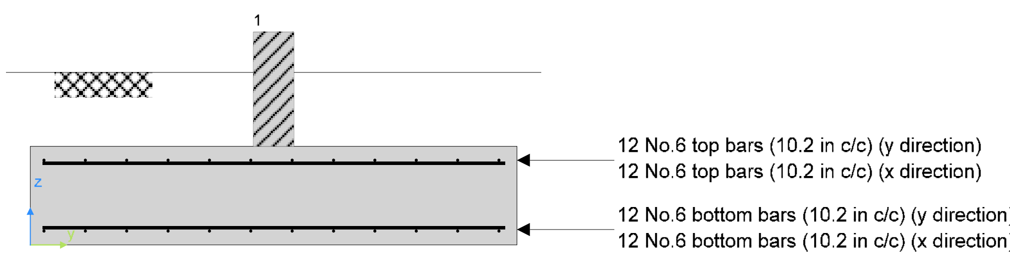

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COLUMN C 5.9

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.853

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	125.3			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.558	3	0.853	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	107.7	171.0	0.630	Pass
Moment, positive, y-direction	kip_ft	107.7	171.0	0.630	Pass
Shear, one-way, x-direction	kips	46.8	60.9	0.768	Pass
Shear, one-way, y-direction	kips	46.8	58.5	0.800	Pass
Shear, two-way, Col 1	psi	142.597	201.246	0.718	Pass
Min.area of reinf, bot., x-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	12.8		Pass
Min.area of reinf, bot., y-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	12.8		Pass

Pad footing details

Length of footing; L_x = **7** ft

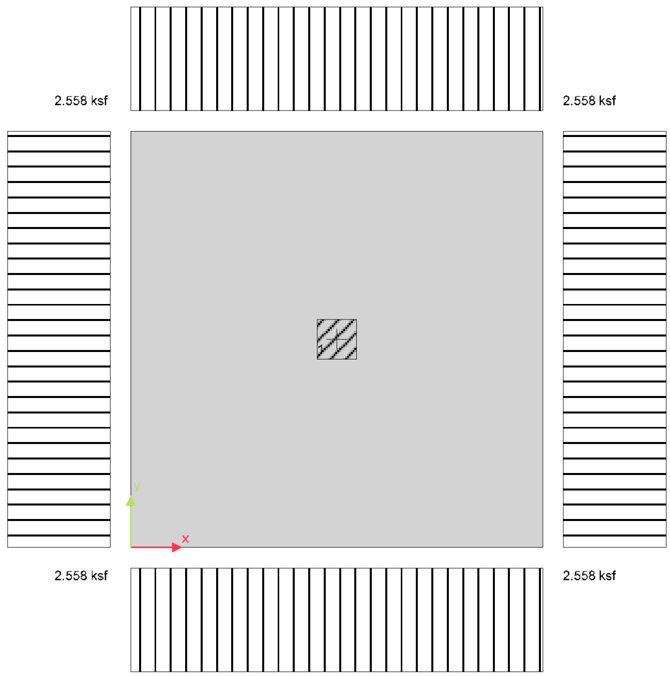
Width of footing; L_y = **7** ft

Footing area; $A = L_x \times L_y$ = **49** ft^2

Depth of footing; h = **16** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column; l_{x1} = **8.00** in

Width of column; l_{y1} = **8.00** in

position in x-axis; x_1 = **42.00** in

position in y-axis; y_1 = **42.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **200** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **60.6** kips

Live load in z; F_{Lz1} = **46.2** kips

Live roof load in z; F_{Lrz1} = **11.5** kips

Snow load in z; F_{Sz1} = **12.6** kips

Wind load in z; F_{Wz1} = **-13.5** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.538)

1.0D + 1.0L (0.853)

1.0D + 1.0Lr (0.617)
1.0D + 0.7S (0.598)
1.0D + 1.0R (0.538)
1.0D + 0.75L + 0.75Lr (0.833)
1.0D + 0.75L + 0.525S (0.819)
1.0D + 0.75L + 0.75R (0.774)
1.0D + 0.6W (0.483)
(1.0 + 0.14 × S_{DS})D + 0.7E (0.578)
1.0D + 0.75L + 0.75Lr + 0.45W (0.791)
1.0D + 0.75L + 0.525S + 0.45W (0.778)
1.0D + 0.75L + 0.75R + 0.45W (0.733)
(1.0 + 0.105 × S_{DS})D + 0.75L + 0.1S + 0.525E (0.812)
0.6D + 0.6W (0.268)
(0.6 - 0.14 × S_{DS})D + 0.7E (0.283)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **125.3** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **438.7** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **438.7** kip_ft

Uplift verification

Vertical force; F_{dz} = **125.34** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.558** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.558** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.558** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.558** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **2.558** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **2.558** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross}$ = **3** ksf

q_{max} / q_{allow} = **0.853**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete;	f'_c = **4500** psi
Yield strength of reinforcement;	f_y = **60000** psi
Compression-controlled strain limit (21.2.2);	ε_{ty} = **0.00200**
Cover to top of footing;	c_{nom_t} = **3** in
Cover to side of footing;	c_{nom_s} = **3** in
Cover to bottom of footing;	c_{nom_b} = **3** in
Concrete type;	Normal weight
Concrete modification factor;	λ = **1.00**
Column type;	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.451)
1.2D + 1.6L + 0.5Lr (0.778)
1.2D + 1.6L + 0.3S (0.800)
1.2D + 1.6L + 0.5R (0.780)
1.2D + 1.0L + 1.6Lr (0.718)
1.2D + 1.0L + 1.0S (0.718)
1.2D + 1.0L + 1.6R (0.718)
1.2D + 1.6Lr + 0.5W (0.718)
1.2D + 1.0S + 0.5W (0.718)
1.2D + 1.6R + 0.5W (0.718)
1.2D + 1.0L + 0.5Lr + 1.0W (0.718)
1.2D + 1.0L + 0.3S + 1.0W (0.718)
1.2D + 1.0L + 0.5R + 1.0W (0.718)
$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.718)
0.9D + 1.0W (0.718)
$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.718)

Combination 3 results: 1.2D + 1.6L + 0.3S

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **172.7** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_S \times (F_{Sz1} \times x_1)$ = **604.3** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **604.3** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.524}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.524}$ ksf

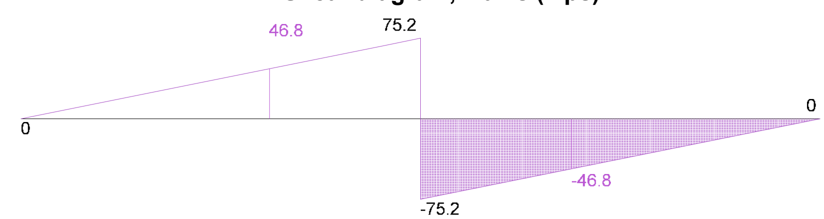
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.524}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.524}$ ksf

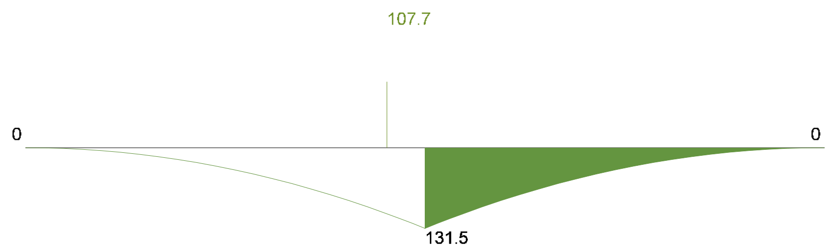
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.524}$ ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.524}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} = \mathbf{107.695}$ kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} = \mathbf{3.08}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{2.419}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{12.625}$ in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.575}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.83}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{0.697}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05133}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{189.996}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{170.997}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.630}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **46.76** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **12.625** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.0029**

Shear strength reduction factor; ϕ_v = **0.75**

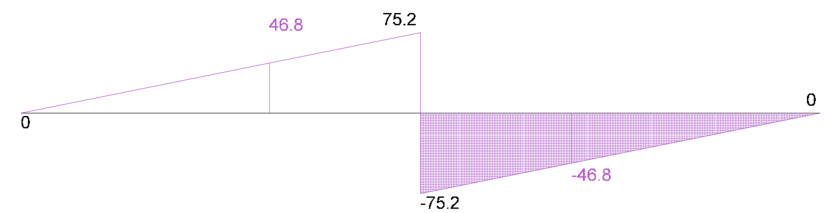
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v)$ = **81.2** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **60.9** kips

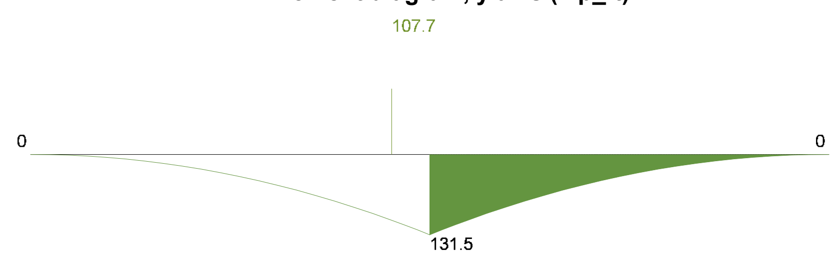
$V_{u.x} / \phi V_n$ = **0.768**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **107.695** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **2.419** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **11.875** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.575** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04810**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **178.446** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n =$ **160.602** kip_ft
	$M_{u.y.max} / \phi M_n =$ **0.671**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;	$V_{u.y} =$ **46.76** kips
Depth to reinforcement;	$d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **11.875** in
Size effect factor (22.5.5.1.3);	$\lambda_s =$ **1**
Ratio of longitudinal reinforcement;	$\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00309**
Shear strength reduction factor;	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v) =$ **77.951** kips
Design shear capacity;	$\phi V_n = \phi_v \times V_n =$ **58.463** kips
	$V_{u.y} / \phi V_n =$ **0.800**

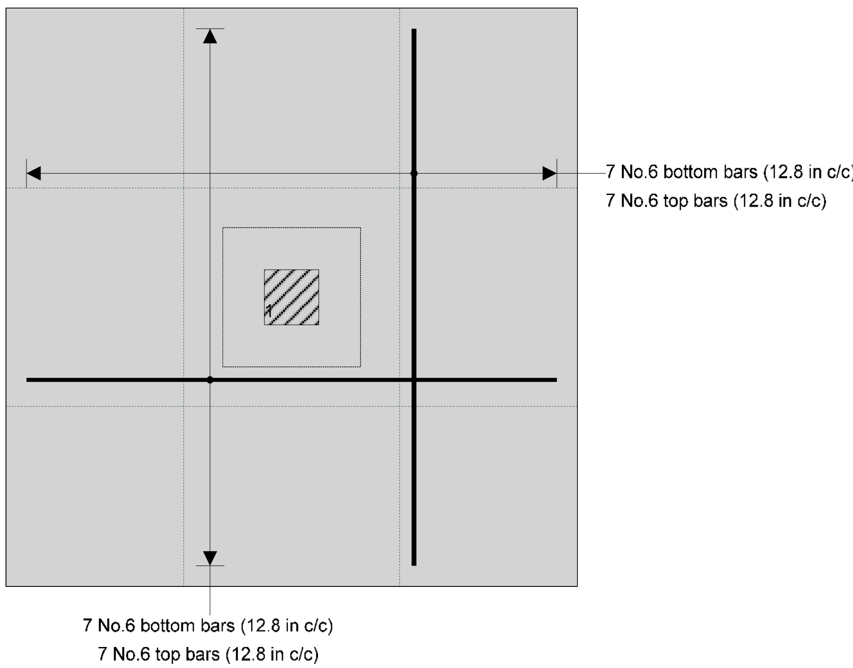
PASS - Design shear capacity exceeds ultimate shear load

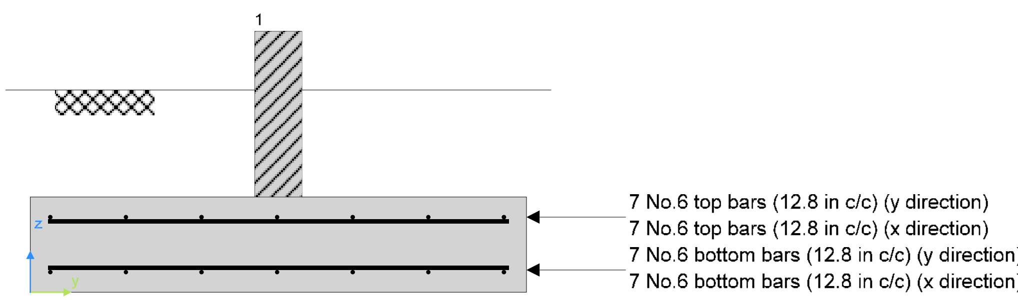
Two-way shear design at column 1

Depth to reinforcement;	$d_{v2} =$ **12.25** in
Shear perimeter length (22.6.4);	$l_{xp} =$ **20.250** in
Shear perimeter width (22.6.4);	$l_{yp} =$ **20.250** in
Shear perimeter (22.6.4);	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **81.000** in
Shear area;	$A_p = l_{x,perim} \times l_{y,perim} =$ **410.063** in^2
Surcharge loaded area;	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **346.063** in^2
Ultimate bearing pressure at center of shear area;	$q_{up.avg} =$ **3.524** ksf
Ultimate shear load;	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **141.492** kips
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) =$ **142.597** psi
Column geometry factor (Table 22.6.5.2);	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (22.6.5.3);	$\alpha_s =$ **40**
Size effect factor (22.5.5.1.3);	$\lambda_s =$ **1**
Concrete shear strength (22.6.5.2);	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **402.492** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **539.969** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **268.328** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi
Shear strength reduction factor;	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2);	$v_n = v_{cp} =$ **268.328** psi
Design shear stress capacity (8.5.1.1(d));	$\phi v_n = \phi_v \times v_n =$ **201.246** psi
	$v_{ug} / \phi v_n =$ **0.709**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Tekla Tedds Michael Baker International 7090 S Union Park Ave Midvale, UT 84107	Project Creech DRP PH 2				Job Ref.	
	Section Column C - 5.9				Sheet no./rev. 57	
	Calc. by ACV	Date 10/16/2025	Chk'd by	Date	App'd by	Date





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COLUMN A.9 4

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.799

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	239.7			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.397	3	0.799	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	288.2	481.9	0.598	Pass
Moment, positive, y-direction	kip_ft	288.2	481.9	0.598	Pass
Shear, one-way, x-direction	kips	80.3	128.2	0.626	Pass
Shear, one-way, y-direction	kips	80.3	128.2	0.626	Pass
Shear, two-way, Col 1	psi	104.811	201.246	0.521	Pass
Min.area of reinf, bot., x-direction	in^2	5.184	5.280		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.2		Pass
Min.area of reinf, bot., y-direction	in^2	5.184	5.280		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.2		Pass

Pad footing details

Length of footing; L_x = **10** ft

Width of footing; L_y = **10** ft

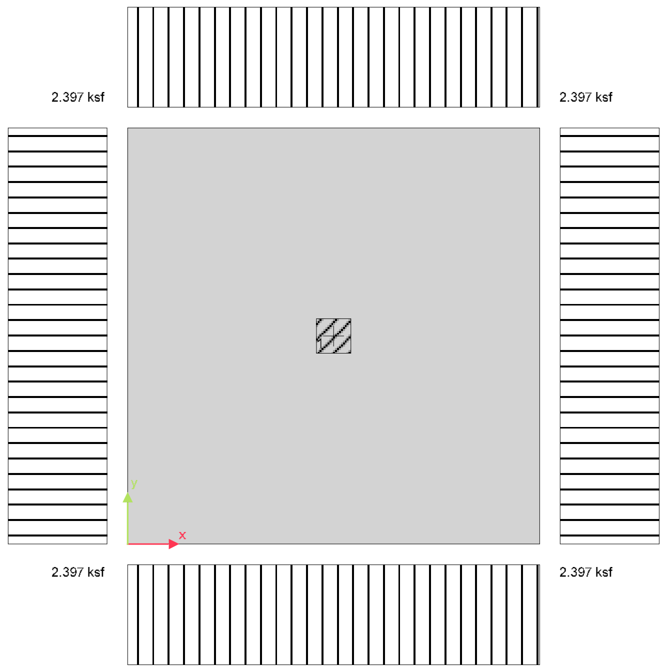
Footing area; $A = L_x \times L_y$ = **100** ft^2

Depth of footing; h = **24** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3

Tekla Tedds	Project				Job Ref.	
Michael Baker International 7090 S Union Park Ave Midvale, UT 84107	Creech DRP PH 2					
	Section Column A.9-5				Sheet no./rev. 59	
	Calc. by ACV	Date 10/16/2025	Chk'd by	Date	App'd by	Date



Column no.1 details

Length of column; l_{x1} = **10.00** in

Width of column; l_{y1} = **10.00** in

position in x-axis; x_1 = **60.00** in

position in y-axis; y_1 = **60.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **300** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **109.2** kips

Live load in z; F_{Lz1} = **82.6** kips

Live roof load in z; F_{Lrz1} = **22.7** kips

Snow load in z; F_{Sz1} = **23.8** kips

Wind load in z; F_{Wz1} = **-26.6** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.524)
1.0D + 1.0L (0.799)
1.0D + 1.0Lr (0.599)
1.0D + 0.7S (0.579)
1.0D + 1.0R (0.524)
1.0D + 0.75L + 0.75Lr (0.787)
1.0D + 0.75L + 0.525S (0.772)
1.0D + 0.75L + 0.75R (0.730)
1.0D + 0.6W (0.470)
(1.0 + 0.14 × S_{DS})D + 0.7E (0.562)
1.0D + 0.75L + 0.75Lr + 0.45W (0.747)
1.0D + 0.75L + 0.525S + 0.45W (0.732)
1.0D + 0.75L + 0.75R + 0.45W (0.690)
(1.0 + 0.105 × S_{DS})D + 0.75L + 0.1S + 0.525E (0.767)
0.6D + 0.6W (0.261)
(0.6 - 0.14 × S_{DS})D + 0.7E (0.276)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **239.7** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **1198.4** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **1198.4** kip_ft

Uplift verification

Vertical force; F_{dz} = **239.675** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.397** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.397** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.397** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.397** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **2.397** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **2.397** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} = \mathbf{3}$ ksf

$q_{max} / q_{allow} = \mathbf{0.799}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; $f'_c = \mathbf{4500}$ psi

Yield strength of reinforcement; $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2); $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to top of footing; $c_{nom\_t} = \mathbf{3}$ in

Cover to side of footing; $c_{nom\_s} = \mathbf{3}$ in

Cover to bottom of footing; $c_{nom\_b} = \mathbf{3}$ in

Concrete type; Normal weight

Concrete modification factor; $\lambda = \mathbf{1.00}$

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.349)

1.2D + 1.6L + 0.5Lr (0.626)

1.2D + 1.6L + 0.3S (0.617)

1.2D + 1.6L + 0.5R (0.615)

1.2D + 1.0L + 1.6Lr (0.584)

1.2D + 1.0L + 1.0S (0.555)

1.2D + 1.0L + 1.6R (0.521)

1.2D + 1.6Lr + 0.5W (0.521)

1.2D + 1.0S + 0.5W (0.521)

1.2D + 1.6R + 0.5W (0.521)

1.2D + 1.0L + 0.5Lr + 1.0W (0.521)

1.2D + 1.0L + 0.3S + 1.0W (0.521)

1.2D + 1.0L + 0.5R + 1.0W (0.521)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.522)

0.9D + 1.0W (0.521)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.521)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} = \mathbf{332.0}$ kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1)$ = **1660.0** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **1660.0** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.32** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.32** ksf

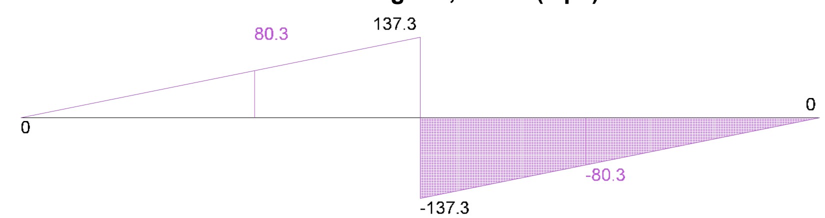
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.32** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.32** ksf

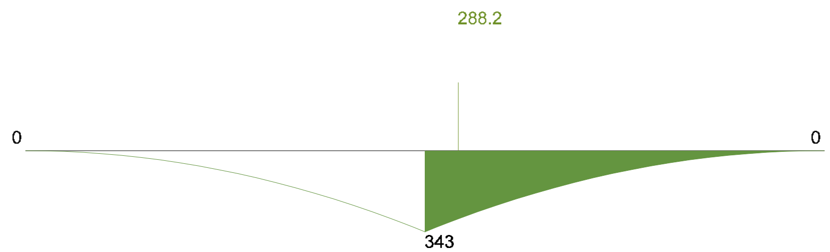
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.32** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.32** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max}$ = **288.215** kip_ft

Tension reinforcement provided; 12 No.6 bottom bars (10.2 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **5.28** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h$ = **5.184** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **20.625** in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.690** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.837** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.07096**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **535.389** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n$ = **481.85** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.598**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **80.319** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **20.625** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00213**

Shear strength reduction factor; ϕ_v = **0.75**

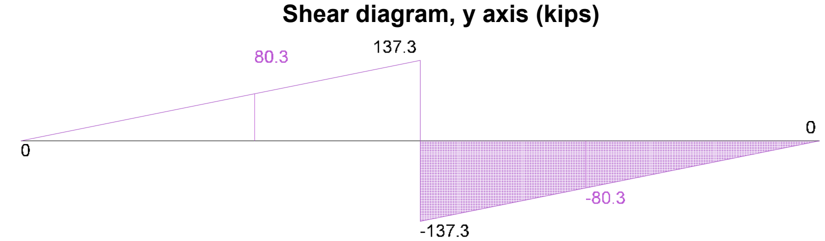
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v)$ = **170.985** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **128.239** kips

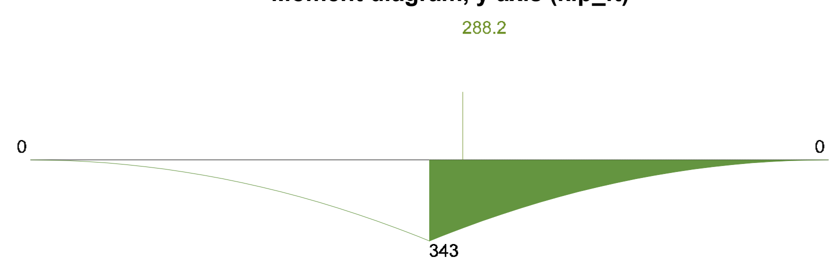
$V_{u.x} / \phi V_n$ = **0.626**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **288.215** kip_ft

Tension reinforcement provided; 12 No.6 bottom bars (10.2 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **5.28** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **5.184** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 =$ **19.875** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) =$ **0.690** in

Neutral axis factor; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.837** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.06827**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **515.589** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **464.03** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.621**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} =$ **80.319** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **19.875** in

Size effect factor (22.5.5.1.3); $\lambda_s =$ **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00221**

Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v) =$ **166.814** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **125.111** kips

$V_{u.y} / \phi V_n =$ **0.642**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} =$ **20.25** in

Shear perimeter length (22.6.4); $l_{xp} =$ **30.250** in

Shear perimeter width (22.6.4); $l_{yp} =$ **30.250** in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **121.000** in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} =$ **915.062** in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **815.062** in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} =$ **3.320** ksf

Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **256.813** kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi) =$ **104.811** psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3); $\alpha_s =$**40**

Size effect factor (22.5.5.1.3); $\lambda_s =$ **1**

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **402.492** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **583.226** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **268.328** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi

Tekla Tedds	Project				Job Ref.	
Michael Baker International 7090 S Union Park Ave Midvale, UT 84107	Creech DRP PH 2					
	Section Column A.9-5				Sheet no./rev. 65	
	Calc. by ACV	Date 10/16/2025	Chk'd by	Date	App'd by	Date

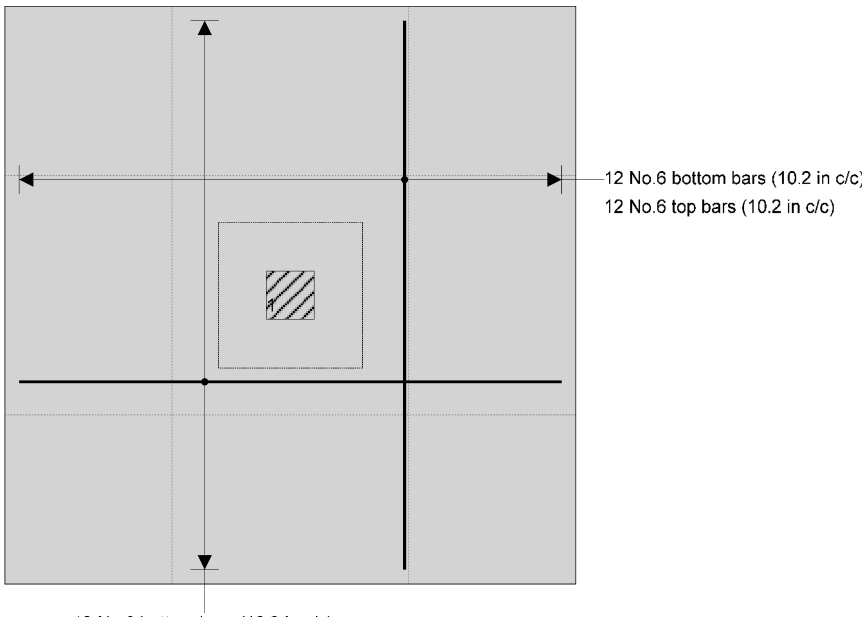
Shear strength reduction factor; ϕ_v = **0.75**

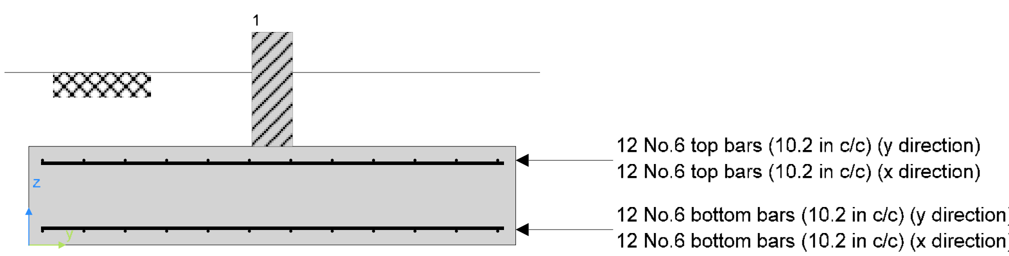
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp}$ = **268.328** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n$ = **201.246** psi

$v_{ug} / \phi v_n$ = **0.521**

PASS - Design shear stress capacity exceeds ultimate shear stress load





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COLUMN A.9 4.1

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.953

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	102.9			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.858	3	0.953	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	76.8	142.6	0.539	Pass
Moment, positive, y-direction	kip_ft	76.8	142.6	0.539	Pass
Shear, one-way, x-direction	kips	39.8	46.7	0.854	Pass
Shear, one-way, y-direction	kips	39.8	46.7	0.854	Pass
Shear, two-way, Col 1	psi	161.965	201.246	0.805	Pass
Min.area of reinf, bot., x-direction	in^2	1.814	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.8		Pass
Min.area of reinf, bot., y-direction	in^2	1.814	3.080		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.8		Pass

Pad footing details

Length of footing; L_x = **6** ft

Width of footing; L_y = **6** ft

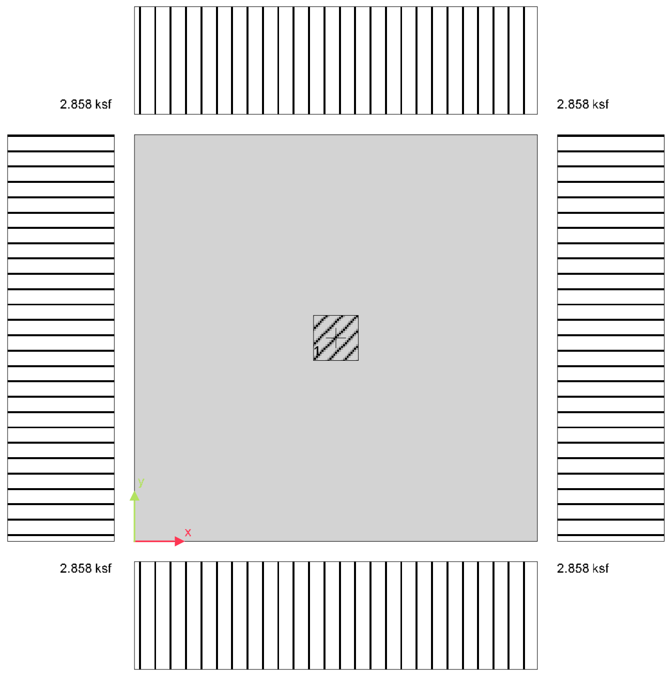
Footing area; $A = L_x \times L_y$ = **36** ft^2

Depth of footing; h = **14** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3

Tekla Tedds	Project				Job Ref.	
Michael Baker International 7090 S Union Park Ave Midvale, UT 84107	Creech DRP PH 2					
	Section				Sheet no./rev.	
	Column A.9 - 4.1				67	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	ACV	10/16/2025				



Column no.1 details

Length of column; l_{x1} = **8.00** in

Width of column; l_{y1} = **8.00** in

position in x-axis; x_1 = **36.00** in

position in y-axis; y_1 = **36.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **175** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **48.4** kips

Live load in z; F_{Lz1} = **41.8** kips

Live roof load in z; F_{Lrz1} = **9.5** kips

Snow load in z; F_{Sz1} = **10.9** kips

Wind load in z; F_{Wz1} = **-11.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.566)

1.0D + 1.0L (0.953)

1.0D + 1.0Lr (0.654)

1.0D + 0.7S (0.636)

1.0D + 1.0R (0.566)

1.0D + 0.75L + 0.75Lr (0.922)

1.0D + 0.75L + 0.525S (0.909)

1.0D + 0.75L + 0.75R (0.856)

1.0D + 0.6W (0.504)

$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.607)

1.0D + 0.75L + 0.75Lr + 0.45W (0.875)

1.0D + 0.75L + 0.525S + 0.45W (0.862)

1.0D + 0.75L + 0.75R + 0.45W (0.809)

$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.897)

0.6D + 0.6W (0.277)

$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.298)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **102.9** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **308.7** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **308.7** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **102.9** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.858** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.858** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.858** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.858** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.858** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.858** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} = \mathbf{3}$ ksf

$q_{max} / q_{allow} = \mathbf{0.953}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; $f'_c = \mathbf{4500}$ psi

Yield strength of reinforcement; $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2); $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to top of footing; $c_{nom\_t} = \mathbf{3}$ in

Cover to side of footing; $c_{nom\_s} = \mathbf{3}$ in

Cover to bottom of footing; $c_{nom\_b} = \mathbf{3}$ in

Concrete type; Normal weight

Concrete modification factor; $\lambda = \mathbf{1.00}$

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.446)

1.2D + 1.6L + 0.5Lr (0.854)

1.2D + 1.6L + 0.3S (0.844)

1.2D + 1.6L + 0.5R (0.823)

1.2D + 1.0L + 1.6Lr (0.805)

1.2D + 1.0L + 1.0S (0.805)

1.2D + 1.0L + 1.6R (0.805)

1.2D + 1.6Lr + 0.5W (0.805)

1.2D + 1.0S + 0.5W (0.805)

1.2D + 1.6R + 0.5W (0.805)

1.2D + 1.0L + 0.5Lr + 1.0W (0.805)

1.2D + 1.0L + 0.3S + 1.0W (0.805)

1.2D + 1.0L + 0.5R + 1.0W (0.805)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.805)

0.9D + 1.0W (0.805)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.805)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} = \mathbf{145.0}$ kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1)$ = **434.8** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **434.8** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.026** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.026** ksf

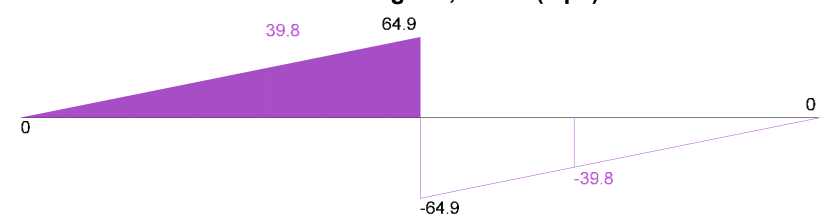
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.026** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.026** ksf

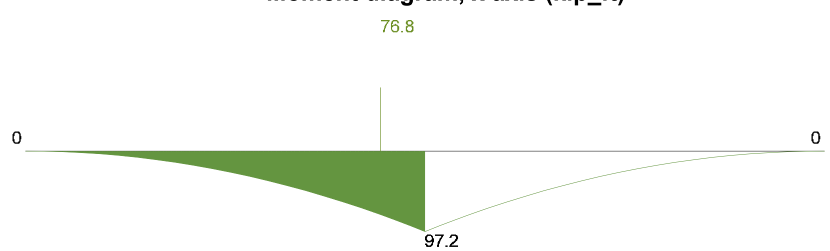
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **4.026** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **4.026** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max}$ = **76.827** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h$ = **1.814** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **10.625** in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.671** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.813** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03619**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **158.458** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **142.612** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.539**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x} =$ **39.836** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2) =$ **10.625** in

Size effect factor (22.5.5.1.3); $\lambda_s =$ **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v) =$ **0.00403**

Shear strength reduction factor; $\phi_v =$ **0.75**

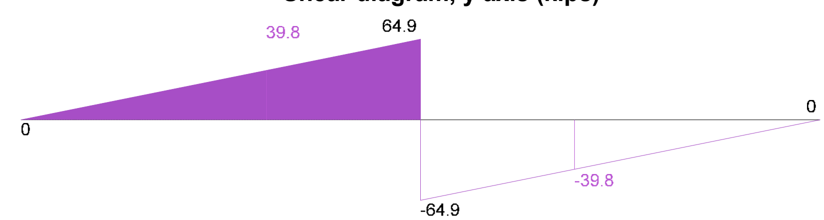
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v) =$ **65.311** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **48.983** kips

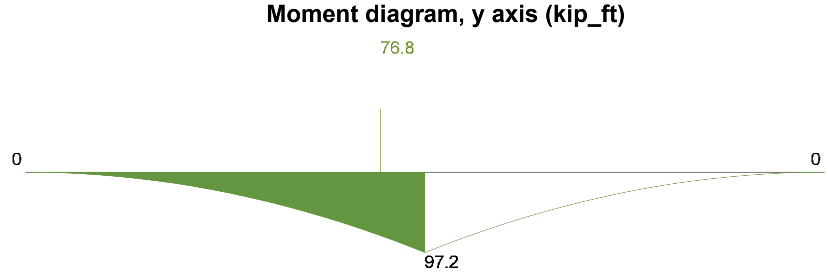
$V_{u.x} / \phi V_n =$ **0.813**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max} =$ **76.827** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} =$ **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h =$ **1.814** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = \mathbf{9.875}$ in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{0.671}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.83}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{0.813}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.03342}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = \mathbf{146.908}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{132.217}$ kip_ft

$M_{u.y.max} / \phi M_n = \mathbf{0.581}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} = \mathbf{39.836}$ kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) = \mathbf{9.875}$ in

Size effect factor (22.5.5.1.3); $\lambda_s = \mathbf{1}$

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) = \mathbf{0.00433}$

Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v) = \mathbf{62.2}$ kips

Design shear capacity; $\phi V_n = \phi_v \times V_n = \mathbf{46.65}$ kips

$V_{u.y} / \phi V_n = \mathbf{0.854}$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} = \mathbf{10.25}$ in

Shear perimeter length (22.6.4); $l_{xp} = \mathbf{18.250}$ in

Shear perimeter width (22.6.4); $l_{yp} = \mathbf{18.250}$ in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{73.000}$ in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} = \mathbf{333.062}$ in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{269.062}$ in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} = \mathbf{4.026}$ ksf

Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{121.191}$ kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi) = \mathbf{161.965}$ psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} = \mathbf{1.00}$

Column location factor (22.6.5.3); $\alpha_s = \mathbf{40}$

Size effect factor (22.5.5.1.3); $\lambda_s = \mathbf{1}$

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{402.492}$ psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{510.926}$ psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{268.328}$ psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{268.328}$ psi

Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

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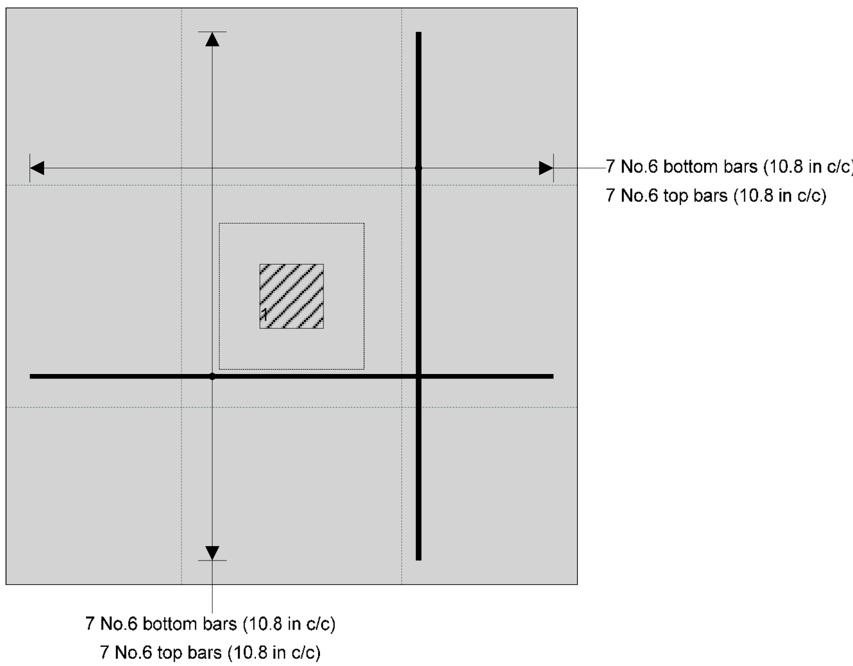
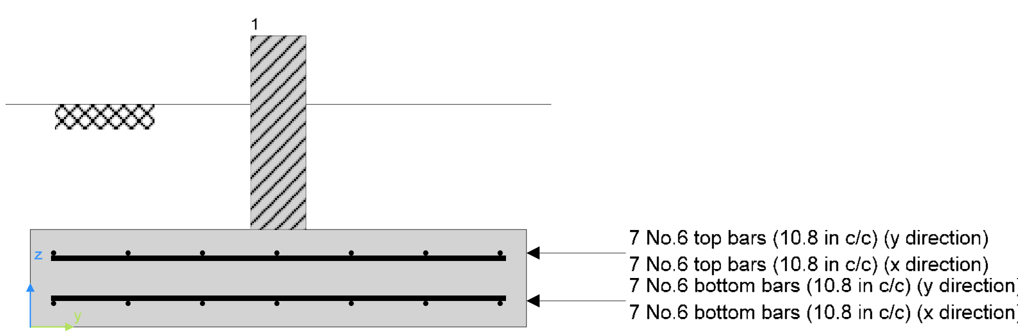
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **268.328** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.805**

PASS - Design shear stress capacity exceeds ultimate shear stress load





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COLUMN A.9 4.9

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status; PASS;

Overall design utilisation; 0.674

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	99.0			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.021	3	0.674	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	81.1	160.6	0.505	Pass
Moment, positive, y-direction	kip_ft	81.1	171.0	0.475	Pass
Shear, one-way, x-direction	kips	35.2	60.9	0.579	Pass
Shear, one-way, y-direction	kips	35.2	58.5	0.603	Pass
Shear, two-way, Col 1	psi	107.418	201.246	0.542	Pass
Min.area of reinf, bot., x-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	12.8		Pass
Min.area of reinf, bot., y-direction	in^2	2.419	3.080		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	12.8		Pass

Pad footing details

Length of footing; L_x = **7** ft

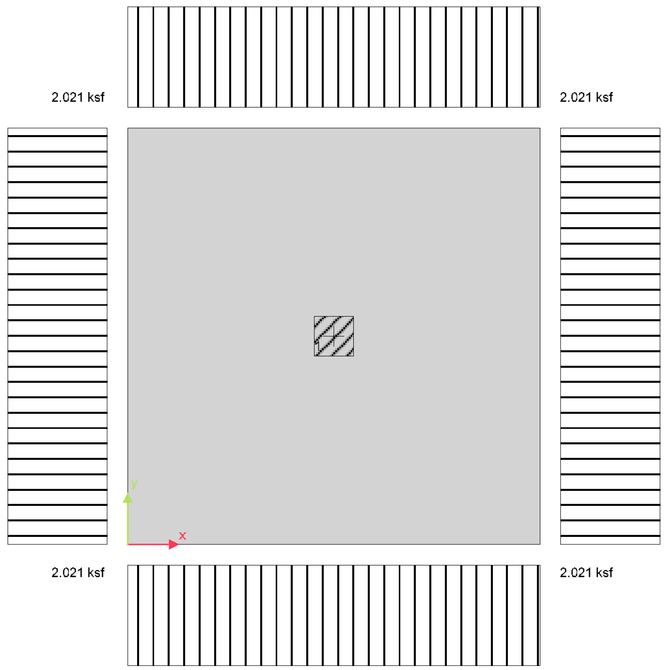
Width of footing; L_y = **7** ft

Footing area; $A = L_x \times L_y$ = **49** ft^2

Depth of footing; h = **16** in

Depth of soil over footing; h_{soil} = **18** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column; l_{x1} = **8.00** in

Width of column; l_{y1} = **8.00** in

position in x-axis; x_1 = **42.00** in

position in y-axis; y_1 = **42.00** in

Soil properties

Gross allowable bearing pressure; q_{allow_Gross} = **3** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **200** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z; F_{Dz1} = **46.1** kips

Live load in z; F_{Lz1} = **34.4** kips

Live roof load in z; F_{Lrz1} = **9.4** kips

Snow load in z; F_{Sz1} = **10.0** kips

Wind load in z; F_{Wz1} = **-11.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.440)

1.0D + 1.0L (0.674)

1.0D + 1.0Lr (0.504)
1.0D + 0.7S (0.487)
1.0D + 1.0R (0.440)
1.0D + 0.75L + 0.75Lr (0.663)
1.0D + 0.75L + 0.525S (0.651)
1.0D + 0.75L + 0.75R (0.615)
1.0D + 0.6W (0.395)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.472)
1.0D + 0.75L + 0.75Lr + 0.45W (0.630)
1.0D + 0.75L + 0.525S + 0.45W (0.617)
1.0D + 0.75L + 0.75R + 0.45W (0.582)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.646)
0.6D + 0.6W (0.219)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.231)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **99.0** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **346.6** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **346.6** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **99.04** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in
Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.021** ksf
$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.021** ksf
$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.021** ksf
$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.021** ksf
Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.021** ksf
Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.021** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} =$ **3** ksf
$q_{max} / q_{allow} =$ **0.674**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete; f'_c = **4500** psi

Yield strength of reinforcement; f_y = **60000** psi

Compression-controlled strain limit (21.2.2); ε_{ty} = **0.00200**

Cover to top of footing; c_{nom_t} = **3** in

Cover to side of footing; c_{nom_s} = **3** in

Cover to bottom of footing; c_{nom_b} = **3** in

Concrete type; Normal weight

Concrete modification factor; λ = **1.00**

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.343)
1.2D + 1.6L + 0.5Lr (0.587)
1.2D + 1.6L + 0.3S (0.603)
1.2D + 1.6L + 0.5R (0.563)
1.2D + 1.0L + 1.6Lr (0.557)
1.2D + 1.0L + 1.0S (0.542)
1.2D + 1.0L + 1.6R (0.542)
1.2D + 1.6Lr + 0.5W (0.542)
1.2D + 1.0S + 0.5W (0.542)
1.2D + 1.6R + 0.5W (0.542)
1.2D + 1.0L + 0.5Lr + 1.0W (0.542)
1.2D + 1.0L + 0.3S + 1.0W (0.542)
1.2D + 1.0L + 0.5R + 1.0W (0.542)
$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.542)
0.9D + 1.0W (0.542)
$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.542)

Combination 3 results: 1.2D + 1.6L + 0.3S

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **135.6** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_S \times (F_{Sz1} \times x_1)$ = **474.6** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **474.6** kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.768}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.768}$ ksf

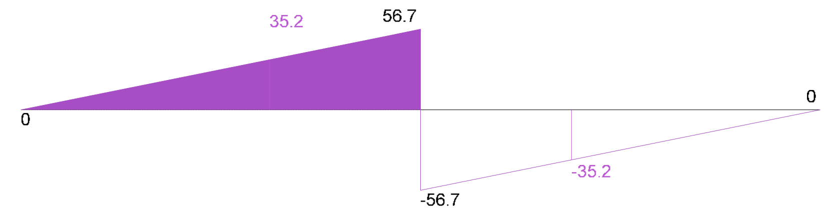
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.768}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{2.768}$ ksf

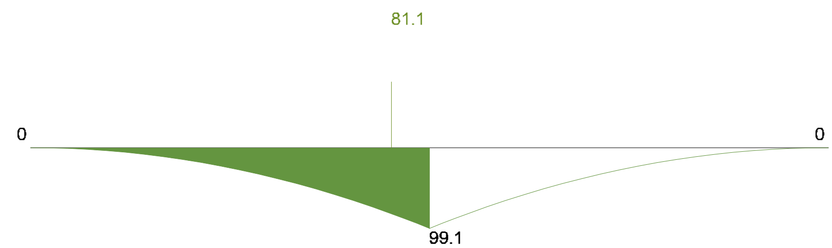
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{2.768}$ ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{2.768}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} = \mathbf{81.144}$ kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} = \mathbf{3.08}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{2.419}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{12.625}$ in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.575}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.83}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{0.697}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05133}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{189.996}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{170.997}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.475}$

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PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x}$ = **35.232** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **12.625** in

Size effect factor (22.5.5.1.3); λ_s = **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.0029**

Shear strength reduction factor; ϕ_v = **0.75**

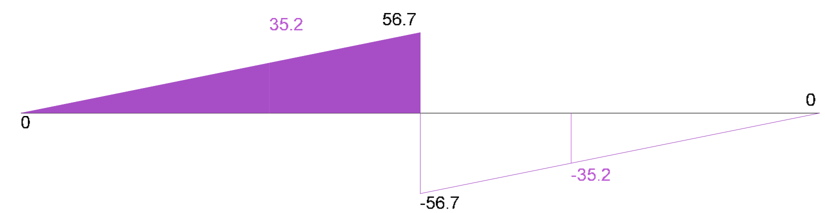
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v)$ = **81.2** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **60.9** kips

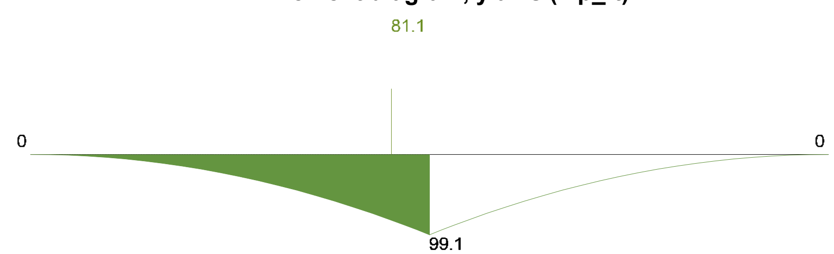
$V_{u.x} / \phi V_n$ = **0.579**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **81.144** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **2.419** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **11.875** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.575** in

Neutral axis factor; β_1 = **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04810**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **178.446** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **160.602** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.505**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} =$ **35.232** kips

Depth to reinforcement; $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **11.875** in

Size effect factor (22.5.5.1.3); $\lambda_s =$ **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00309**

Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v) =$ **77.951** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **58.463** kips

$V_{u.y} / \phi V_n =$ **0.603**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} =$ **12.25** in

Shear perimeter length (22.6.4); $l_{xp} =$ **20.250** in

Shear perimeter width (22.6.4); $l_{yp} =$ **20.250** in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **81.000** in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} =$ **410.063** in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **346.063** in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} =$ **2.768** ksf

Ultimate shear load; $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **106.586** kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi}) =$ **107.418** psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3); $\alpha_s =$ **40**

Size effect factor (22.5.5.1.3); $\lambda_s =$ **1**

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **402.492** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **539.969** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **268.328** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi

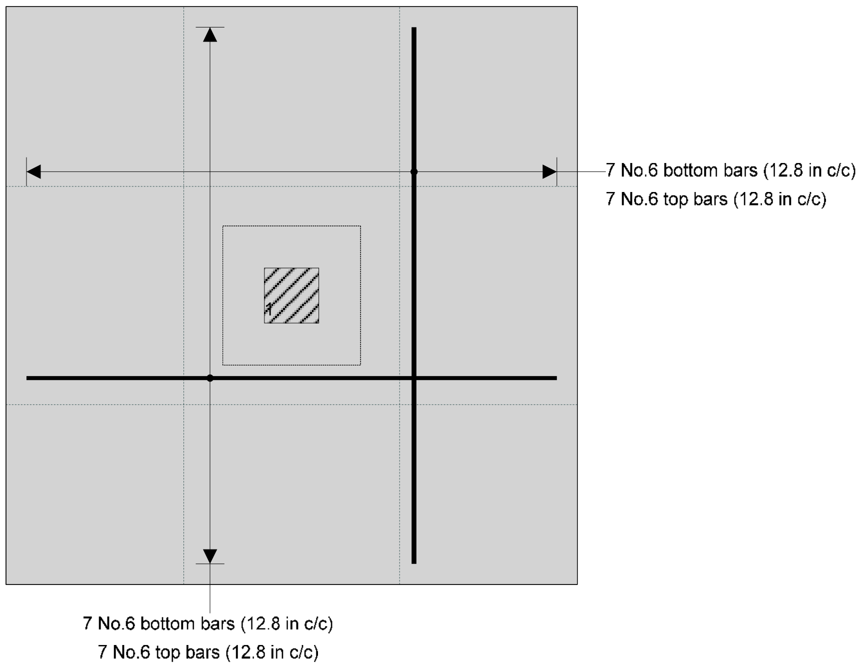
Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **268.328** psi

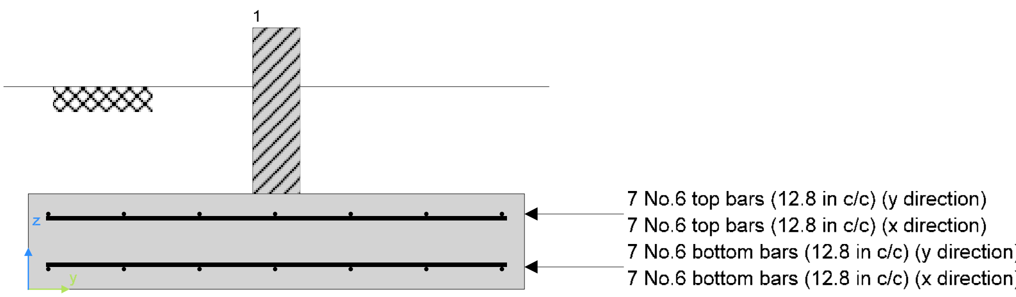
Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.534**

PASS - Design shear stress capacity exceeds ultimate shear stress load



7 No.6 bottom bars (12.8 in c/c)
7 No.6 top bars (12.8 in c/c)
7 No.6 bottom bars (12.8 in c/c)
7 No.6 top bars (12.8 in c/c)



1
7 No.6 top bars (12.8 in c/c) (y direction)
7 No.6 top bars (12.8 in c/c) (x direction)
7 No.6 bottom bars (12.8 in c/c) (y direction)
7 No.6 bottom bars (12.8 in c/c) (x direction)

;

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status PASS

Overall design utilisation 0.755

Description	**Unit**	**Applied**	**Resisting**	**FoS**	**Result**
Uplift verification	kips	36.2			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.265	3	0.755	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	16.2	47.3	0.343	Pass
Moment, positive, y-direction	kip_ft	16.2	47.3	0.343	Pass
Shear, one-way, x-direction	kips	11.4	24.1	0.473	Pass
Shear, one-way, y-direction	kips	11.4	24.1	0.473	Pass
Shear, two-way, Col 1	psi	79.936	201.246	0.397	Pass
Min.area of reinf, bot., x-direction	in^2	1.037	1.240		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	13.7		Pass
Min.area of reinf, bot., y-direction	in^2	1.037	1.240		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	13.7		Pass

Pad footing details

Length of footing L_x = **4** ft

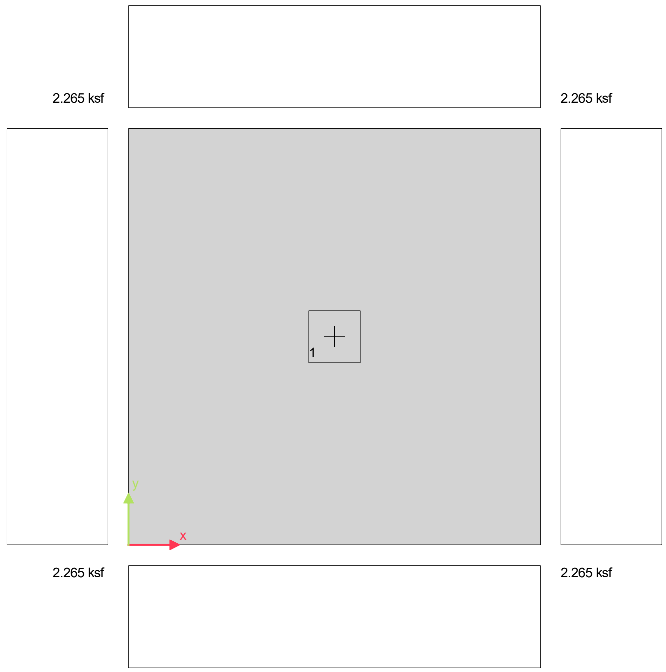
Width of footing L_y = **4** ft

Footing area $A = L_x \times L_y$ = **16** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **6.00** in

Width of column l_{y1} = **6.00** in

position in x-axis x_1 = **24.00** in

position in y-axis y_1 = **24.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **3** ksf

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z F_{Dz1} = **18.0** kips

Live load in z F_{Lz1} = **13.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.484)

1.0D + 1.0L (0.755)

1.0D + 1.0Lr (0.484)

1.0D + 0.7S (0.484)
1.0D + 1.0R (0.484)
1.0D + 0.75L + 0.75Lr (0.687)
1.0D + 0.75L + 0.525S (0.687)
1.0D + 0.75L + 0.75R (0.687)
1.0D + 0.6W (0.484)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.520)
1.0D + 0.75L + 0.75Lr + 0.45W (0.687)
1.0D + 0.75L + 0.525S + 0.45W (0.687)
1.0D + 0.75L + 0.75R + 0.45W (0.687)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.714)
0.6D + 0.6W (0.290)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.255)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{36.2}$ kips

Moments on footing

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{72.5}$ kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{72.5}$ kip_ft

Uplift verification

Vertical force — $F_{dz} = \mathbf{36.235}$ kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0}$ in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.265}$ ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.265}$ ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.265}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{2.265}$ ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{2.265}$ ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{2.265}$ ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} = \mathbf{3}$ ksf

$q_{max} / q_{allow} = \mathbf{0.755}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Material details

Compressive strength of concrete — f'_c = **4500** psi

Yield strength of reinforcement — f_y = **60000** psi

Compression-controlled strain limit (21.2.2) — ε_{ty} = **0.00200**

Cover to top of footing — c_{nom_t} = **3** in

Cover to side of footing — c_{nom_s} = **3** in

Cover to bottom of footing — c_{nom_b} = **3** in

Concrete type — Normal weight

Concrete modification factor — λ = **1.00**

Column type — Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.295)

1.2D + 1.6L + 0.5Lr (0.473)

1.2D + 1.6L + 0.3S (0.473)

1.2D + 1.6L + 0.5R (0.473)

1.2D + 1.0L + 1.6Lr (0.406)

1.2D + 1.0L + 1.0S (0.406)

1.2D + 1.0L + 1.6R (0.406)

1.2D + 1.6Lr + 0.5W (0.397)

1.2D + 1.0S + 0.5W (0.397)

1.2D + 1.6R + 0.5W (0.397)

1.2D + 1.0L + 0.5Lr + 1.0W (0.406)

1.2D + 1.0L + 0.3S + 1.0W (0.406)

1.2D + 1.0L + 0.5R + 1.0W (0.406)

$(1.2 + 0.2 \times S_{DS})$D + 1.0L + 0.15S + 1.0E (0.428)

0.9D + 1.0W (0.397)

$(0.9 - 0.2 \times S_{DS})$D + 1.0E (0.397)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} = \mathbf{48.7}$ kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{97.4}$ kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{97.4}$ kip_ft

Eccentricity of base reaction

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.043}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.043}$ ksf

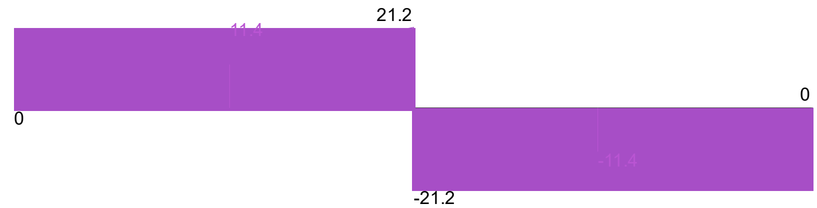
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.043}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{3.043}$ ksf

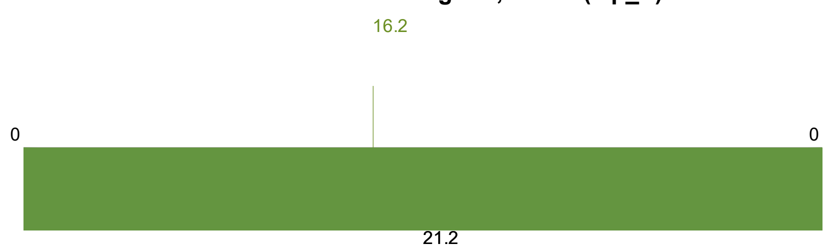
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.043}$ ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{3.043}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} = \mathbf{16.223}$ kip_ft

Tension reinforcement provided 4 No.5 bottom bars (13.7 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} = \mathbf{1.24}$ in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{1.037}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{8.688}$ in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.405}$ in

Neutral axis factor $\beta_1 = \mathbf{0.83}$

Depth to neutral axis $c = a / \beta_1 = \mathbf{0.491}$ in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.05006}$

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{52.606}$ kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity $\phi M_n = \phi_f \times M_n = \mathbf{47.346}$ kip_ft

$M_{u.x.max} / \phi M_n$ = **0.343**

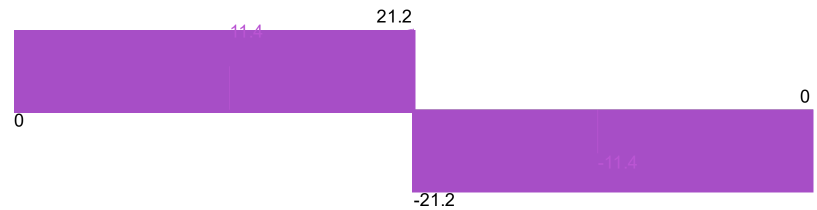
PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

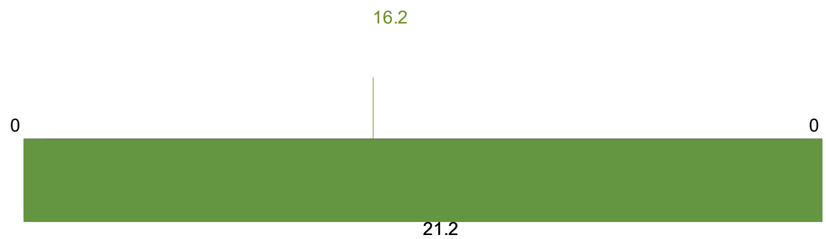
Ultimate shear force	$V_{u.x}$ = **11.421** kips
Depth to reinforcement	$d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2)$ = **8.688** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00297**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v)$ = **32.181** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **24.135** kips
	$V_{u.x} / \phi V_n$ = **0.473**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **16.223** kip_ft
Tension reinforcement provided	4 No.5 bottom bars (13.7 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **1.24** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **1.037** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **8.062** in
Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.405** in
Neutral axis factor	β_1 = **0.83**
Depth to neutral axis	$c = a / \beta_1$ = **0.491** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04624**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **48.731** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **43.858** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.370**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y} =$ **11.421** kips

Depth to reinforcement $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) =$ **8.062** in

Size effect factor (22.5.5.1.3) $\lambda_s = $ **1**

Ratio of longitudinal reinforcement $\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.0032**

Shear strength reduction factor $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v) =$ **30.618** kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **22.964** kips

$V_{u.y} / \phi V_n =$ **0.497**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement $d_{v2} =$ **8.375** in

Shear perimeter length (22.6.4) $l_{xp} =$ **14.375** in

Shear perimeter width (22.6.4) $l_{yp} =$ **14.375** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **57.500** in

Shear area $A_p = l_{x,perim} \times l_{y,perim} =$ **206.641** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **170.641** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **3.043** ksf

Ultimate shear load $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **38.494** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) =$ **79.936** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3) $\alpha_s =$ **40**

Size effect factor (22.5.5.1.3) $\lambda_s = $ **1**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **402.492** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **524.990** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **268.328** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **268.328** psi

Shear strength reduction factor $\phi_v =$ **0.75**

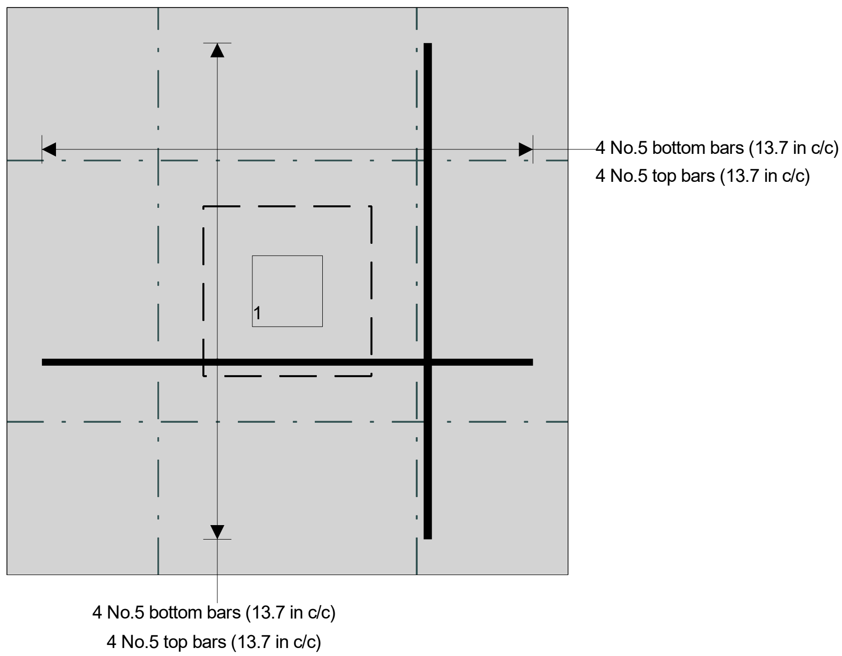
Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp} =$ **268.328** psi

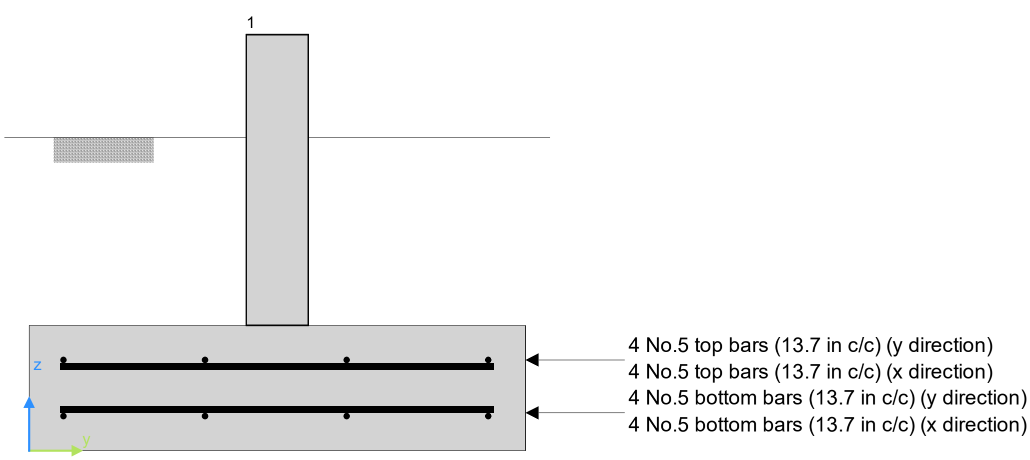
Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.397**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Tekla Tedds Michael Baker Internation 7090 S Union Park Ave Midvale, UT 84107	Project Creech DRP PH 2				Job Ref.	
	Section Column A.1 - 4.7				Sheet no./rev. 8	
	Calc. by ACV	Date 11/7/2025	Chk'd by	Date	App'd by	Date





Tekla Tedds	Project				Job Ref.	
Michael Baker Internation 7090 S Union Park Ave Midvale, UT 84107	Creech DRP PH 2					
	Section				Sheet no./rev.	
	Continuous Footing Admin				1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	ACV	10/17/2025				

FOOTING ANALYSIS

In accordance with ACI318-

Tedds calculation version 3.3.11

Summary results

Overall design status PASS

Overall design utilisation 0.997

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	1.4			Pass
Overturning stability, y	kip_ft	1.07	-2.80	2.61	Pass
Sliding stability, y	kips	0.8	0.8	1.003	Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	0.756	3	0.252	Pass

Strip footing details - considering a one foot strip

Length of footing L_x = **1** ft

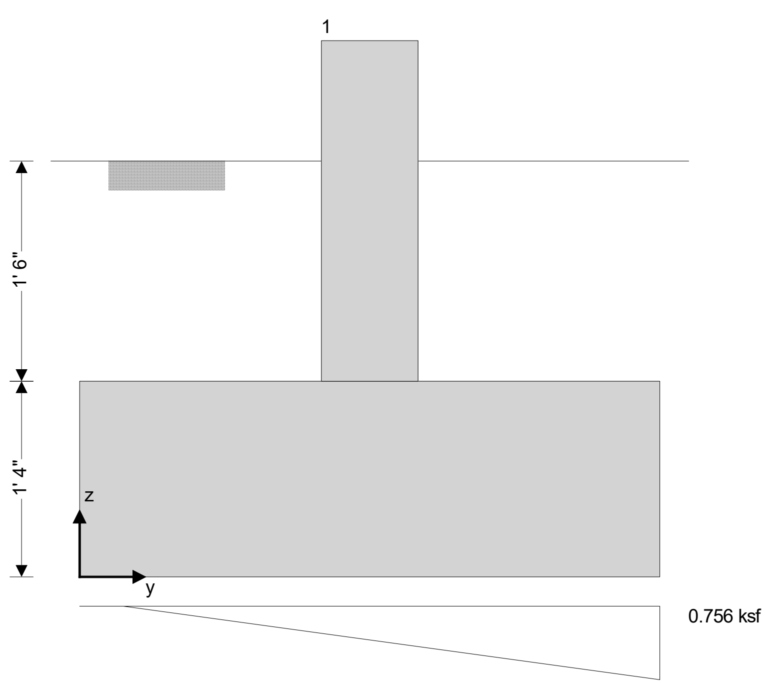
Width of footing L_y = **4** ft

Footing area $A = L_x \times L_y$ = **4** ft^2

Depth of footing h = **16** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Wall no.1 details

Width of wall l_{y1} = **8** in

position in y-axis y_1 = **24** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **3** ksf

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **200** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Wall no.1 loads per linear foot

Dead load in z F_{Dz1} = **1.3** kips

Live load in z F_{Lz1} = **1.8** kips

Live roof load in z F_{Lrz1} = **0.2** kips

Snow load in z F_{Sz1} = **0.3** kips

Wind load in y F_{Wy1} = **0.8** kips

Seismic load in y F_{Ey1} = **1.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.222)

1.0D + 1.0L (0.372)

1.0D + 1.0Lr (0.242)

1.0D + 0.7S (0.236)

1.0D + 1.0R (0.222)

1.0D + 0.75L + 0.75Lr (0.349)

1.0D + 0.75L + 0.525S (0.345)

1.0D + 0.75L + 0.75R (0.334)

1.0D + 0.6W (0.301)

$(1.0 + 0.14 \times S_{DS})$D + 0.7E (0.488)

1.0D + 0.75L + 0.75Lr + 0.45W (0.407)

1.0D + 0.75L + 0.525S + 0.45W (0.403)

1.0D + 0.75L + 0.75R + 0.45W (0.392)

$(1.0 + 0.105 \times S_{DS})$D + 0.75L + 0.1S + 0.525E (0.449)

0.6D + 0.6W (0.501)

$(0.6 - 0.14 \times S_{DS})$D + 0.7E (0.997)

Combination 16 results: $(0.6 - 0.14 \times S_{DS})$D + 0.7E

Forces on footing per linear foot

Force in y-axis $F_{dy} = \gamma_E \times F_{Ey1}$ = **0.8** kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})$ = **1.4** kips

Moments on footing per linear foot

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_E \times (F_{Ey1} \times (h))$ = **3.9** kip_ft

Uplift verification

Vertical force F_{dz} = **1.399** kips

PASS - Footing is not subject to uplift

Stability against overturning in y direction, moment about y is L_y

Overturning moment $M_{OTyL} = \gamma_E \times (F_{Ey1} \times (h))$ = **1.07** kip_ft

Resisting moment $M_{RyL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2)) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times (y_1 - L_y))$ = **-2.8** kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL})$ = **2.606**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction} = max(F_{dz}, 0 kN) \times tan(\delta_{bb})$ = **0.807** kips

Stability against sliding in y direction

Total sliding resistance $F_{Ry} = F_{RFriction}$ = **0.807** kips

Factor of safety $abs(F_{RTy} / F_{dy})$ = **1**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **9.209** in

Length of bearing in y-axis $L'_{yd} = min(L_y, 3 \times (L_y / 2 - abs(e_{dy})))$ = **44.373** in

Strip base pressures

q_1 = **0** ksf

$q_2 = 4 \times F_{dz} / (3 \times 1 ft \times (L_y - 2 \times e_{dy}))$ = **0.756** ksf

Minimum base pressure $q_{min} = min(q_1, q_2)$ = **0** ksf

Maximum base pressure $q_{max} = max(q_1, q_2)$ = **0.756** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **3** ksf

q_{max} / q_{allow} = **0.252**

PASS - Allowable bearing capacity exceeds design base pressure