Michael Baker
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Creech DRP Phase 2

Aircraft Maintenance Facility (AMF)

Hangar Structural Calculations

to be updated in 100% Submission

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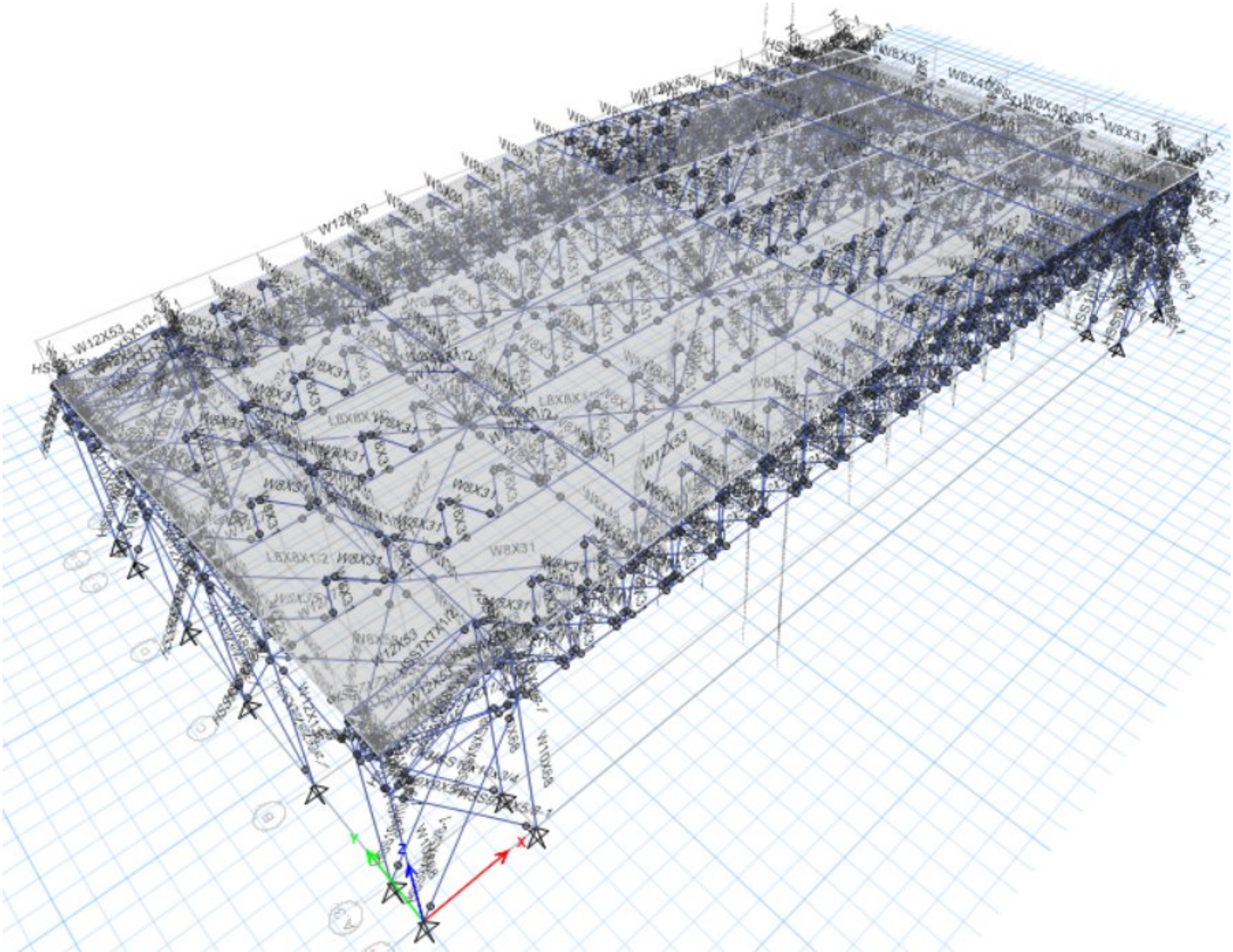
Criteria and Model Information


W12X53
W8X31
W10X68
HSS7X7X1/2
HSS5X5X1/2
L8X8X1/2
W8X31

Steel Frame Design

Steel Frame Design Preferences - AISC 360-16

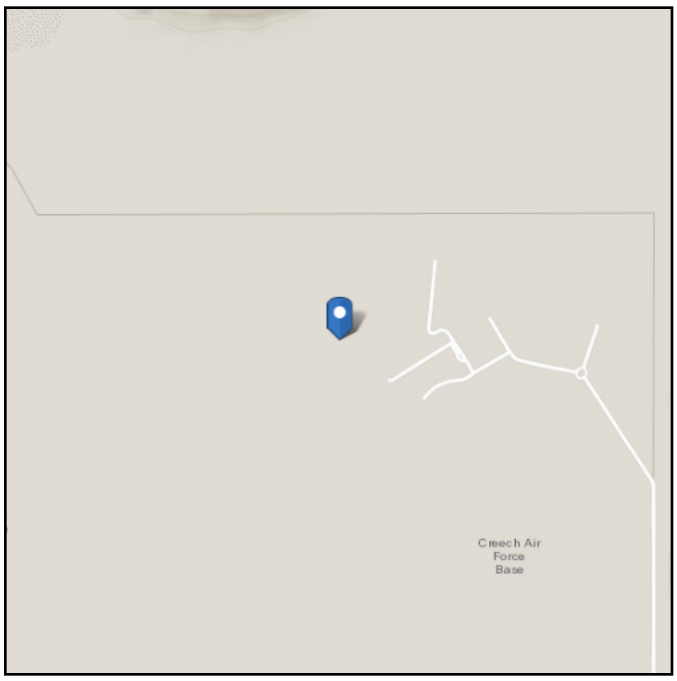
Item	Value
Multi-Response Design	Step-by-Step - All
Frame Type	SCBF
Seismic Design Category	D
Importance Factor	1.25
Design System Rho	1
Design System Sds	0.59
Design System R	6
Design System Omega0	2
Design System Cd	5
Design Provision	LRFD
Analysis Method	Direct Analysis
Second Order Method	General 2nd Order
Stiffness Reduction Method	Tau-b Fixed
Add Notional Load Case	Yes
Beta Factor	1.3
Beta Omega Factor	1.6
Phi (Bending)	0.9
Phi (Compression)	0.9
Phi (Tension-Yielding)	0.9
Phi (Tension-Fracture)	0.75
Phi (Shear)	0.9
Phi (Shear-Short Webbed Rolled I)	1
Phi (Torsion)	0.9
Ignore Seismic Code?	No
Ignore Special Seismic Load?	No
Doubler Plate Plug-Welded?	Yes
HSS Welding Type	ERW
Reduced HSS Thickness	No
Consider Deflection?	Yes
DL Ratio	120
SDL+LL Ratio	120
LL Ratio	360
Total Ratio	240
Total Camber Limit	240
Pattern Live Load Factor	0.75
D/C Ratio Limit	0.75
Maximum Iterations	1

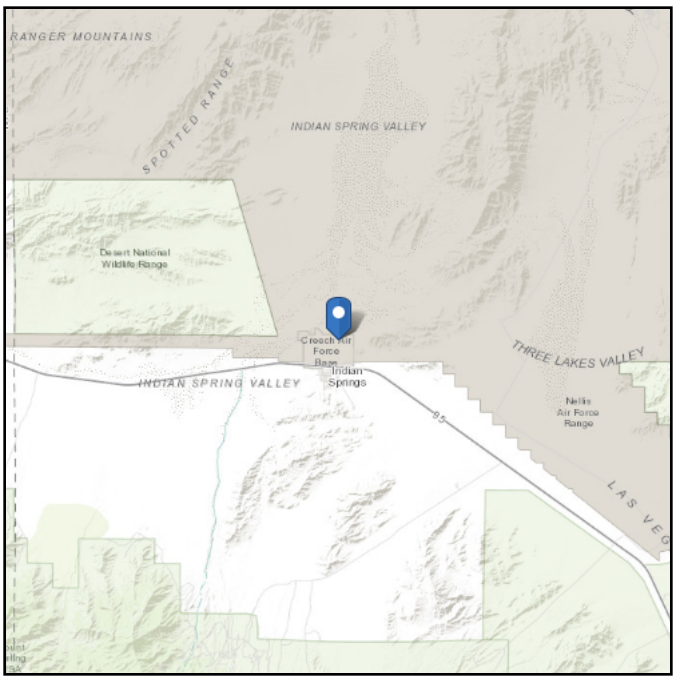
ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-22
Risk Category: III
Soil Class: D - Stiff Soil

Latitude: 36.592935
Longitude: -115.66473
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

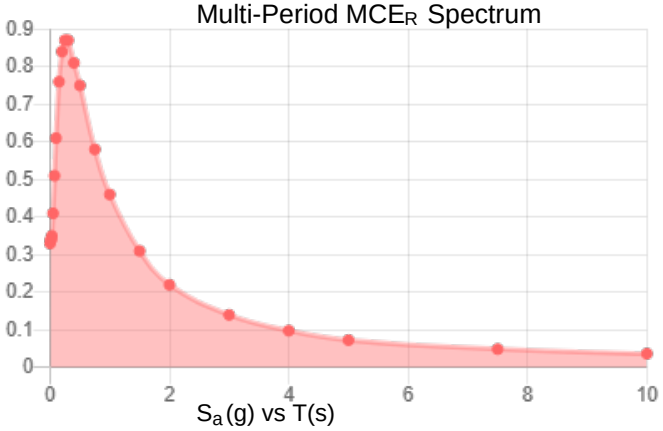
Site Soil Class: D - Stiff Soil

Results:

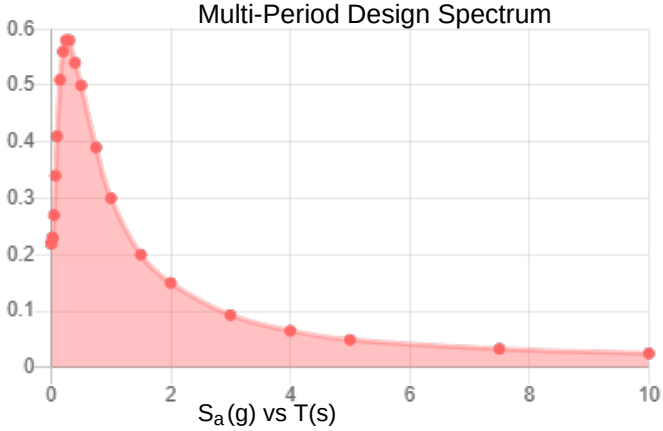
PGA_M :	0.31	T_L :	6
S_{MS} :	0.79	S_S :	0.58
S_{M1} :	0.46	S_1 :	0.17
S_{DS} :	0.52	V_{S30} :	260
S_{D1} :	0.3		

Seismic Design Category: D

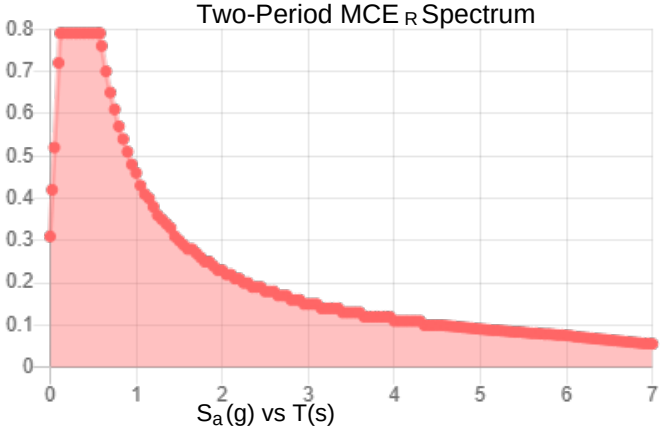
Multi-Period MCE_R Spectrum



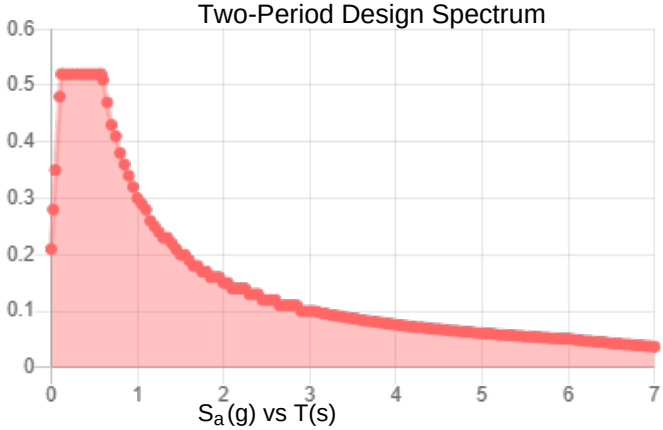
Multi-Period Design Spectrum



Two-Period MCE_R Spectrum



Two-Period Design Spectrum



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

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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)	1.00 / 12	4.8 deg
Building length	242.0 ft	
Least width	117.0 ft	
Mean Roof Ht (h)	48.0 ft	
Parapet ht above grd	48.0 ft	
Minimum parapet ht	0.0 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	N/A
Partitions	N/A
Partitions	N/A
Partitions	N/A
Partitions	N/A

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Roof Design Loads

Items	Description	Multiple	psf (max)	psf (min)
			0.0	0.0
Decking	Metal Roof deck, 1.5, 20 ga.		2.5	2.0
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.		4.0	0.0
Misc.	Misc.		0.0	0.0
			2.0	0.0
		Actual Dead Load ○	21.2 ○	9.5
		Use this DL instead ◉	25.0 ◉	9.0
		Live Load	20.0	0.0
		Snow Load	5.0	0.0
		Ultimate Wind (zone 2 - 100 sf)	19.4	-60.1
ASD Loading		D + Lr	45.0	-
		D + 0.75(0.6W + Lr)	48.7	-
		0.6*D + 0.6*W	-	-30.6
LRFD Loading		1.2D + 1.6 Lr + 0.5W	71.7	-
		1.2D + 1.0W + 0.5Lr	59.4	-
		0.9D + 1.0W	-	-52.0

Roof Live Load Reduction Roof angle 1.00 / 12 4.8 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

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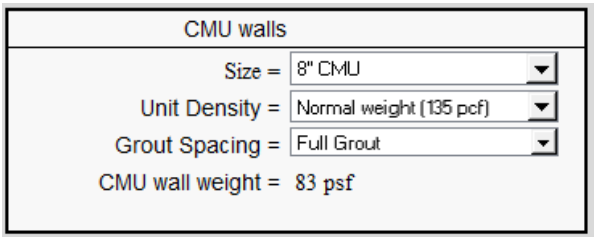
Wall Design Load #1

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"		1.5	0.7
			0.0	0.0
Misc.	Misc.		0.5	0.0
		Actual Dead Load ◉	127.8	○ 122.2
		Use this DL instead ◉	130.0	◉ 120.0



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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 =	0.890	S_{DS} =	0.593	Design Category =	D
Sm1 =	0.480	S_{D1} =	0.320	Design Category =	D

Seismic Design Category = **D**
Redundancy Coefficient ρ = 1.00 Code exception must be met for p to equal 1.0
Number of Stories: 0

Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity
Flexible Diaphragms: Yes
Building System: **Building Frame Systems**
Seismic resisting system: **Steel special concentrically braced frames**
System Structural Height Limit: **160 ft**
Actual Structural Height (hn) = 48.0 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 6
Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 5

To = 0.2(Sd1/Sds) = 0.108
Ts = Sd1/Sds = 0.539
Long Period Transition Period (TL) = 6 sec

S_{DS} = 0.593
S_{D1} = 0.320

Seismic Load Effect (E) = Eh +/-Ev = ρQ_E +/- $0.2S_{DS}$ D = Qe +/- 0.119D
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\, Q_E$ +/- $0.2S_{DS}$ D = 2Qe +/- 0.119D

Q_E = horizontal seismic force
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) = $C_T h_n^x$ = 0.365 sec x= 0.75 Tmax = CuTa = 0.511 sec
User calculated fundamental period = T = 0.365 sec

Method 2:
Seismic response coef. (Cs) = SdsI/R = 0.124
need not exceed Cs = Sd1 I /RT = 0.183
but not less than Cs = 0.044Sds*I = 0.033
USE Cs = 0.124

Design Base Shear V = 0.124W

Method 1:
Enter Sa =
Seismic response coef. (Cs) = SaI/R = 0.000
but not less than Cs = 0.044Sds*I = 0.033
Cs = Method not applicable

SEISMIC FORCES AT FLOORS - ELF Procedure

Total Stories =	1	Floor Dead Load =	0.0 psf	Roof Snow Load =	0.0 psf
Building length L =	242.0 ft	Floor LL to include =	0.0 psf	Roof Equip wt =	0.0 kips
Building width W =	117.0 ft	Floor Equip wt =	0.0 kips	Parapet weight =	0.0 psf
hn =	48.0 ft	Partition weight =	0.0 psf	Parapet height =	0.0 ft
k =	1.000	Ext Wall Weight =	29.7 psf		
V =	0.124W	Roof Dead Load =	25.0 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 = 0.148 wpx
Fpx max = $0.4S_{DS}$ Ie wpx = 0.297 wpx

Seismic Forces (Including all exterior walls)

V = 150.8k

Level (x)	EL above Seismic Base hx (ft)	Level Weight Wx (kips)	Wx hx^k (ft-kips)	Cvx = Wx hx^k / Σ Wi hi^k	Base Shear Distribution Fx=CvxV	Σ Fx (k)	Story M	Diaphragm Force Fpx Σ Wi (k)	Fpx	Design Fpx
Roof	48.00	1,220	58,543	1.000	150.76	150.8	0	1,220	150.8	180.9
1	0.00	0	0	0.000	0.00	0.0	0	0	0.0	0.0
Base		1,220		1.000		150.8	7,237			
							7,237 = Base M			

Diaphragm Forces excluding parallel exterior walls

Diaphragm Force Fpx Parallel to Bldg Length V= 108k

Cvx =	Fx=CvxV	Σ Fx (k)	Σ Wi (k)	Fpx	Design Fpx
1.000	108.12	108.1	875	108.1	129.7
0.000	0.00	0.0	0	0.0	0.0
1.000		108.1			

Diaphragm Force Fpx Normal to Bldg Length V= 130k

Level (x)	Cvx =	Fx=CvxV	Σ Fx (k)	Σ Wi (k)	Fpx	Design Fpx
Roof	1.000	130.1	130.1	1,053	130.1	156.2
1	0.000	0.0	0.0	0	0.0	0.0
Base	1.000		130.1			

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Wind Loads : ASCE 7- 22

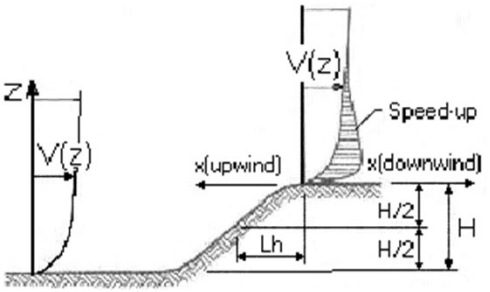
Ultimate Wind Speed	105 mph
Nominal Wind Speed	81.3 mph
Risk Category	III
Exposure Category	C
Enclosure Classif.	Partially Enclosed
Internal pressure	+/-0.55
Bldg Directionality (Kd)	0.85
Kh MWFRS<=60	1.079
Kh all other	1.079
Type of roof	Gable

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 = 0.30 K_3 = 1.000

At Mean Roof Ht:
Kzt = $(1+K_1K_2K_3)^2$ = 1.00



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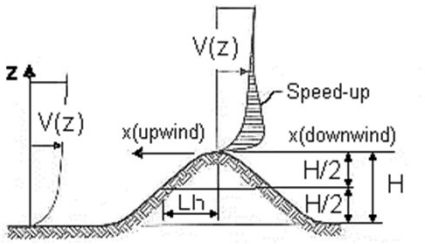

2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	48.0 ft
B =	117.0 ft
/z (0.6h) =	28.8 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 = 0.41 Rigid structure (low rise bldg)

G = 0.85 Using rigid structure default

Rigid Structure

$\bar{e}$ =	0.20	
ℓ =	500 ft	
z_{min} =	15 ft	
c =	0.20	
g_Q, g_v =	3.4	
L_z =	486.6 ft	
Q =	0.87	
I_z =	0.20	
G =	**0.86**	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 =	99.5		
N_1 =	3.42		
R_n =	0.064		
R_h =	0.446	η = 1.553	h = 48.0 ft
R_B =	0.229	η = 3.786	
R_L =	0.037	η = 26.218	
g_R =	4.104		
R =	0.600		
Gf =	0.992		

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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) =	30.5 psf	Kh =	1.079	GCpi =	+/-0.55
(Kd qh) =	**25.9 psf**	Bldg dim parallel to ridge =	242.0 ft	G =	0.85
Roof Angle (θ) =	4.8 deg	Bldg dim normal to ridge =	117.0 ft	z for qi :	48.0 ft
Roof tributary area:		h =	48.0 ft	Kd qi =	25.9 psf for positive internal pressures
Wind normal to ridge =(h/2)*L:	5808 sf	ridge ht =	53.0 ft		
Wind parallel to ridge =(h/2)*L:	2808 sf				

Ultimate Wind Surface Pressures (psf)

Surface	Wind Normal to Ridge L/B = 0.48		h/L = 0.41		Wind Parallel to Ridge L/B = 2.07		h/L = 0.20	
	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	17.6	see table below			0.80	17.6	see table below	
Leeward Wall (LW)	-0.50	-11.0	-25.2	3.2		-0.30	-6.5	-20.8	7.7
Side Wall (SW)	-0.70	-15.4	-29.6	-1.2		-0.70	-15.4	-29.6	-1.2
Leeward Roof (LR)		**				Included in windward roof			
Neg Windward Roof: 0 to h/2*	-0.90	-19.8	-34.0	-5.6	0 to h/2*	-0.90	-19.8	-34.0	-5.6
h/2 to h*	-0.90	-19.8	-34.0	-5.6	h/2 to h*	-0.90	-19.8	-34.0	-5.6
h to 2h*	-0.50	-11.0	-25.2	3.2	h to 2h*	-0.50	-11.0	-25.2	3.2
> 2h*	-0.30	-6.6	-20.8	7.6	> 2h*	-0.30	-6.6	-20.8	7.6
Pos/min windward roof press.	-0.18	-4.0	-18.2	10.3	Min press.	-0.18	-4.0	-18.2	10.3

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

Windward roof overhangs : 17.6 psf (upward : add to qhGCp windward roof pressure)

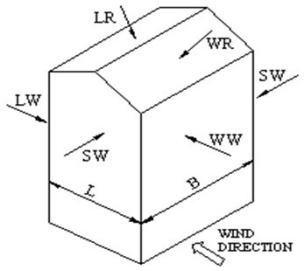
Parapet

z	Kz	Kzt	Kdqp (psf)
48.0 ft	1.079	1.00	25.9

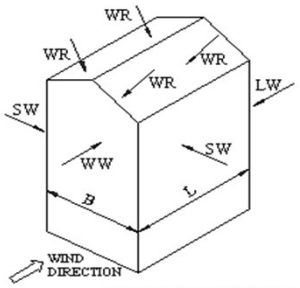
Windward parapet: 38.8 psf (GCpn = +1.5)
Leeward parapet: -25.9 psf (GCpn = -1.0)

Windward Wall Pressures at "z" (psf)

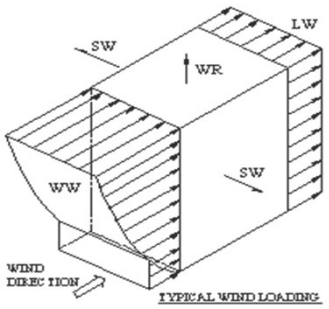
z	Kz	Kzt	Windward Wall q_zGC_p	Windward Wall w/+q_iGC_{pi}	Windward Wall w/-q_hGC_{pi}	Combined WW + LW Wind Normal to Ridge	Combined WW + LW Wind Parallel to Ridge
0 to 15'	0.85	1.00	13.9	-0.4	28.1	24.9	20.4
20.0 ft	0.90	1.00	14.7	0.5	29.0	25.7	21.3
25.0 ft	0.94	1.00	15.4	1.2	29.7	26.4	21.9
30.0 ft	0.98	1.00	16.0	1.8	30.2	27.0	22.5
40.0 ft	1.04	1.00	17.0	2.7	31.2	28.0	23.5
h= 48.0 ft	1.08	1.00	17.6	3.4	31.8	28.6	24.1
ridge = 53.0 ft	1.10	1.00	18.0	3.7	32.2	29.0	24.5



WIND NORMAL TO RIDGE



WIND PARALLEL TO RIDGE



TYPICAL WIND LOADING

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JOB TITLE			
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Elevated Buildings

Horizontal MWFRS wind pressures on objects below hb

h = 48.0 ft
hb = 0.0 ft
z = 12.0 ft

Elevated Building Geometry limitation 1

Bldg Length = 242.0 ft
Bldg Width = 117.0 ft
Area of elevated building above = 28,314 sf

Area of below elements / Area of Bldg above = 0.4%

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

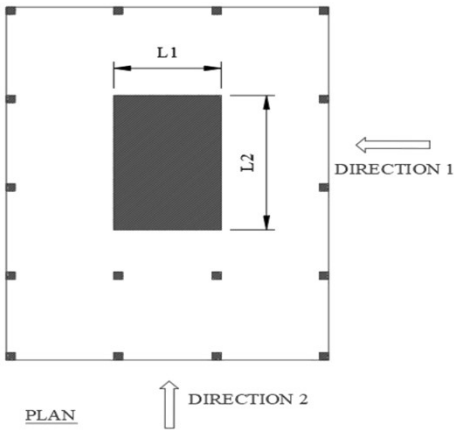
Direction 1 L/B = 0.48 Max L/B = 0.500 OK
Direction 2 L/B = 2.07 Max L/B = 0.500 OK
Meets geometry Limitation No 1 for both directions

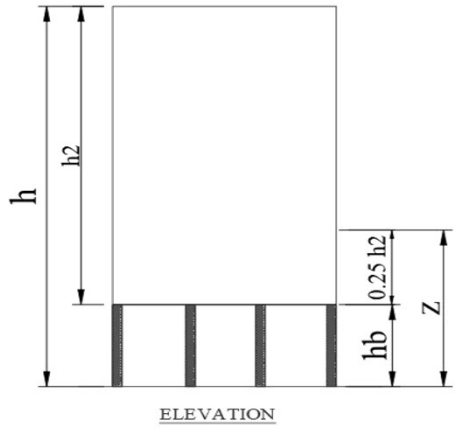
Elevated Building Geometry limitation 2

Direction 1		Direction 2	
Projected width of all columns facing direction 1 =	32.0 ft	Projected width of columns direction 2 =	30.0 ft
Projected L2 width of enclosed areas below bldg =	40.0 ft	Projected L1 width of enclosed areas =	42.0 ft
Total projected width below bldg (width) =	72.0 ft	Total projected width below bldg (width) =	72.0 ft
Projected area ratio =	29.8% OK	Projected area ratio =	61.5% OK

Meets geometry Limitation No 2 for both directions

hb = 0, therefore building is not an elevated building





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

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 = 0.48		hb/L = 0.00			L/B = 2.07		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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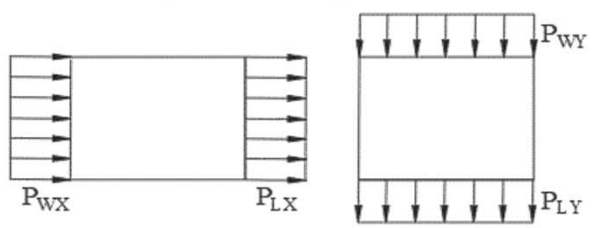
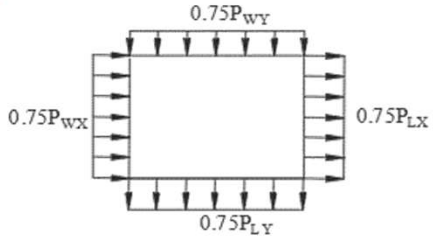
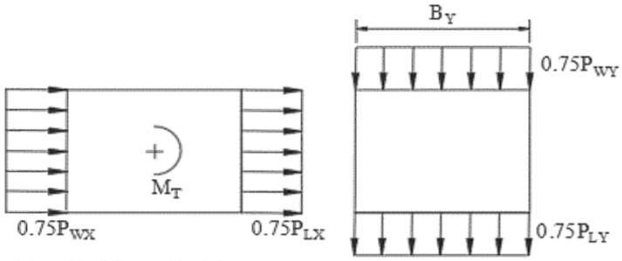
NOTE: ASCE 7 requires the application of full and partial loading of the wind pressures per the 4 cases below.



CASE 1



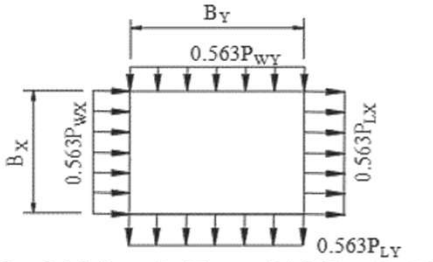
CASE 3



$M_T = 0.75(P_{WX}+P_{LX})B_X e_X$
$e_X = \pm 0.15\ B_X$

$M_T = 0.75(P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm 0.15\ B_Y$

CASE 2



$M_T = 0.563(P_{WX}+P_{LX})B_X e_X + 0.563(P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm 0.15\ B_X$ $e_Y = \pm 0.15\ B_Y$

CASE 4

Wind Forces at Floors

Building dimension (parallel with ridge) = 242.0 ft e = 36.30 ft
Total Floors above grade = 0 Building dimension (normal to ridge) = 117.0 ft e = 17.55 ft
T/Fdn (dist below grade) = 2.0 ft L is the building dimension parallel to the wind direction

	Elevation	Height of	Wind Normal to Ridge						Wind Parallel to Ridge				
Level	Above Grade (ft)	Centroid to Fdn (ft)	L	B	Area (sf)	Applied Force (k)	Story Shear (k)	Overturning Moment ('k)	Area	Applied Force (k)	Story Shear (k)	Overturning Moment ('k)	
Equip,etc	0.00	2.00	wind on equip, screenwalls, etc =			2				0			
Parapet	48.00	50.00	117.0	242.0	0.0	0.0			0.0	0.0			
T/Ridge	53.00	52.50	117.0	242.0	1,210.0	0.0			292.5	7.2			
Roof	48.00	50.00	117.0	242.0	5,808.0	166.2	168.2	-96.0	2,808.0	67.8	74.9	17.9	Roof
GRD	0.00	2.00						7,975.7				3,614.4	GRD
FDN	0.00	0.00						8,312.0				3,764.3	FDN

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Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Base pressure (qh) =	30.5 psf	Kh =	1.079	
(Kd qh) =	**25.9 psf**	h =	48.0 ft	0.2h = 9.6 ft
Minimum parapet ht =	0.0 ft	0.6h =	28.8 ft	
Roof Angle (θ) =	4.8 deg	GCpi =	+/-0.55	
Type of roof =	Gable	Kd qi =	25.9 psf	for positive internal pressures

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	-2.25	-1.84	-1.55	-1.55	-58.3	-47.6	-40.1	-40.1	-47.6	-47.6
Negative Zone 1'	-1.45	-1.45	-1.10	-0.95	-37.5	-37.5	-28.5	-24.6	-37.5	-37.5
Negative Zone 2	-2.85	-2.32	-1.95	-1.95	-73.8	-60.1	-50.5	-50.5	-60.1	-60.1
Negative Zone 3	-3.75	-2.69	-1.95	-1.95	-97.1	-69.7	-50.5	-50.5	-69.7	-69.7
Positive All Zones	0.85	0.75	0.75	0.75	22.0	19.4	19.4	19.4	19.4	19.4
Overhang Zone 1&1'	-1.70	-1.60	-1.00	-1.00	-44.0	-41.4	-25.9	-25.9	-41.4	-41.4
Overhang Zone 2	-2.30	-1.59	-1.10	-1.10	-59.5	-41.3	-28.5	-28.5	-41.3	-41.3
Overhang Zone 3	-3.20	-1.96	-1.10	-1.10	-82.8	-50.8	-28.5	-28.5	-50.8	-50.8

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 14.2 psf)

Parapet

Kd qp = 25.9 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 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Zone 3 :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CASE B:	Interior zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	Corner zone :	0.0	0.0	0.0	0.0	0.0	0.0	0.0

wall a = 11.7 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	-1.54	-1.38	-1.33	-1.27	-39.9	-35.8	-34.5	-32.9	-35.8	-34.5
Negative Zone 5	-1.81	-1.49	-1.40	-1.27	-46.9	-38.6	-36.2	-32.9	-38.6	-36.2
Positive Zone 4 & 5	1.45	1.29	1.24	1.18	37.5	33.4	32.2	30.6	33.4	32.2

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

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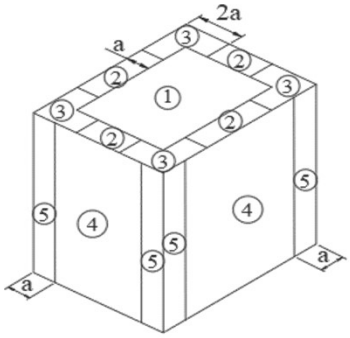
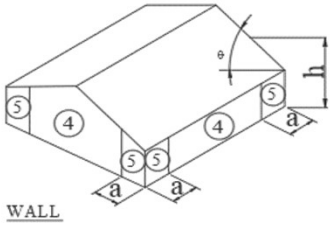
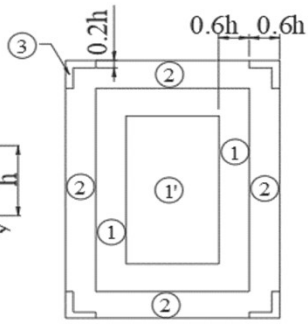
Location of C&C Wind Pressure Zones - ASCE 7-22



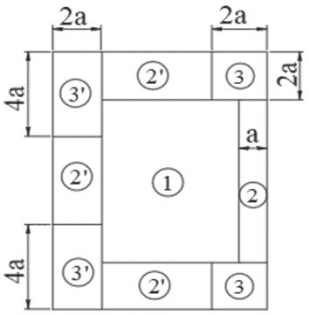
Roofs w/ θ ≤ 10°
and all walls
h > 60'



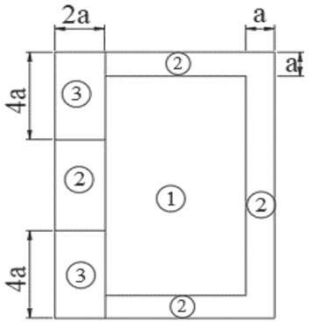
Walls h ≤ 60'
& alt design h<90'



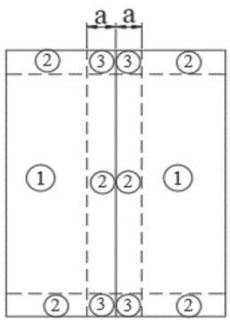
Multispan Gable & Sawtooth ≤ 10°
& Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



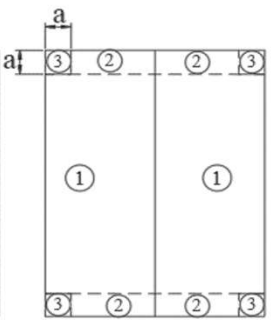
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



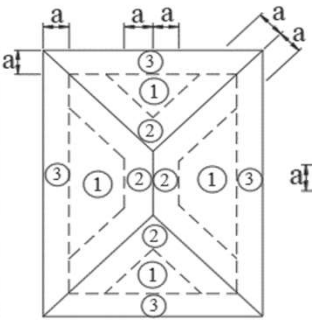
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



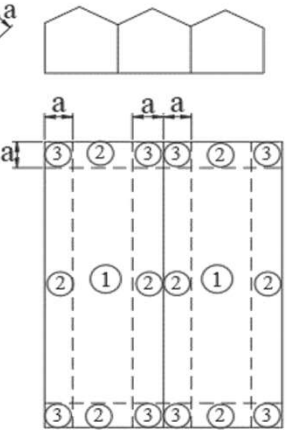
Gable 7° < θ ≤ 27°



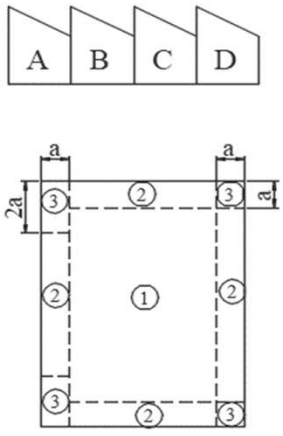
Gable 27° < θ ≤ 45°



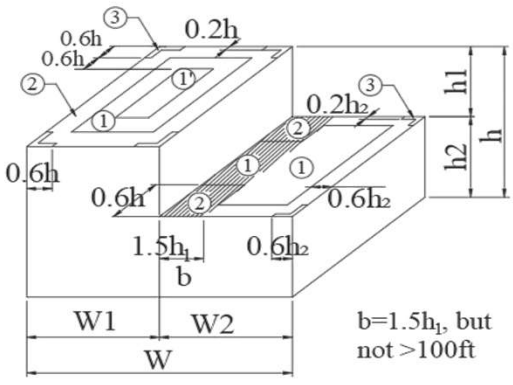
Hip 7° < θ ≤ 45°

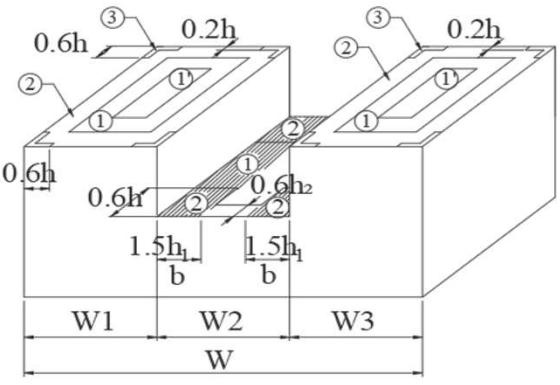


Multispan gable 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



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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JOB TITLE			
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Snow Loads : ASCE 7- 22

Ultimate Snow Forces

Roof slope	=	4.8 deg	
Horiz. eave to ridge dist (W)	=	58.5 ft	
Roof length parallel to ridge (L)	=	242.0 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**	Near ground level surface balanced snow load = **21.0 psf**
Rain on Snow Surcharge Angle		1.17 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	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
Uniform Roof Design Snow Load	=	**21.0 psf**	

Unbalanced Snow Loads - for Hip & Gable roofs only

Winter Wind Parameter W2 = 0.55

Required if slope is between 7 on 12	=	30.26 deg	
and 2.38 deg	=	2.38 deg	**Unbalanced snow loads must be applied**
Windward snow load	=	5.2 psf	= 0.3Ps
Leeward snow load from ridge to 26.12'	=	31.1 psf	= hdγ / √S + Ps
Leeward snow load from 26.12' to the eave	=	17.5 psf	= Ps

Load Patterns

Load Pattern Definitions

Name	Is Auto Load	Type	Self Weight Multiplier	Auto Load
~LLRF	Yes	Other	0	
Dead	No	Dead	1	
Exx	No	Seismic	0	ASCE 7-22
Exx(1/3)	Yes	Seismic	0	ASCE 7-22
Exx(2/3)	Yes	Seismic	0	ASCE 7-22
Exx(3/3)	Yes	Seismic	0	ASCE 7-22
Eyy	No	Seismic	0	ASCE 7-22
Eyy(1/3)	Yes	Seismic	0	ASCE 7-22
Eyy(2/3)	Yes	Seismic	0	ASCE 7-22
Eyy(3/3)	Yes	Seismic	0	ASCE 7-22
Live	No	Roof Live	0	
Mass	No	Other	0	
Sb	No	Snow	0	
Su1	No	Snow	0	
Su2	No	Snow	0	
Wx	No	Wind	0	None
Wxx	No	Wind	0	None
Wy	No	Wind	0	None
Wyy	No	Wind	0	None

Load Cases

Load Case Definitions

Name	Type
Dead	Linear Static
Live	Linear Static
Exx	Linear Static
Eyy	Linear Static
Mass	Linear Static
Wx	Linear Static
Wy	Linear Static
Wxx	Linear Static
Wyy	Linear Static
Sb	Linear Static
Su1	Linear Static
Su2	Linear Static

Load Combinations

Load Combination Definitions

Name	Type	Is Auto	Load Name	SF	Notes
DStlD1	Linear Add	Yes	Dead	1	Dead [Deflections]
DStlS1	Linear Add	Yes	Dead	1.4	Dead [Strength]
DStlS2-1	Linear Add	No	Dead	1.2	Dead + Live + Snow [Strength]
DStlS2-1			Sb	1.6	
DStlS2-2	Linear Add	No	Dead	1.2	Dead + Live + Snow [Strength]
DStlS2-2			Su1	1.6	
DStlS2-3	Linear Add	No	Dead	1.2	Dead + Live + Snow [Strength]
DStlS2-3			Su2	1.6	
DStlS3	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live [Strength]
DStlS3			Live	1.6	
DStlS4-1	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS4-1			Sb	0.5	
DStlS4-1			Wx	1	
DStlS4-2	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS4-2			Su1	0.5	
DStlS4-2			Wx	1	
DStlS4-3	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS4-3			Su2	0.5	
DStlS4-3			Wx	1	
DStlS5-1	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS5-1			Sb	0.5	
DStlS5-1			Wx	-1	
DStlS5-2	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS5-2			Su1	0.5	
DStlS5-2			Wx	-1	
DStlS5-3	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS5-3			Su2	0.5	
DStlS5-3			Wx	-1	
DStlS6-1	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS6-1			Sb	0.5	
DStlS6-1			Wy	1	
DStlS6-2	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS6-2			Su1	0.5	
DStlS6-2			Wy	1	
DStlS6-3	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS6-3			Su2	0.5	
DStlS6-3			Wy	1	
DStlS7-1	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS7-1			Sb	0.5	
DStlS7-1			Wy	-1	
DStlS7-2	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS7-2			Su1	0.5	
DStlS7-2			Wy	-1	
DStlS7-3	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS7-3			Su2	0.5	
DStlS7-3			Wy	-1	
DStlS8-1	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS8-1			Sb	0.5	
DStlS8-1			Wxx	1	
DStlS8-2	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS8-2			Su1	0.5	
DStlS8-2			Wxx	1	
DStlS8-3	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS8-3			Su2	0.5	

Name	Type	Is Auto	Load Name	SF	Notes
DStlS8-3			Wxx	1	
DStlS9-1	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS9-1			Sb	0.5	
DStlS9-1			Wxx	-1	
DStlS9-2	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS9-2			Su1	0.5	
DStlS9-2			Wxx	-1	
DStlS9-3	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS9-3			Su2	0.5	
DStlS9-3			Wxx	-1	
DStlS10-1	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS10-1			Sb	0.5	
DStlS10-1			Wyy	1	
DStlS10-2	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS10-2			Su1	0.5	
DStlS10-2			Wyy	1	
DStlS10-3	Linear Add	No	Dead	1.2	Dead + Live + Wind + Snow [Strength]
DStlS10-3			Su2	0.5	
DStlS10-3			Wyy	1	
DStlS11-1	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS11-1			Sb	0.5	
DStlS11-1			Wyy	-1	
DStlS11-2	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS11-2			Su1	0.5	
DStlS11-2			Wyy	-1	
DStlS11-3	Linear Add	No	Dead	1.2	Dead + Live - Wind + Snow [Strength]
DStlS11-3			Su2	0.5	
DStlS11-3			Wyy	-1	
DStlS12	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live + Wind [Strength]
DStlS12			Live	0.5	
DStlS12			Wx	1	
DStlS13	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live - Wind [Strength]
DStlS13			Live	0.5	
DStlS13			Wx	-1	
DStlS14	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live + Wind [Strength]
DStlS14			Live	0.5	
DStlS14			Wy	1	
DStlS15	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live - Wind [Strength]
DStlS15			Live	0.5	
DStlS15			Wy	-1	
DStlS16	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live + Wind [Strength]
DStlS16			Live	0.5	
DStlS16			Wxx	1	
DStlS17	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live - Wind [Strength]
DStlS17			Live	0.5	
DStlS17			Wxx	-1	
DStlS18	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live + Wind [Strength]
DStlS18			Live	0.5	
DStlS18			Wyy	1	
DStlS19	Linear Add	Yes	Dead	1.2	Dead + Live + Roof Live - Wind [Strength]
DStlS19			Live	0.5	
DStlS19			Wyy	-1	
DStlS20-1	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS20-1			Sb	1.6	
DStlS20-1			Wx	0.5	
DStlS20-2	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS20-2			Su1	1.6	

Name	Type	Is Auto	Load Name	SF	Notes
DStlS20-2			Wx	0.5	
DStlS20-3	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS20-3			Su2	1.6	
DStlS20-3			Wx	0.5	
DStlS21-1	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS21-1			Sb	1.6	
DStlS21-1			Wx	-0.5	
DStlS21-2	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS21-2			Su1	1.6	
DStlS21-2			Wx	-0.5	
DStlS21-3	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS21-3			Su2	1.6	
DStlS21-3			Wx	-0.5	
DStlS22-1	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS22-1			Sb	1.6	
DStlS22-1			Wy	0.5	
DStlS22-2	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS22-2			Su1	1.6	
DStlS22-2			Wy	0.5	
DStlS22-3	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS22-3			Su2	1.6	
DStlS22-3			Wy	0.5	
DStlS23-1	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS23-1			Sb	1.6	
DStlS23-1			Wy	-0.5	
DStlS23-2	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS23-2			Su1	1.6	
DStlS23-2			Wy	-0.5	
DStlS23-3	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS23-3			Su2	1.6	
DStlS23-3			Wy	-0.5	
DStlS24-1	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS24-1			Sb	1.6	
DStlS24-1			Wxx	0.5	
DStlS24-2	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS24-2			Su1	1.6	
DStlS24-2			Wxx	0.5	
DStlS24-3	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS24-3			Su2	1.6	
DStlS24-3			Wxx	0.5	
DStlS25-1	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS25-1			Sb	1.6	
DStlS25-1			Wxx	-0.5	
DStlS25-2	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS25-2			Su1	1.6	
DStlS25-2			Wxx	-0.5	
DStlS25-3	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS25-3			Su2	1.6	
DStlS25-3			Wxx	-0.5	
DStlS26-1	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS26-1			Sb	1.6	
DStlS26-1			Wyy	0.5	
DStlS26-2	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS26-2			Su1	1.6	
DStlS26-2			Wyy	0.5	
DStlS26-3	Linear Add	No	Dead	1.2	Dead + Snow + Wind [Strength]
DStlS26-3			Su2	1.6	

Name	Type	Is Auto	Load Name	SF	Notes
DStlS26-3			Wyy	0.5	
DStlS27-1	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS27-1			Sb	1.6	
DStlS27-1			Wyy	-0.5	
DStlS27-2	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS27-2			Su1	1.6	
DStlS27-2			Wyy	-0.5	
DStlS27-3	Linear Add	No	Dead	1.2	Dead + Snow - Wind [Strength]
DStlS27-3			Su2	1.6	
DStlS27-3			Wyy	-0.5	
DStlS28	Linear Add	Yes	Dead	1.2	Dead + Roof Live + Wind [Strength]
DStlS28			Live	1.6	
DStlS28			Wx	0.5	
DStlS29	Linear Add	Yes	Dead	1.2	Dead + Roof Live - Wind [Strength]
DStlS29			Live	1.6	
DStlS29			Wx	-0.5	
DStlS30	Linear Add	Yes	Dead	1.2	Dead + Roof Live + Wind [Strength]
DStlS30			Live	1.6	
DStlS30			Wy	0.5	
DStlS31	Linear Add	Yes	Dead	1.2	Dead + Roof Live - Wind [Strength]
DStlS31			Live	1.6	
DStlS31			Wy	-0.5	
DStlS32	Linear Add	Yes	Dead	1.2	Dead + Roof Live + Wind [Strength]
DStlS32			Live	1.6	
DStlS32			Wxx	0.5	
DStlS33	Linear Add	Yes	Dead	1.2	Dead + Roof Live - Wind [Strength]
DStlS33			Live	1.6	
DStlS33			Wxx	-0.5	
DStlS34	Linear Add	Yes	Dead	1.2	Dead + Roof Live + Wind [Strength]
DStlS34			Live	1.6	
DStlS34			Wyy	0.5	
DStlS35	Linear Add	Yes	Dead	1.2	Dead + Roof Live - Wind [Strength]
DStlS35			Live	1.6	
DStlS35			Wyy	-0.5	
DStlS36	Linear Add	Yes	Dead	0.9	Dead (min) + Wind [Strength]
DStlS36			Wx	1	
DStlS37	Linear Add	Yes	Dead	0.9	Dead (min) - Wind [Strength]
DStlS37			Wx	-1	
DStlS38	Linear Add	Yes	Dead	0.9	Dead (min) + Wind [Strength]
DStlS38			Wy	1	
DStlS39	Linear Add	Yes	Dead	0.9	Dead (min) - Wind [Strength]
DStlS39			Wy	-1	
DStlS40	Linear Add	Yes	Dead	0.9	Dead (min) + Wind [Strength]
DStlS40			Wxx	1	
DStlS41	Linear Add	Yes	Dead	0.9	Dead (min) - Wind [Strength]
DStlS41			Wxx	-1	
DStlS42	Linear Add	Yes	Dead	0.9	Dead (min) + Wind [Strength]
DStlS42			Wyy	1	
DStlS43	Linear Add	Yes	Dead	0.9	Dead (min) - Wind [Strength]
DStlS43			Wyy	-1	
DStlS44-1	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS44-1			Sb	0.2	
DStlS44-1			Exx	1	
DStlS44-1O	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS44-1O			Sb	0.2	
DStlS44-1O			Exx	2	
DStlS44-2	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]

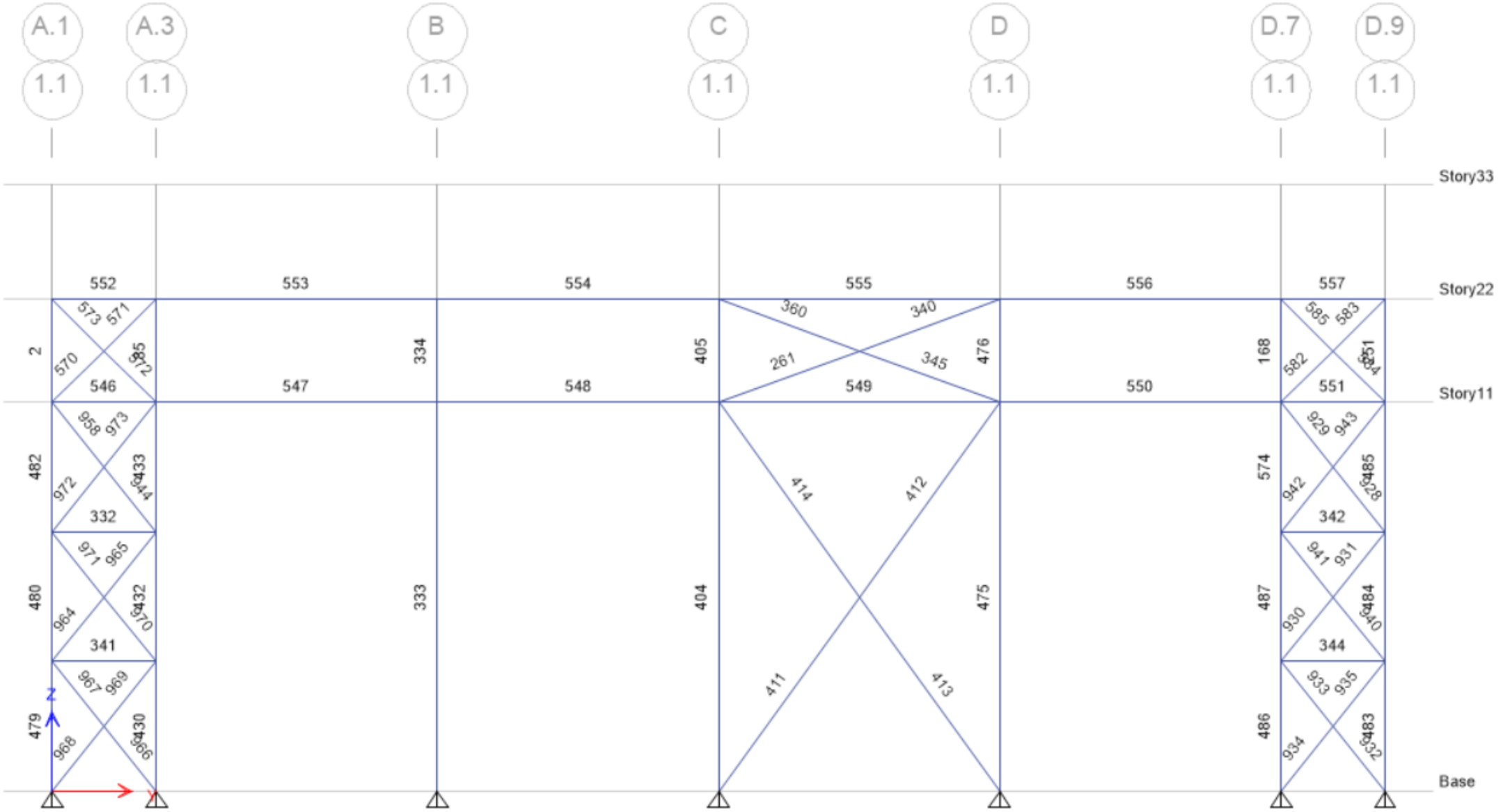
Name	Type	Is Auto	Load Name	SF	Notes
DStlS44-2			Su1	0.2	
DStlS44-2			Exx	1	
DStlS44-2O	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS44-2O			Su1	0.2	
DStlS44-2O			Exx	2	
DStlS44-3	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS44-3			Su2	0.2	
DStlS44-3			Exx	1	
DStlS44-3O	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS44-3O			Su2	0.2	
DStlS44-3O			Exx	2	
DStlS45-1	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS45-1			Sb	0.2	
DStlS45-1			Exx	-1	
DStlS45-1O	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS45-1O			Sb	0.2	
DStlS45-1O			Exx	-2	
DStlS45-2	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS45-2			Su1	0.2	
DStlS45-2			Exx	-1	
DStlS45-2O	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS45-2O			Su1	0.2	
DStlS45-2O			Exx	-2	
DStlS45-3	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS45-3			Su2	0.2	
DStlS45-3			Exx	-1	
DStlS45-3O	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS45-3O			Su2	0.2	
DStlS45-3O			Exx	-2	
DStlS46-1	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS46-1			Sb	0.2	
DStlS46-1			Eyy	1	
DStlS46-1O	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS46-1O			Sb	0.2	
DStlS46-1O			Eyy	2	
DStlS46-2	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS46-2			Su1	0.2	
DStlS46-2			Eyy	1	
DStlS46-2O	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS46-2O			Su1	0.2	
DStlS46-2O			Eyy	2	
DStlS46-3	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS46-3			Su2	0.2	
DStlS46-3			Eyy	1	
DStlS46-3O	Linear Add	No	Dead	1.318	Dead + Live + Static Earthquake [Strength]
DStlS46-3O			Su2	0.2	
DStlS46-3O			Eyy	2	
DStlS47-1	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS47-1			Sb	0.2	
DStlS47-1			Eyy	-1	
DStlS47-1O	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS47-1O			Sb	0.2	
DStlS47-1O			Eyy	-2	
DStlS47-2	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS47-2			Su1	0.2	
DStlS47-2			Eyy	-1	
DStlS47-2O	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]

Name	Type	Is Auto	Load Name	SF	Notes
DStlS47-2O			Su1	0.2	
DStlS47-2O			Eyy	-2	
DStlS47-3	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS47-3			Su2	0.2	
DStlS47-3			Eyy	-1	
DStlS47-3O	Linear Add	No	Dead	1.318	Dead + Live - Static Earthquake [Strength]
DStlS47-3O			Su2	0.2	
DStlS47-3O			Eyy	-2	
DStlS48	Linear Add	Yes	Dead	0.782	Dead (min) + Static Earthquake [Strength]
DStlS48			Exx	1	
DStlS48-O	Linear Add	No	Dead	0.782	Dead (min) + Static Earthquake [Strength]
DStlS48-O			Exx	2	
DStlS49	Linear Add	Yes	Dead	0.782	Dead (min) - Static Earthquake [Strength]
DStlS49			Exx	-1	
DStlS49-O	Linear Add	No	Dead	0.782	Dead (min) - Static Earthquake [Strength]
DStlS49-O			Exx	-2	
DStlS50	Linear Add	Yes	Dead	0.782	Dead (min) + Static Earthquake [Strength]
DStlS50			Eyy	1	
DStlS50-O	Linear Add	No	Dead	0.782	Dead (min) + Static Earthquake [Strength]
DStlS50-O			Eyy	2	
DStlS51	Linear Add	Yes	Dead	0.782	Dead (min) - Static Earthquake [Strength]
DStlS51			Eyy	-1	
DStlS51-O	Linear Add	No	Dead	0.782	Dead (min) - Static Earthquake [Strength]
DStlS51-O			Eyy	-2	
ENV	Envelope	No	DStlS1	1	
ENV			DStlS2-1	1	
ENV			DStlS2-2	1	
ENV			DStlS2-3	1	
ENV			DStlS3	1	
ENV			DStlS4-1	1	
ENV			DStlS4-2	1	
ENV			DStlS4-3	1	
ENV			DStlS5-1	1	
ENV			DStlS5-2	1	
ENV			DStlS5-3	1	
ENV			DStlS6-1	1	
ENV			DStlS6-2	1	
ENV			DStlS6-3	1	
ENV			DStlS7-1	1	
ENV			DStlS7-2	1	
ENV			DStlS7-3	1	
ENV			DStlS8-1	1	
ENV			DStlS8-2	1	
ENV			DStlS8-3	1	
ENV			DStlS9-1	1	
ENV			DStlS9-2	1	
ENV			DStlS9-3	1	
ENV			DStlS10-1	1	
ENV			DStlS10-2	1	
ENV			DStlS10-3	1	
ENV			DStlS11-1	1	
ENV			DStlS11-2	1	
ENV			DStlS11-3	1	
ENV			DStlS12	1	
ENV			DStlS13	1	
ENV			DStlS14	1	
ENV			DStlS15	1	

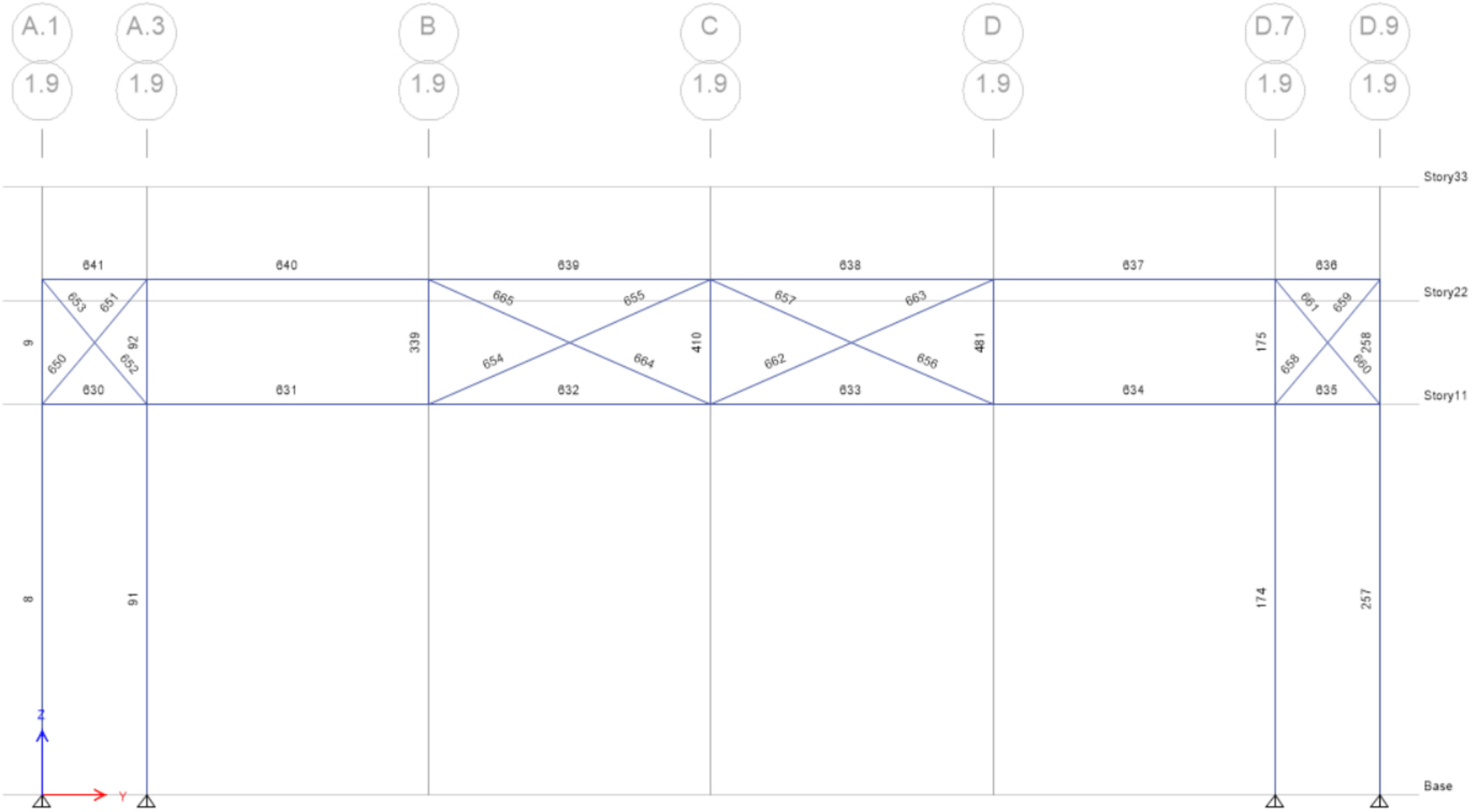
Name	Type	Is Auto	Load Name	SF	Notes
ENV			DStlS16	1	
ENV			DStlS17	1	
ENV			DStlS18	1	
ENV			DStlS19	1	
ENV			DStlS20-1	1	
ENV			DStlS20-2	1	
ENV			DStlS20-3	1	
ENV			DStlS21-1	1	
ENV			DStlS21-2	1	
ENV			DStlS21-3	1	
ENV			DStlS22-1	1	
ENV			DStlS22-2	1	
ENV			DStlS22-3	1	
ENV			DStlS23-1	1	
ENV			DStlS23-2	1	
ENV			DStlS23-3	1	
ENV			DStlS24-1	1	
ENV			DStlS24-2	1	
ENV			DStlS24-3	1	
ENV			DStlS25-1	1	
ENV			DStlS25-2	1	
ENV			DStlS25-3	1	
ENV			DStlS26-1	1	
ENV			DStlS26-2	1	
ENV			DStlS26-3	1	
ENV			DStlS27-1	1	
ENV			DStlS27-2	1	
ENV			DStlS27-3	1	
ENV			DStlS28	1	
ENV			DStlS29	1	
ENV			DStlS30	1	
ENV			DStlS31	1	
ENV			DStlS32	1	
ENV			DStlS33	1	
ENV			DStlS34	1	
ENV			DStlS35	1	
ENV			DStlS36	1	
ENV			DStlS37	1	
ENV			DStlS38	1	
ENV			DStlS39	1	
ENV			DStlS40	1	
ENV			DStlS41	1	
ENV			DStlS42	1	
ENV			DStlS43	1	
ENV			DStlS44-1O	1	
ENV			DStlS44-2O	1	
ENV			DStlS44-3O	1	
ENV			DStlS45-1O	1	
ENV			DStlS45-2O	1	
ENV			DStlS45-3O	1	
ENV			DStlS46-1O	1	
ENV			DStlS46-2O	1	
ENV			DStlS46-3O	1	
ENV			DStlS47-1O	1	
ENV			DStlS47-2O	1	
ENV			DStlS47-3O	1	
ENV			DStlS48-O	1	

Name	Type	Is Auto	Load Name	SF	Notes
ENV			DStlS49-O	1	
ENV			DStlS50-O	1	
ENV			DStlS51-O	1	

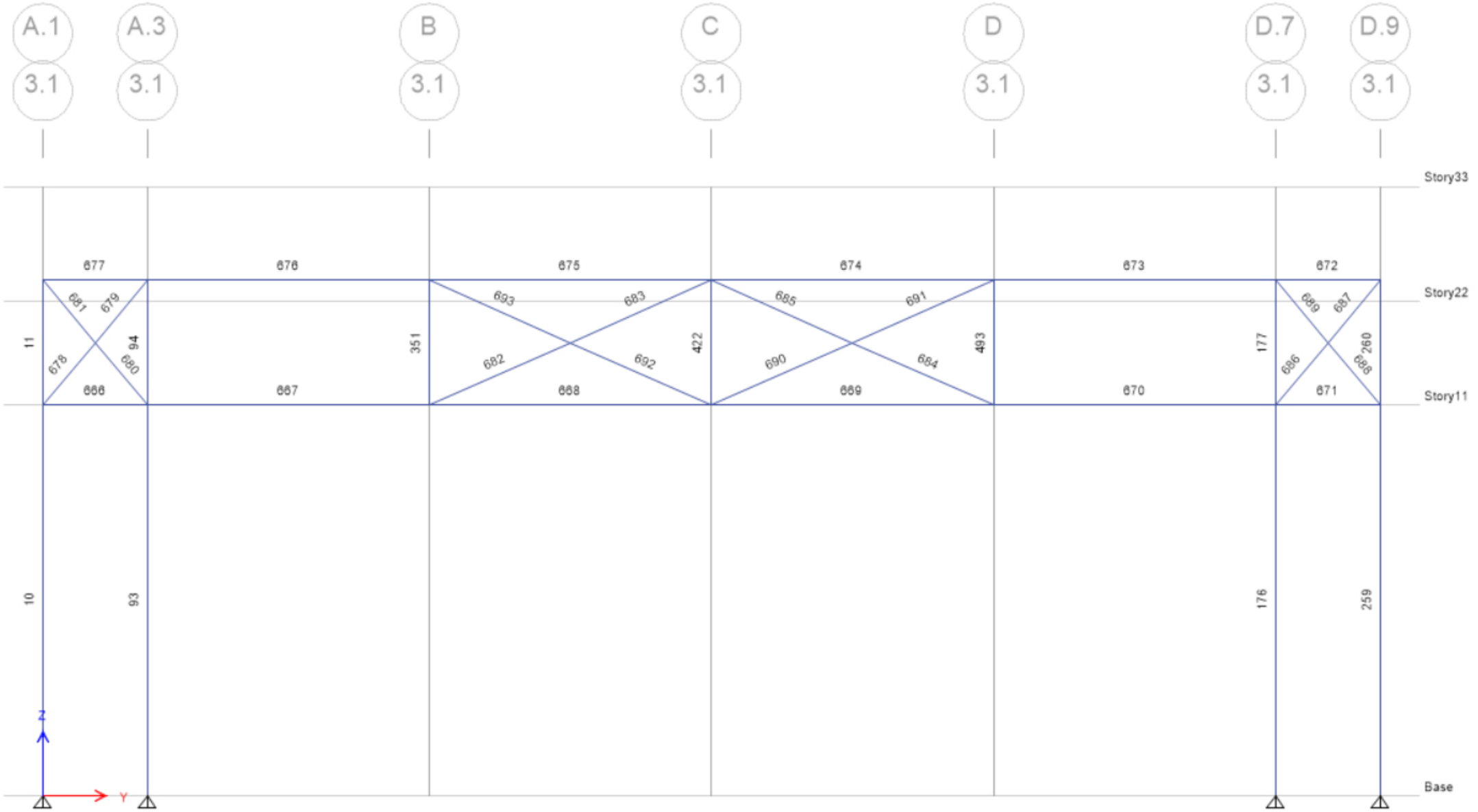
Hangar Framing - Elevation 1.1



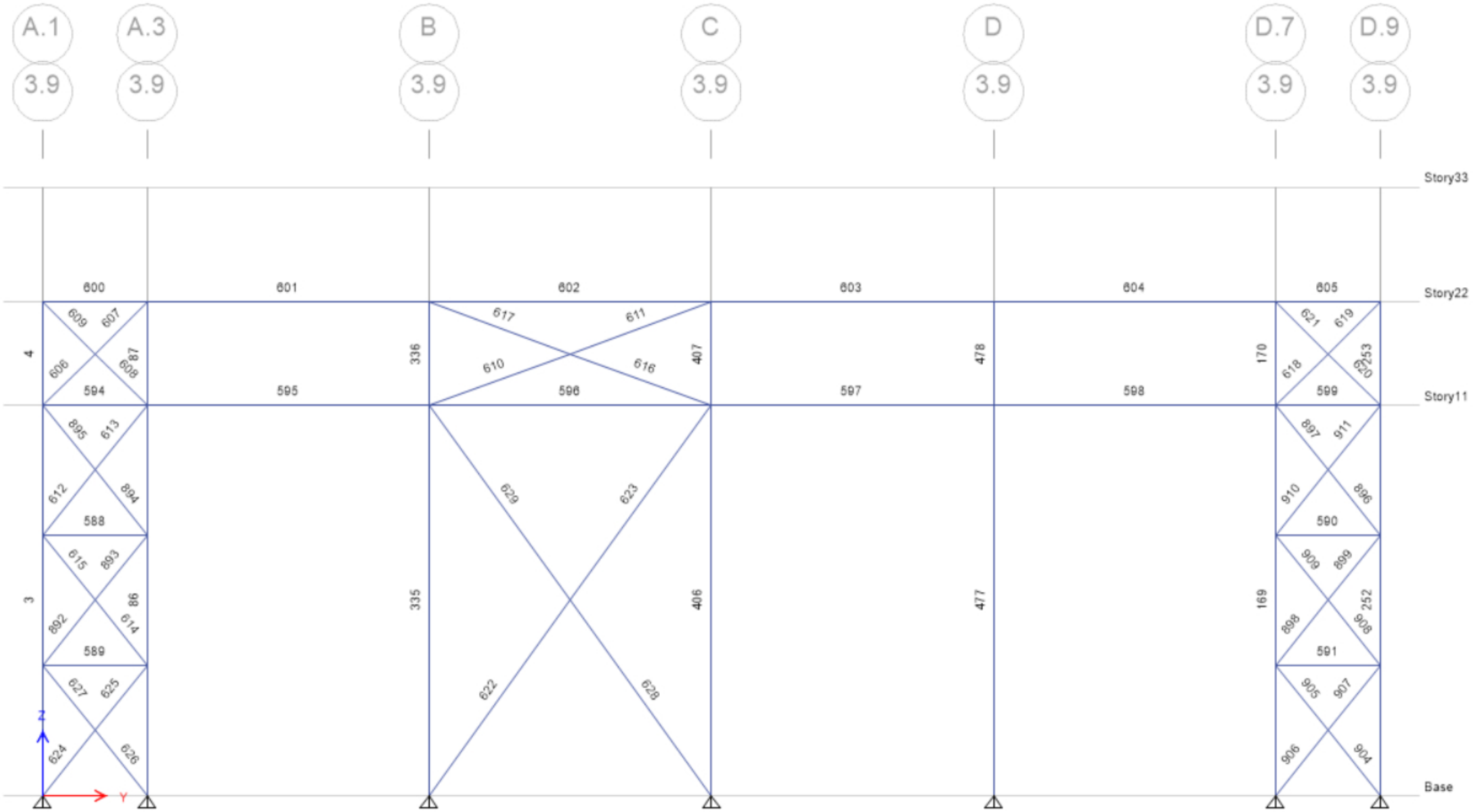
Hangar Framing - Elevation 1.9



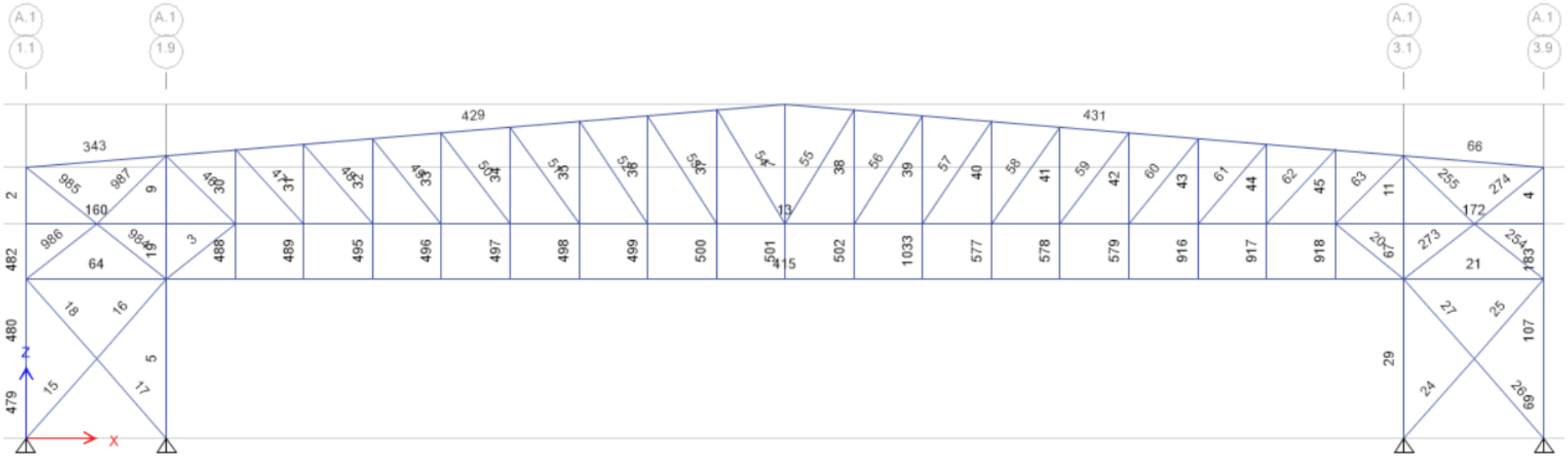
Hangar Framing - Elevation 3.1



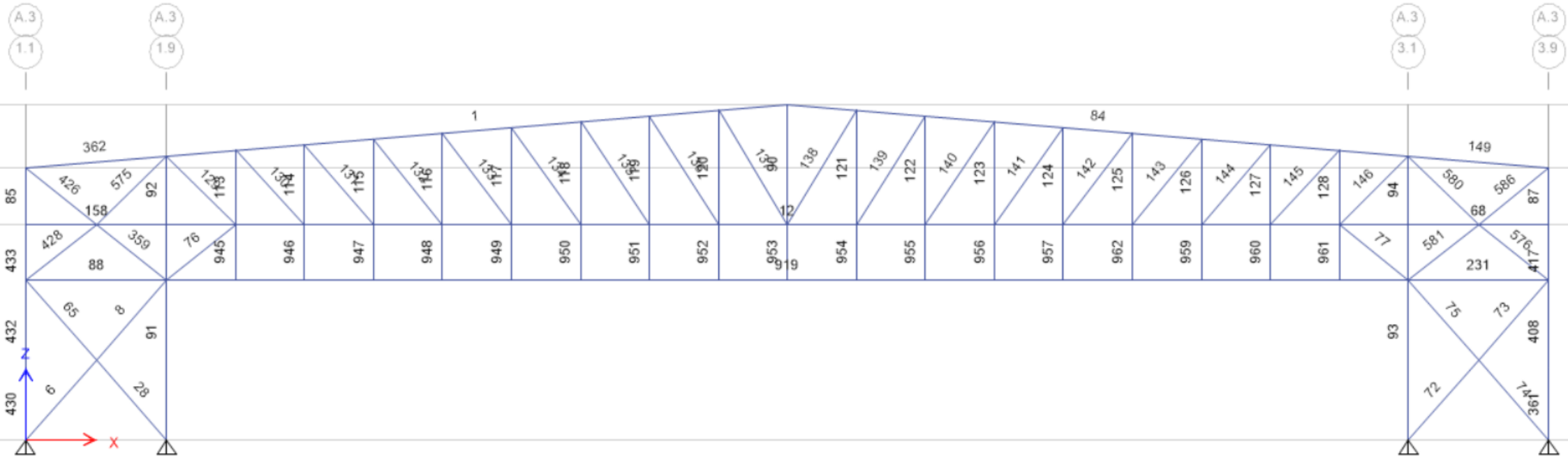
Hangar Framing - Elevation 3.9



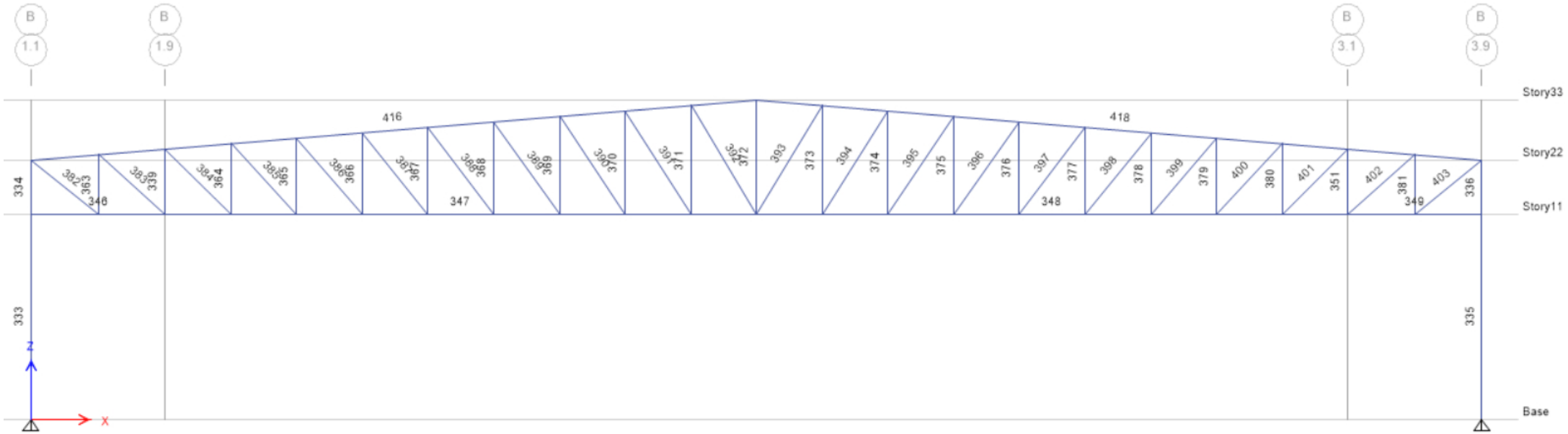
Hangar Framing - Elevation A.1



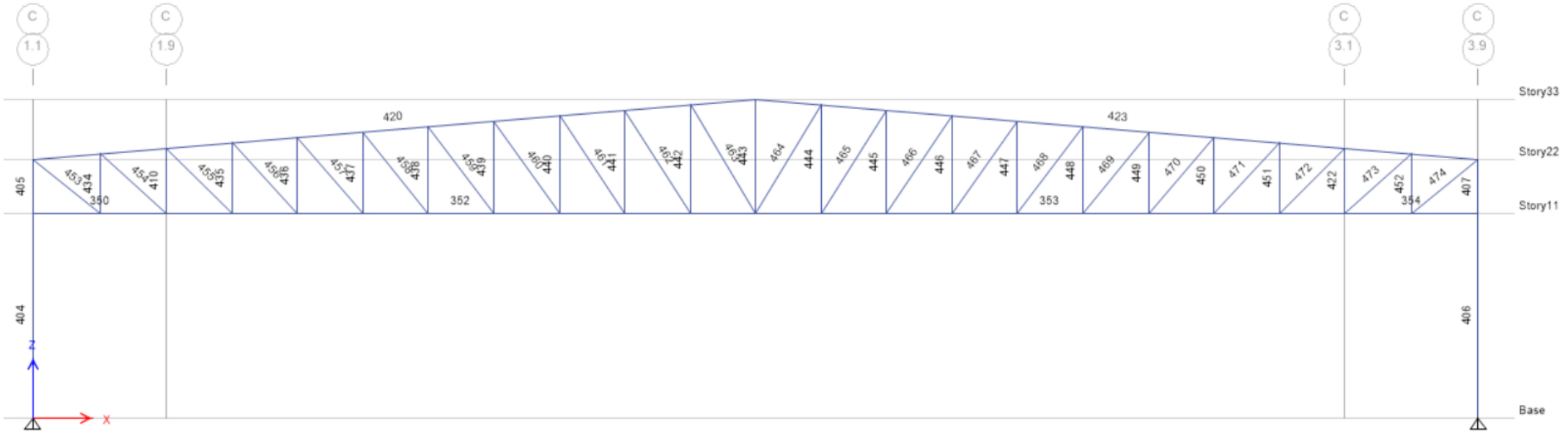
Hangar Framing - Elevation A.3



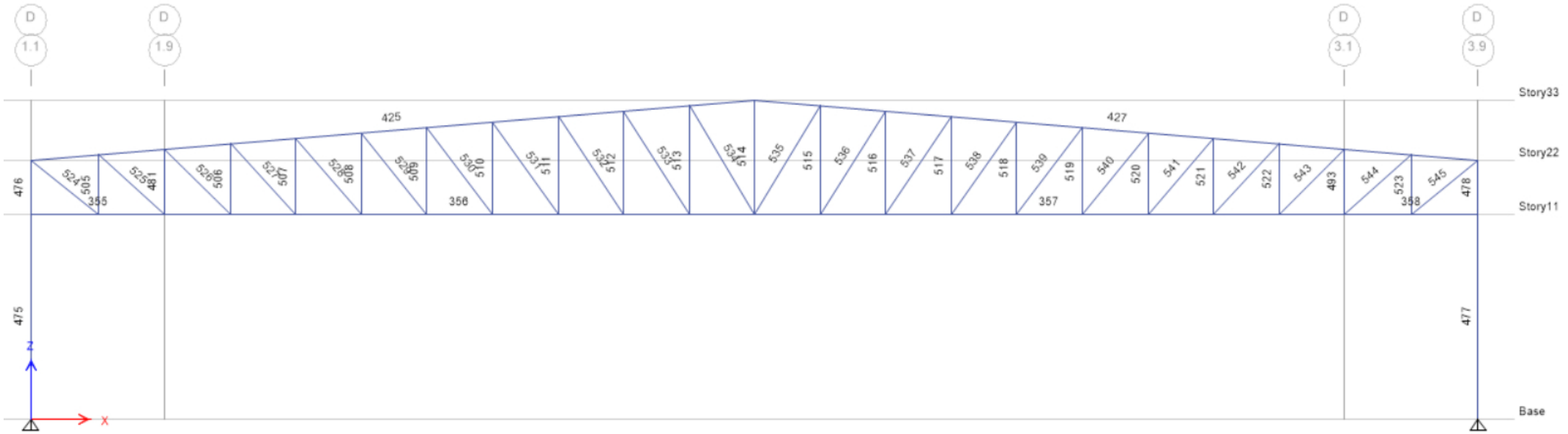
Hangar Framing - Elevation B



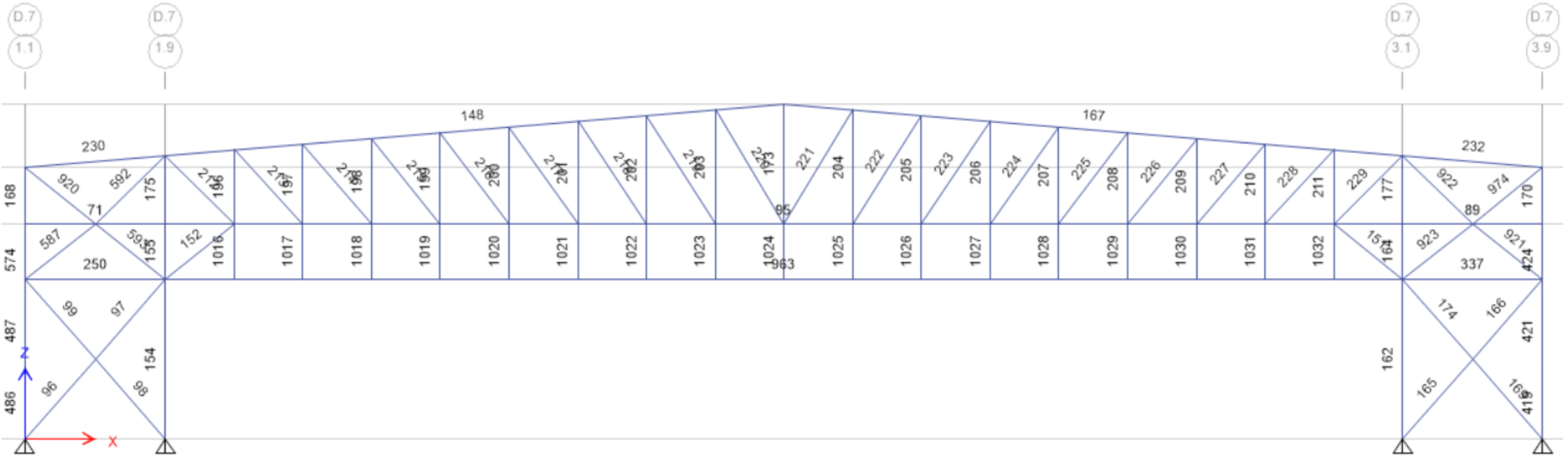
Hangar Framing - Elevation C



Hangar Framing - Elevation D



Hangar Framing - Elevation D.7



Hangar Framing - Elevation D.9

Michael Baker
INTERNATIONAL

Gravity Framing and Column Calculations

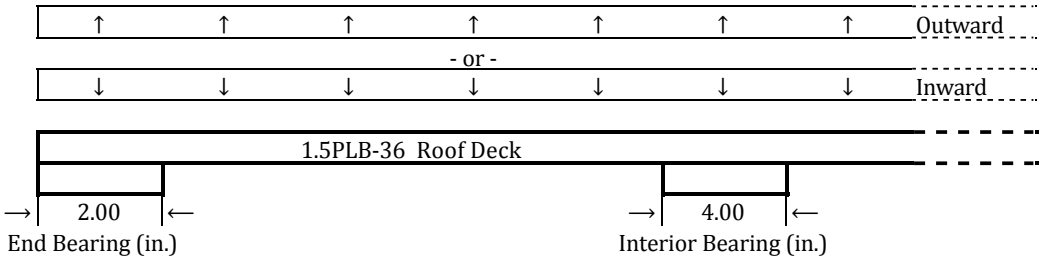
20 Gage 1.5PLB-36 Grade 50

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)	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"
1	ΦWn	269	222	187	159	137	119	105	93
	L/240	103	78	60	47	38	31	25	21
2	ΦWn	271	224	189	161	139	121	107	95
	L/240	274	206	159	125	100	81	67	56
3	ΦWn	336	279	235	201	173	151	133	118
	L/240	215	161	124	98	78	64	52	44

> 1.2D + 0.5S =1.2*25 + 0.5*32 = 46 psf

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

Span	(ft-in)	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"
1	ΦWn	275	227	191	163	140	122	107	95
	ΦRn	386	351	322	297	276	257	241	227
	L/240	114	86	66	52	41	34	28	23
2	ΦWn	176	160	147	136	126	118	104	93
	ΦRn	176	160	147	136	126	118	110	104
	L/240	249	187	144	113	91	74	61	51
3	ΦWn	201	182	167	154	143	134	125	115
	ΦRn	201	182	167	154	143	134	125	118
	L/240	195	147	113	89	71	58	48	40

> 0.9D + W = -0.9*9.5 + 67 = 58 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: $W \le \Phi Wn$

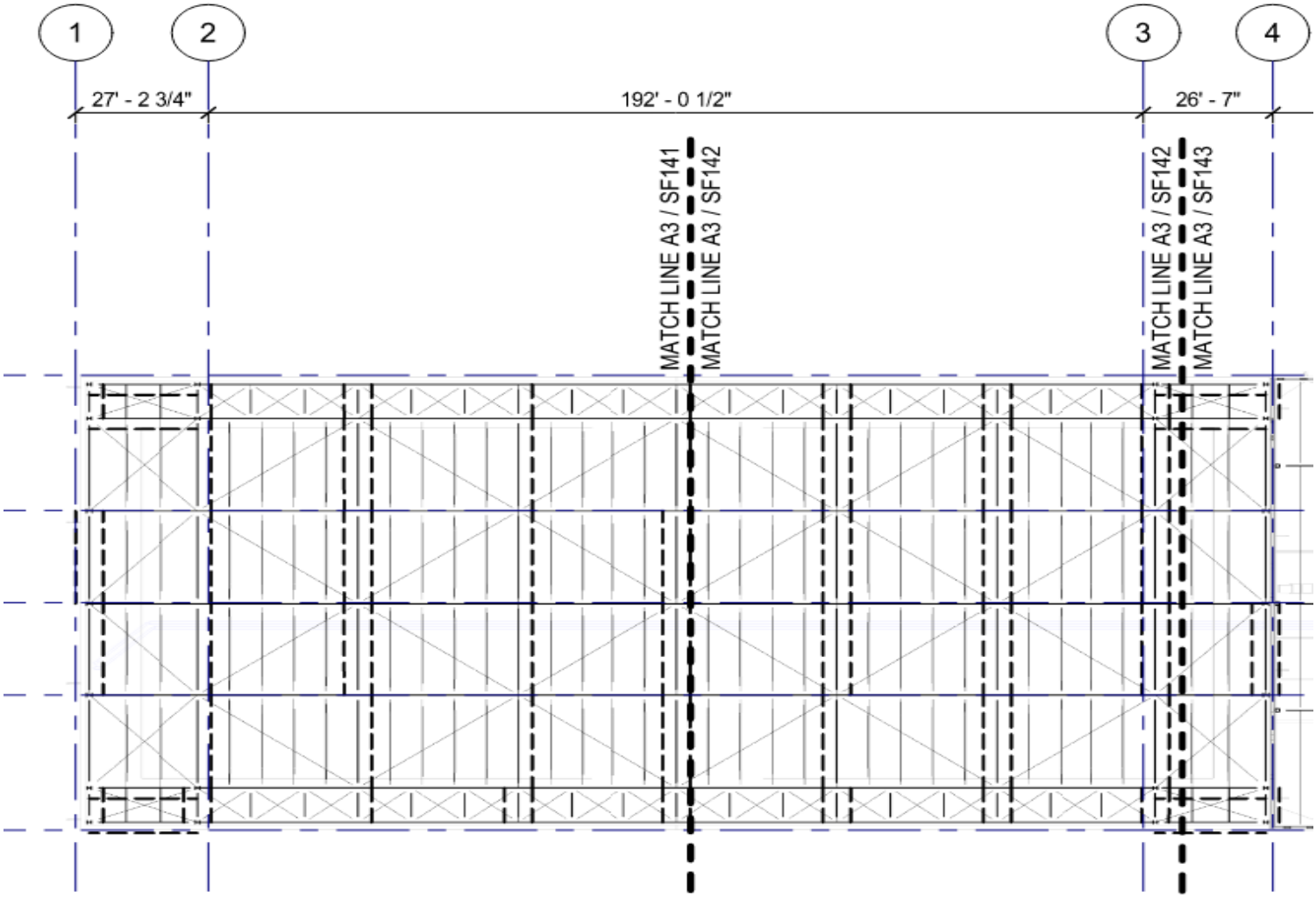
W = Required strength of the governing LRFD load combintaion

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: 10/7/2025

NOTICE: Design defects that could cause injury or death may result from relying on the information in this document without independent verification by a qualified professional. The information in this document is provided "AS IS". Nucor Corporation and its affiliates expressly disclaim: (i) any and all representations, warranties and conditions and (ii) all liability arising out of or related to this document and the information in it.


1
2
3
4
27' - 2 3/4"
192' - 0 1/2"
26' - 7"
MATCH LINE A3 / SF141
MATCH LINE A3 / SF142
MATCH LINE A3 / SF142
MATCH LINE A3 / SF143

PROJECT INFO							
Project Number	**Project Name**	**Design Mark**	**Engineer**	**Design Method**	**Load Type**	**Design Code**	**Date**
-	-	-	-	ASD	Not Reduced	ASCE 7-22	10/08/2025

SPECIFICATIONS							
Designation*	**Span (ft)**	**Tributary Width (ft)**	**TCXL (ft)**	**TCXR (ft)**	**Slope (in/ft)**	**LL Deflection Limit (L/X)**	**Min I (in^4)**
24K345/158	25	7.5	0	0	0	240	0

UNIFORM LOADS						
All uniform loads are included in designation loading for non-SP joists.						
Dead (psf)	**Dead Collateral (psf)**	**Live (psf)**	**Live Roof (psf)**	**Rain (psf)**	**Snow [1.0S] (psf)**	**Downward Wind [1.0W] (psf)**
25	0	0	20	0	21	0

UPLIFT ZONES			
Type	**Magnitude (psf)**	**Start Location (ft)****	**Length (ft)**
Gross (1.0W)	16	0	25

ESTIMATED PROPERTIES						
All provided properties are accurate for comparison. Contact Vulcraft if exact values are required.						
Self Weight (psf)***	**I (in^4)********	**LL Deflection (L/X)*******	**TC Width (in)**	**Bridging (T/X/EX/B)**	**Seat Depth (in)**	**Max Duct Sizes (Rd | Sq | H x W)********
1.5	173	991 [0.3]	4.00	1/0/0/2	2.50	11" | 9" | 8" x 9"

**Designation loads are unfactored*

***Negative value indicates TCXL location*

****Ensure joist self weight has been included in specified dead load*

*****Minimum moment of inertia should be provided to Vulcraft if required for structural performance*

******Calculated deflections have been increased by 15% to account for shear deformation*

*******Actual duct sizes and locations should be provided to Vulcraft to ensure clearance during final design*

RECORD KEY
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Created using the Vulcraft Joist Wizard (2024.10.18.6)

NUCOR® VULCRAFT

Hangar – All beam sections with DC ratios (Gravity Design)

Label	Section	D/C Ratio
1	W12X53	0.309 = 0.001 + 0.238 + 0.07
6	W12X45	0.396 = 0.381 + 0.015 + 0
12	W12X53	0.319 = 0.316 + 0.003 + 0
13	W12X53	0.321 = 0.319 + 0.003 + 0
14	W12X53	0.082 = 0.079 + 0.003 + 0
15	W12X53	0.012 = 0.009 + 0.003 + 0
16	W12X53	0.043 = 0.04 + 0.003 + 0
17	W12X53	0.083 = 0.079 + 0.003 + 0
18	W12X53	0.231 = 0.228 + 0.003 + 0
19	W12X53	0.267 = 0.264 + 0.003 + 0
20	W12X53	0.272 = 0.269 + 0.003 + 0
21	W12X53	0.27 = 0.267 + 0.003 + 0
22	W12X53	0.27 = 0.267 + 0.003 + 0
23	W12X53	0.272 = 0.269 + 0.003 + 0
24	W12X53	0.267 = 0.264 + 0.003 + 0
25	W12X53	0.23 = 0.228 + 0.003 + 0
26	W12X53	0.082 = 0.079 + 0.003 + 0
27	W12X53	0.043 = 0.04 + 0.003 + 0
28	W12X53	0.013 = 0.009 + 0.003 + 0
29	W12X53	0.082 = 0.079 + 0.003 + 0
66	W12X53	0.12 = 0.024 + 0.091 + 0.006
84	W12X53	0.309 = 2.377E-04 + 0.237 + 0.072
89	W12X45	0.094 = 0.077 + 0.017 + 0
95	W12X53	0.321 = 0.318 + 0.003 + 0
96	W12X53	0.323 = 0.32 + 0.003 + 0
97	W12X53	0.083 = 0.08 + 0.003 + 0
98	W12X53	0.033 = 0.03 + 0.003 + 0
99	W12X53	0.094 = 0.091 + 0.003 + 0
100	W12X53	0.229 = 0.226 + 0.003 + 0
101	W12X53	0.248 = 0.245 + 0.003 + 0
102	W12X53	0.271 = 0.268 + 0.003 + 0
103	W12X53	0.301 = 0.298 + 0.003 + 0
104	W12X53	0.315 = 0.312 + 0.003 + 0
105	W12X53	0.315 = 0.312 + 0.003 + 0
106	W12X53	0.301 = 0.298 + 0.003 + 0
107	W12X53	0.271 = 0.268 + 0.003 + 0
108	W12X53	0.248 = 0.245 + 0.003 + 0
109	W12X53	0.229 = 0.226 + 0.003 + 0
110	W12X53	0.094 = 0.091 + 0.003 + 0

111	W12X53	0.034 = 0.031 + 0.003 + 0
112	W12X53	0.084 = 0.081 + 0.003 + 0
148	W12X53	0.31 = 0.001 + 0.238 + 0.07
149	W12X53	0.275 = 0.028 + 0.242 + 0.004
167	W12X53	0.307 = 0.001 + 0.236 + 0.07
171	W12X45	0.09 = 0.073 + 0.017 + 0
172	W12X45	0.088 = 0.071 + 0.017 + 0
178	W12X45	0.417 = 0.402 + 0.015 + 0
179	W12X53	0.32 = 0.317 + 0.003 + 0
180	W12X53	0.085 = 0.081 + 0.003 + 0
181	W12X53	0.035 = 0.032 + 0.003 + 0
182	W12X53	0.094 = 0.09 + 0.003 + 0
183	W12X53	0.228 = 0.225 + 0.003 + 0
184	W12X53	0.247 = 0.244 + 0.003 + 0
185	W12X53	0.27 = 0.267 + 0.003 + 0
186	W12X53	0.3 = 0.297 + 0.003 + 0
187	W12X53	0.314 = 0.311 + 0.003 + 0
188	W12X53	0.315 = 0.312 + 0.003 + 0
189	W12X53	0.301 = 0.298 + 0.003 + 0
190	W12X53	0.271 = 0.268 + 0.003 + 0
191	W12X53	0.248 = 0.245 + 0.003 + 0
192	W12X53	0.229 = 0.226 + 0.003 + 0
193	W12X53	0.094 = 0.091 + 0.003 + 0
194	W12X53	0.034 = 0.03 + 0.003 + 0
195	W12X53	0.084 = 0.081 + 0.003 + 0
230	W12X53	0.276 = 0.028 + 0.244 + 0.004
232	W12X53	0.273 = 0.028 + 0.241 + 0.004
233	W12X53	0.195 = 0.011 + 0.093 + 0.091
254	W12X45	0.423 = 0.407 + 0.015 + 0
255	W12X45	0.383 = 0.368 + 0.015 + 0
262	W12X53	0.322 = 0.319 + 0.003 + 0
263	W12X53	0.327 = 0.324 + 0.003 + 0
264	W12X53	0.084 = 0.081 + 0.003 + 0
265	W12X53	0.014 = 0.01 + 0.003 + 0
266	W12X53	0.042 = 0.039 + 0.003 + 0
267	W12X53	0.082 = 0.078 + 0.003 + 0
268	W12X53	0.229 = 0.226 + 0.003 + 0
269	W12X53	0.266 = 0.263 + 0.003 + 0
270	W12X53	0.271 = 0.268 + 0.003 + 0
271	W12X53	0.27 = 0.267 + 0.003 + 0
272	W12X53	0.27 = 0.267 + 0.003 + 0
273	W12X53	0.272 = 0.269 + 0.003 + 0

274	W12X53	0.267 = 0.264 + 0.003 + 0
275	W12X53	0.231 = 0.228 + 0.003 + 0
276	W12X53	0.083 = 0.08 + 0.003 + 0
277	W12X53	0.044 = 0.04 + 0.003 + 0
278	W12X53	0.012 = 0.009 + 0.003 + 0
313	W12X53	0.119 = 0.025 + 0.089 + 0.006
314	W12X53	0.199 = 0.014 + 0.094 + 0.091
315	W12X53	0.12 = 0.023 + 0.091 + 0.006
341	W12X53	0.08 = 0.077 + 0.003 + 0
342	W12X53	0.315 = 0.312 + 0.003 + 0
343	W12X53	0.119 = 0.025 + 0.089 + 0.005
346	W12X72	0.305 = 0.295 + 0.006 + 0.004
347	W12X72	0.536 = 0.499 + 0.037 + 2.166E-04
348	W12X72	0.536 = 0.498 + 0.037 + 2.125E-04
349	W12X72	0.304 = 0.294 + 0.006 + 0.004
350	W12X72	0.088 = 0.065 + 0.023 + 0
352	W12X72	0.563 = 0.523 + 0.04 + 0
353	W12X72	0.562 = 0.523 + 0.04 + 0
354	W12X72	0.088 = 0.066 + 0.022 + 2.324E-04
355	W12X72	0.307 = 0.297 + 0.006 + 0.004
356	W12X72	0.536 = 0.499 + 0.037 + 2.025E-04
357	W12X72	0.536 = 0.498 + 0.037 + 2.138E-04
358	W12X72	0.29 = 0.279 + 0.006 + 0.006
359	W12X45	0.089 = 0.072 + 0.017 + 0
362	W12X53	0.276 = 0.029 + 0.244 + 0.004
416	W12X87	5.711 = 5.632 + 0.035 + 0.043
418	W12X87	5.731 = 5.652 + 0.036 + 0.043
420	W12X87	5.248 = 5.2 + 0.049 + 0
423	W12X87	5.246 = 5.197 + 0.049 + 0
425	W12X87	5.707 = 5.628 + 0.035 + 0.043
427	W12X87	5.707 = 5.63 + 0.035 + 0.042
429	W12X53	0.194 = 0.011 + 0.093 + 0.091
431	W12X53	0.201 = 0.013 + 0.094 + 0.094
546	W8X40	0.041 = 0.039 + 0.003 + 0
547	W8X31	0.09 = 0.065 + 0.026 + 0
548	W8X40	0.302 = 0.282 + 0.02 + 0
549	W8X40	0.303 = 0.283 + 0.02 + 0
550	W8X31	0.092 = 0.066 + 0.026 + 0
551	W8X40	0.041 = 0.038 + 0.003 + 0
552	W8X40	0.018 = 0.016 + 0.003 + 0
553	W8X31	0.067 = 0.039 + 0.028 + 0
554	W8X40	0.071 = 0.047 + 0.024 + 0

555	W8X40	0.072 = 0.048 + 0.024 + 0
556	W8X31	0.067 = 0.04 + 0.027 + 0
557	W8X40	0.018 = 0.016 + 0.003 + 0
558	W8X48	0.015 = 0.012 + 0.003 + 0
559	W8X48	0.672 = 0.648 + 0.024 + 0
560	W8X48	0.557 = 0.533 + 0.024 + 0
561	W8X48	0.557 = 0.533 + 0.024 + 0
562	W8X48	0.674 = 0.65 + 0.024 + 0
563	W8X48	0.014 = 0.011 + 0.003 + 0
564	W8X48	0.042 = 0.039 + 0.003 + 0
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566	W8X48	0.577 = 0.553 + 0.024 + 0
567	W8X48	0.577 = 0.553 + 0.024 + 0
568	W8X48	0.119 = 0.092 + 0.027 + 0
569	W8X48	0.042 = 0.039 + 0.003 + 0
588	W8X40	0.008 = 0.005 + 0.003 + 0
589	W8X40	0.006 = 0.003 + 0.003 + 0
590	W8X40	0.008 = 0.005 + 0.003 + 0
591	W8X40	0.006 = 0.003 + 0.003 + 0
594	W8X31	0.061 = 0.058 + 0.003 + 0
595	W8X31	0.076 = 0.049 + 0.027 + 0
596	W8X40	0.318 = 0.298 + 0.02 + 0
597	W8X40	0.32 = 0.3 + 0.02 + 0
598	W8X31	0.054 = 0.026 + 0.028 + 0
599	W8X31	0.074 = 0.071 + 0.003 + 0
600	W8X31	0.02 = 0.017 + 0.003 + 0
601	W8X31	0.066 = 0.039 + 0.028 + 0
602	W8X40	0.067 = 0.043 + 0.024 + 0
603	W8X40	0.079 = 0.056 + 0.023 + 0
604	W8X31	0.071 = 0.044 + 0.027 + 0
605	W8X31	0.022 = 0.019 + 0.003 + 0
630	W8X31	0.26 = 0.258 + 0.002 + 0
631	W8X31	0.093 = 0.061 + 0.032 + 0
632	W8X31	0.605 = 0.575 + 0.03 + 0
633	W8X31	0.607 = 0.577 + 0.03 + 0
634	W8X31	0.094 = 0.062 + 0.032 + 0
635	W8X31	0.262 = 0.259 + 0.002 + 0
636	W8X31	0.148 = 0.028 + 0.111 + 0.01
637	W8X31	0.363 = 0.001 + 0.328 + 0.034
638	W8X31	0.302 = 0.055 + 0.224 + 0.023
639	W8X31	0.302 = 0.055 + 0.224 + 0.023
640	W8X31	0.363 = 0.001 + 0.327 + 0.034

641	W8X31	0.147 = 0.027 + 0.11 + 0.009
642	W8X48	0.03 = 0.027 + 0.003 + 0
643	W8X48	0.623 = 0.599 + 0.024 + 0
644	W8X48	0.678 = 0.654 + 0.024 + 0
645	W8X48	0.676 = 0.652 + 0.024 + 0
646	W8X48	0.624 = 0.6 + 0.024 + 0
647	W8X48	0.03 = 0.027 + 0.003 + 0
648	W8X48	0.042 = 0.039 + 0.003 + 0
649	W8X48	0.12 = 0.092 + 0.027 + 0
666	W8X31	0.261 = 0.259 + 0.002 + 0
667	W8X31	0.09 = 0.058 + 0.032 + 0
668	W8X31	0.649 = 0.619 + 0.03 + 0
669	W8X31	0.625 = 0.595 + 0.03 + 0
670	W8X31	0.077 = 0.044 + 0.032 + 0
671	W8X31	0.252 = 0.25 + 0.002 + 0
672	W8X31	0.152 = 0.025 + 0.112 + 0.015
673	W8X31	0.366 = 0.004 + 0.328 + 0.034
674	W8X31	0.303 = 0.056 + 0.224 + 0.023
675	W8X31	0.304 = 0.056 + 0.226 + 0.023
676	W8X31	0.367 = 0.001 + 0.33 + 0.035
677	W8X31	0.153 = 0.028 + 0.11 + 0.015
694	W8X48	0.575 = 0.551 + 0.024 + 0
695	W8X48	0.575 = 0.551 + 0.024 + 0
696	W8X48	0.119 = 0.092 + 0.027 + 0
697	W8X48	0.042 = 0.039 + 0.003 + 0
698	W8X48	0.015 = 0.012 + 0.003 + 0
699	W8X48	0.675 = 0.651 + 0.024 + 0
700	W8X48	0.56 = 0.536 + 0.024 + 0
701	W8X48	0.554 = 0.53 + 0.024 + 0
702	W8X48	0.669 = 0.645 + 0.024 + 0
703	W8X48	0.014 = 0.011 + 0.003 + 0
704	W8X31	0.041 = 0.007 + 0.028 + 0.005
705	L6X6X7/8	0.129 = 0.051 + 0.047 + 0.031
706	L6X6X7/8	0.463 = 0.427 + 0.016 + 0.02
707	L6X6X7/8	0.16 = 0.064 + 0.069 + 0.027
708	L6X6X5/16	0.217 = 0.19 + 0.018 + 0.009
709	L6X6X5/16	0.302 = 0.216 + 0.062 + 0.024
710	L6X6X5/16	0.055 = 0.026 + 0.016 + 0.013
711	L6X6X5/16	0.183 = 0.115 + 0.043 + 0.026
712	L6X6X5/16	0.204 = 0.154 + 0.037 + 0.013
713	L6X6X5/16	0.195 = 0.132 + 0.052 + 0.01
714	L6X6X5/16	0.194 = 0.132 + 0.052 + 0.011

715	L6X6X5/16	0.204 = 0.154 + 0.037 + 0.013
716	L6X6X5/16	0.183 = 0.115 + 0.042 + 0.026
717	L6X6X5/16	0.056 = 0.027 + 0.016 + 0.013
718	L6X6X5/16	0.303 = 0.216 + 0.062 + 0.024
719	L6X6X5/16	0.218 = 0.191 + 0.018 + 0.009
720	L6X6X7/8	0.161 = 0.064 + 0.069 + 0.027
721	L6X6X7/8	0.466 = 0.429 + 0.017 + 0.02
722	L6X6X7/8	0.129 = 0.051 + 0.047 + 0.031
723	W8X31	0.034 = 0.001 + 0.03 + 0.003
724	W8X31	0.12 = 0.085 + 0.023 + 0.012
725	L6X6X7/8	0.546 = 0.514 + 0.003 + 0.029
726	L6X6X7/8	0.129 = 0.025 + 0.072 + 0.033
727	L6X6X7/8	0.328 = 0.296 + 0.018 + 0.013
728	L6X6X5/16	0.289 = 0.187 + 0.079 + 0.023
729	L6X6X5/16	0.118 = 0.092 + 0.007 + 0.019
730	L6X6X5/16	0.314 = 0.238 + 0.05 + 0.026
731	L6X6X5/16	0.186 = 0.145 + 0.027 + 0.014
732	L6X6X5/16	0.188 = 0.124 + 0.058 + 0.006
733	L6X6X5/16	0.217 = 0.162 + 0.044 + 0.01
734	L6X6X5/16	0.217 = 0.163 + 0.044 + 0.01
735	L6X6X5/16	0.188 = 0.123 + 0.059 + 0.006
736	L6X6X5/16	0.186 = 0.146 + 0.027 + 0.014
737	L6X6X5/16	0.314 = 0.238 + 0.05 + 0.026
738	L6X6X5/16	0.117 = 0.091 + 0.007 + 0.019
739	L6X6X5/16	0.288 = 0.187 + 0.078 + 0.023
740	L6X6X7/8	0.325 = 0.293 + 0.018 + 0.013
741	L6X6X7/8	0.129 = 0.025 + 0.072 + 0.032
742	L6X6X7/8	0.544 = 0.512 + 0.004 + 0.028
743	W8X31	0.123 = 0.09 + 0.021 + 0.013
744	W8X31	0.049 = 0.046 + 0.003 + 0
745	W8X31	0.024 = 0.021 + 0.003 + 0
746	W8X31	0.029 = 0.025 + 0.003 + 0
747	W8X31	0.037 = 0.033 + 0.003 + 0
748	W8X31	0.046 = 0.043 + 0.003 + 0
749	W8X31	0.049 = 0.045 + 0.003 + 0
750	W8X31	0.049 = 0.045 + 0.003 + 0
751	W8X31	0.046 = 0.043 + 0.003 + 0
752	W8X31	0.037 = 0.033 + 0.003 + 0
753	W8X31	0.029 = 0.025 + 0.003 + 0
754	W8X31	0.024 = 0.02 + 0.003 + 0
755	W8X31	0.049 = 0.046 + 0.003 + 0
756	W8X31	0.04 = 0.007 + 0.028 + 0.005

757	L6X6X7/8	0.137 = 0.033 + 0.07 + 0.034
758	L6X6X7/8	0.474 = 0.435 + 0.021 + 0.019
759	L6X6X7/8	0.163 = 0.064 + 0.077 + 0.023
760	L6X6X5/16	0.225 = 0.194 + 0.03 + 0.001
761	L6X6X5/16	0.304 = 0.217 + 0.058 + 0.03
762	L6X6X5/16	0.056 = 0.03 + 0.014 + 0.012
763	L6X6X5/16	0.184 = 0.115 + 0.043 + 0.026
764	L6X6X5/16	0.196 = 0.152 + 0.033 + 0.011
765	L6X6X5/16	0.181 = 0.133 + 0.044 + 0.005
766	L6X6X5/16	0.179 = 0.131 + 0.043 + 0.005
767	L6X6X5/16	0.199 = 0.155 + 0.034 + 0.01
768	L6X6X5/16	0.182 = 0.114 + 0.042 + 0.026
769	L6X6X5/16	0.05 = 0.024 + 0.014 + 0.012
770	L6X6X5/16	0.303 = 0.216 + 0.058 + 0.03
771	L6X6X5/16	0.216 = 0.185 + 0.031 + 0.001
772	L6X6X7/8	0.162 = 0.062 + 0.077 + 0.023
773	L6X6X7/8	0.459 = 0.418 + 0.022 + 0.019
774	L6X6X7/8	0.147 = 0.042 + 0.071 + 0.034
775	W8X31	0.033 = 3.188E-04 + 0.03 + 0.002
776	W8X31	0.118 = 0.083 + 0.024 + 0.012
777	L6X6X7/8	0.547 = 0.503 + 0.036 + 0.008
778	L6X6X7/8	0.13 = 0.023 + 0.077 + 0.03
779	L6X6X7/8	0.32 = 0.285 + 0.022 + 0.014
780	L6X6X5/16	0.291 = 0.187 + 0.072 + 0.032
781	L6X6X5/16	0.114 = 0.09 + 0.018 + 0.006
782	L6X6X5/16	0.314 = 0.237 + 0.049 + 0.027
783	L6X6X5/16	0.185 = 0.146 + 0.026 + 0.013
784	L6X6X5/16	0.182 = 0.123 + 0.056 + 0.004
785	L6X6X5/16	0.214 = 0.163 + 0.044 + 0.007
786	L6X6X5/16	0.211 = 0.161 + 0.043 + 0.008
787	L6X6X5/16	0.184 = 0.124 + 0.056 + 0.004
788	L6X6X5/16	0.182 = 0.144 + 0.025 + 0.013
789	L6X6X5/16	0.315 = 0.238 + 0.05 + 0.027
790	L6X6X5/16	0.122 = 0.099 + 0.018 + 0.006
791	L6X6X5/16	0.291 = 0.187 + 0.072 + 0.032
792	L6X6X7/8	0.337 = 0.301 + 0.022 + 0.013
793	L6X6X7/8	0.131 = 0.025 + 0.076 + 0.03
794	L6X6X7/8	0.561 = 0.516 + 0.036 + 0.009
795	W8X31	0.124 = 0.09 + 0.021 + 0.013
796	W8X31	0.049 = 0.046 + 0.003 + 0
797	W8X31	0.023 = 0.02 + 0.003 + 0
798	W8X31	0.029 = 0.025 + 0.003 + 0

799	W8X31	0.037 = 0.033 + 0.003 + 0
800	W8X31	0.046 = 0.043 + 0.003 + 0
801	W8X31	0.049 = 0.045 + 0.003 + 0
802	W8X31	0.049 = 0.045 + 0.003 + 0
803	W8X31	0.046 = 0.043 + 0.003 + 0
804	W8X31	0.036 = 0.033 + 0.003 + 0
805	W8X31	0.028 = 0.025 + 0.003 + 0
806	W8X31	0.024 = 0.021 + 0.003 + 0
807	W8X31	0.049 = 0.046 + 0.003 + 0
808	W8X58	0.382 = 0.258 + 0.118 + 0.007
809	W8X58	0.635 = 0.497 + 0.134 + 0.004
810	W8X31	0.429 = 0.35 + 0.079 + 0
811	W8X31	0.246 = 0.217 + 0.029 + 0
812	W8X31	0.3 = 0.206 + 0.095 + 0
813	W8X31	0.299 = 0.204 + 0.095 + 0
814	W8X31	0.229 = 0.2 + 0.029 + 0
815	W8X31	0.427 = 0.348 + 0.079 + 0
816	W8X58	0.605 = 0.467 + 0.133 + 0.005
817	W8X58	0.386 = 0.259 + 0.119 + 0.009
818	L8X8X1/2	0.29 = 0.062 + 0.126 + 0.103
819	L8X8X1/2	0.439 = 0.049 + 0.229 + 0.161
820	L8X8X1/2	0.629 = 0.295 + 0.196 + 0.138
821	L8X8X1/2	0.555 = 0.221 + 0.196 + 0.138
822	L8X8X1/2	0.55 = 0.216 + 0.196 + 0.138
823	L8X8X1/2	0.632 = 0.298 + 0.196 + 0.138
824	L8X8X1/2	0.435 = 0.045 + 0.229 + 0.161
825	L8X8X1/2	0.297 = 0.1 + 0.108 + 0.088
826	L8X8X1/2	0.282 = 0.053 + 0.126 + 0.103
827	L8X8X1/2	0.436 = 0.046 + 0.229 + 0.161
828	L8X8X1/2	0.628 = 0.294 + 0.196 + 0.138
829	L8X8X1/2	0.555 = 0.221 + 0.196 + 0.138
830	L8X8X1/2	0.55 = 0.216 + 0.196 + 0.138
831	L8X8X1/2	0.633 = 0.299 + 0.196 + 0.138
832	L8X8X1/2	0.436 = 0.046 + 0.229 + 0.161
833	L8X8X1/2	0.293 = 0.064 + 0.126 + 0.103
834	W8X58	0.388 = 0.261 + 0.119 + 0.009
835	W8X58	0.609 = 0.47 + 0.133 + 0.006
836	W8X31	0.429 = 0.35 + 0.079 + 0
837	W8X31	0.255 = 0.085 + 0.17 + 0
838	W8X31	0.301 = 0.207 + 0.094 + 0
839	W8X31	0.298 = 0.204 + 0.095 + 0
840	W8X31	0.216 = 0.187 + 0.029 + 0

841	W8X31	0.427 = 0.348 + 0.079 + 0
842	W8X58	0.634 = 0.5 + 0.131 + 0.002
843	W8X58	0.362 = 0.242 + 0.118 + 0.002
844	W8X31	0.347 = 0.006 + 0.295 + 0.046
845	W8X31	0.34 = 0.04 + 0.294 + 0.005
846	W8X31	0.397 = 0.072 + 0.298 + 0.026
847	W8X31	0.397 = 0.072 + 0.297 + 0.027
848	W8X31	0.339 = 0.04 + 0.294 + 0.005
849	W8X31	0.346 = 0.006 + 0.295 + 0.046
850	W8X31	0.16 = 0.004 + 0.155 + 4.708E-04
851	W8X31	0.447 = 0.007 + 0.44 + 0
852	W8X31	0.183 = 0.007 + 0.176 + 1.177E-04
853	W8X31	0.18 = 0.004 + 0.176 + 0
854	W8X31	0.444 = 0.006 + 0.439 + 1.299E-04
855	W8X31	0.158 = 0.004 + 0.154 + 0.001
856	W8X31	0.346 = 0.006 + 0.296 + 0.045
857	W8X31	0.341 = 0.041 + 0.295 + 0.005
858	W8X31	0.399 = 0.073 + 0.299 + 0.027
859	W8X31	0.395 = 0.072 + 0.296 + 0.026
860	W8X31	0.339 = 0.041 + 0.293 + 0.005
861	W8X31	0.343 = 0.006 + 0.293 + 0.044
862	W8X31	0.021 = 4.23E-04 + 0.002 + 0.018
863	W8X31	0.028 = 0.002 + 0.004 + 0.022
864	W8X31	0.017 = 0.011 + 0.002 + 0.004
865	W8X31	0.018 = 0.003 + 0.001 + 0.014
866	W8X31	0.028 = 0.001 + 0.001 + 0.025
867	W8X31	0.034 = 0.001 + 0.002 + 0.032
868	W8X31	0.041 = 0.002 + 0.003 + 0.036
869	W8X31	0.026 = 0.001 + 0.004 + 0.02
870	W8X31	0.018 = 2.774E-04 + 0.003 + 0.015
871	W8X31	0.018 = 0.011 + 0.002 + 0.004
872	W8X31	0.018 = 0.003 + 0.001 + 0.014
873	W8X31	0.028 = 0.001 + 0.001 + 0.025
874	W8X31	0.035 = 0.001 + 0.002 + 0.032
875	W8X31	0.042 = 0.002 + 0.003 + 0.037
876	W8X31	0.021 = 4.221E-04 + 0.002 + 0.018
877	W8X31	0.028 = 0.002 + 0.004 + 0.022
878	W8X31	0.035 = 0.001 + 0.002 + 0.032
879	W8X31	0.027 = 0.001 + 0.001 + 0.025
880	W8X31	0.017 = 0.003 + 0.001 + 0.014
881	W8X31	0.018 = 0.011 + 0.002 + 0.004
882	W8X31	0.017 = 0.011 + 0.002 + 0.003

883	W8X31	0.018 = 0.003 + 0.001 + 0.014
884	W8X31	0.027 = 0.001 + 0.001 + 0.025
885	W8X31	0.034 = 0.001 + 0.002 + 0.032
886	W8X31	0.041 = 0.002 + 0.003 + 0.036
887	W8X31	0.042 = 0.002 + 0.003 + 0.037
888	W8X31	0.025 = 0.001 + 0.004 + 0.019
889	W8X31	0.017 = 1.984E-04 + 0.003 + 0.014

Hangar - Beam 347

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	B166	347	58.28148	DStlS2-1	Special Concentrically Braced Frame	W12X72	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
98.35000	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r/P_y$	$\alpha P_r/P_e$	τ_b	EA factor	EI factor
-0.449	-1.313	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft²)	J (ft⁴)	I_{33} (ft⁴)	I_{22} (ft⁴)	A_{v3} (ft²)	A_{v2} (ft²)
0.1465	0.000141	0.028791	0.009404	0.1117	0.0367

Design Properties

S_{33} (ft³)	S_{22} (ft³)	Z_{33} (ft³)	Z_{22} (ft³)	r_{33} (ft)	r_{22} (ft)	C_w (ft⁶)
0.056177	0.018808	0.0625	0.028472	0.44327	0.25333	0.002185

Material Properties

E (lb/ft²)	f_y (lb/ft²)	R_y	C_{pr}	α
4176000000	7200000	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	98.35	0.44327	221.876	200	221.876	300
Minor (2-2)	1	32.78333	0.25333	129.407	200	129.407	300

Stress Check Forces and Moments

Location (ft)	P_u (lb)	M_{u33} (lb-ft)	M_{u22} (lb-ft)	V_{u2} (lb)	V_{u3} (lb)	T_u (lb-ft)
58.28148	473556.76	15169.01	-44.96	-1374.77	-5.76	0.31

Axial Force & Biaxial Moment Design Factors (H1.2,H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	1	1	1	1	1
Minor Bending	0.333	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.333	1	1.284

Demand/Capacity (D/C) Ratio Eqn.(H1.2,H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.536 =	0.499 + 0.037 + 2.166E-04

Axial Force and Capacities

P_u Force (lb)	ϕP_{nc} Capacity (lb)	ϕP_{nt} Capacity (lb)
473556.76	96828.43	949500

Moments and Capacities

	M_u Moment (lb-ft)	ϕM_n (lb-ft)	ϕM_n No LTB (lb-ft)	ϕM_n Cb=1 (lb-ft)
Major Bending	15169.01	360979.13	405000	281179.04
Minor Bending	44.96	184500		

Shear Design

	V_u Force (lb)	ϕV_n Capacity (lb)	Stress Ratio
Major Shear	1374.77	158670	0.009
Minor Shear	5.76	434160	1.327E-05

Hangar - Beam 418

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story33	B192	418	0.55074	DStlS2-1	Special Concentrically Braced Frame	W12X87	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
121.06371	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r/P_y$	$\alpha P_r/P_e$	τ_b	EA factor	EI factor
0.256	0.1	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft²)	J (ft⁴)	I_{33} (ft⁴)	I_{22} (ft⁴)	A_{v3} (ft²)	A_{v2} (ft²)
0.1778	0.000246	0.035687	0.011622	0.1361	0.0447

Design Properties

S_{33} (ft³)	S_{22} (ft³)	Z_{33} (ft³)	Z_{22} (ft³)	r_{33} (ft)	r_{22} (ft)	C_w (ft⁶)
0.068519	0.023052	0.076389	0.034954	0.44804	0.25569	0.002736

Material Properties

E (lb/ft²)	f_y (lb/ft²)	R_y	C_{pr}	α
4176000000	7200000	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	11.18823	0.44804	24.972	200	24.972	300
Minor (2-2)	1	12.10637	0.25569	47.349	200	47.349	300

Stress Check Forces and Moments

Location (ft)	P_u (lb)	M_{u33} (lb-ft)	M_{u22} (lb-ft)	V_{u2} (lb)	V_{u3} (lb)	T_u (lb-ft)
0.55074	-328073.13	-146993.13	-192.78	-18942.29	25.53	146.1

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.092	1	1	1	1	1
Minor Bending	0.1	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.1	1	1.293

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.6 =	0.336 + 0.264 + 0.001

Axial Force and Capacities

P_u Force (lb)	ϕP_{nc} Capacity (lb)	ϕP_{nt} Capacity (lb)
328073.13	977828.47	1152000

Moments and Capacities

	M_u Moment (lb-ft)	ϕM_n (lb-ft)	ϕM_n No LTB (lb-ft)	ϕM_n Cb=1 (lb-ft)
Major Bending	146993.13	495000	495000	487687.42
Minor Bending	192.78	226500		

Shear Design

	V_u Force (lb)	ϕV_n Capacity (lb)	Stress Ratio
Major Shear	18942.29	193125	0.098
Minor Shear	25.53	529254	4.823E-05

End Reaction Major Shear Forces

Left End Reaction (lb)	Load Combo	Right End Reaction (lb)	Load Combo
		16120.49	DStlS2-2

Hangar - Beam 668

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	B126	668	13.24615	DStlS2-1	Special Concentrically Braced Frame	W8X31	Non-Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
24.60000	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r/P_y$	$\alpha P_r/P_e$	τ_b	EA factor	EI factor
0.13	0.489	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft²)	J (ft⁴)	I_{33} (ft⁴)	I_{22} (ft⁴)	A_{v3} (ft²)	A_{v2} (ft²)
0.0634	0.000026	0.005305	0.001789	0.0483	0.0158

Design Properties

S_{33} (ft³)	S_{22} (ft³)	Z_{33} (ft³)	Z_{22} (ft³)	r_{33} (ft)	r_{22} (ft)	C_w (ft⁶)
0.015914	0.005367	0.017593	0.00816	0.28925	0.16798	0.000178

Material Properties

E (lb/ft²)	f_y (lb/ft²)	R_y	C_{pr}	α
4176000000	7200000	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	24.6	0.28925	85.046	200	85.046	300
Minor (2-2)	1	24.6	0.16798	146.442	200	146.442	300

Stress Check Forces and Moments

Location (ft)	P_u (lb)	M_{u33} (lb-ft)	M_{u22} (lb-ft)	V_{u2} (lb)	V_{u3} (lb)	T_u (lb-ft)
13.24615	-59552.85	2803.42	0	27.77	-25.73	-2.89

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	1	1	1	1	1
Minor Bending	1	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
1	1	1.136

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.649 =	0.619 + 0.03 + 0

Axial Force and Capacities

P_u Force (lb)	ϕP_{nc} Capacity (lb)	ϕP_{nt} Capacity (lb)
59552.85	96179.16	410850

Moments and Capacities

	M_u Moment (lb-ft)	ϕM_n (lb-ft)	ϕM_n No LTB (lb-ft)	ϕM_n Cb=1 (lb-ft)
Major Bending	2803.42	82519.11	113877.37	72616.82
Minor Bending	0	52791.33		

Shear Design

	V_u Force (lb)	ϕV_n Capacity (lb)	Stress Ratio
Major Shear	27.77	68400	4.059E-04
Minor Shear	25.73	187920	1.369E-04

Hangar – All column sections with DC ratios (Gravity Design)

Label	Section	D/C Ratio
2	W10X68	0.05 = 0.024 + 0.009 + 0.017
3	W10X68	0.047 = 0.043 + 0.004 + 0.001
4	W10X68	0.056 = 0.026 + 0.009 + 0.021
5	W10X68	0.032 = 0.006 + 0.01 + 0.016
7	W8X31	0.007 = 0.007 + 0 + 0
8	W10X68	0.336 = 0.313 + 0.017 + 0.006
9	W10X68	0.102 = 0.072 + 0.021 + 0.009
10	W10X68	0.345 = 0.321 + 0.017 + 0.007
11	W10X68	0.105 = 0.074 + 0.021 + 0.01
30	W8X31	0.092 = 0.092 + 0 + 0
31	W8X31	0.085 = 0.085 + 0 + 0
32	W8X31	0.071 = 0.071 + 0 + 0
33	W8X31	0.067 = 0.067 + 0 + 0
34	W8X31	0.055 = 0.055 + 0 + 0
35	W8X31	0.041 = 0.041 + 0 + 0
36	W8X31	0.027 = 0.027 + 0 + 0
37	W8X31	0.014 = 0.014 + 0 + 0
38	W8X31	0.014 = 0.014 + 0 + 0
39	W8X31	0.027 = 0.027 + 0 + 0
40	W8X31	0.041 = 0.041 + 0 + 0
41	W8X31	0.055 = 0.055 + 0 + 0
42	W8X31	0.067 = 0.067 + 0 + 0
43	W8X31	0.071 = 0.071 + 0 + 0
44	W8X31	0.085 = 0.085 + 0 + 0
45	W8X31	0.092 = 0.092 + 0 + 0
85	W10X68	0.077 = 0.023 + 0.031 + 0.023
86	W10X68	0.103 = 0.046 + 0.031 + 0.027
87	W10X68	0.088 = 0.022 + 0.032 + 0.034
90	W8X31	0.013 = 0.013 + 0 + 0
91	W10X68	0.306 = 0.263 + 0.023 + 0.02
92	W10X68	0.389 = 0.256 + 0.005 + 0.128
93	W10X68	0.286 = 0.242 + 0.023 + 0.021
94	W10X68	0.386 = 0.252 + 0.005 + 0.129
113	W8X31	0.506 = 0.506 + 0 + 0
114	W8X31	0.473 = 0.473 + 0 + 0
115	W8X31	0.402 = 0.402 + 0 + 0
116	W8X31	0.32 = 0.32 + 0 + 0
117	W8X31	0.26 = 0.26 + 0 + 0
118	W8X31	0.098 = 0.098 + 0 + 0

119	W8X31	0.069 = 0.069 + 0 + 0
120	W8X31	0.043 = 0.043 + 0 + 0
121	W8X31	0.043 = 0.043 + 0 + 0
122	W8X31	0.069 = 0.069 + 0 + 0
123	W8X31	0.098 = 0.098 + 0 + 0
124	W8X31	0.26 = 0.26 + 0 + 0
125	W8X31	0.32 = 0.32 + 0 + 0
126	W8X31	0.402 = 0.402 + 0 + 0
127	W8X31	0.473 = 0.473 + 0 + 0
128	W8X31	0.506 = 0.506 + 0 + 0
147	W10X68	0.255 = 0.215 + 0.027 + 0.014
168	W10X68	0.077 = 0.023 + 0.031 + 0.023
169	W10X68	0.116 = 0.052 + 0.031 + 0.034
170	W10X68	0.093 = 0.019 + 0.032 + 0.042
173	W8X31	0.013 = 0.013 + 0 + 0
174	W10X68	0.293 = 0.251 + 0.023 + 0.02
175	W10X68	0.389 = 0.256 + 0.005 + 0.128
176	W10X68	0.307 = 0.265 + 0.023 + 0.019
177	W10X68	0.385 = 0.254 + 0.004 + 0.127
196	W8X31	0.507 = 0.507 + 0 + 0
197	W8X31	0.475 = 0.475 + 0 + 0
198	W8X31	0.403 = 0.403 + 0 + 0
199	W8X31	0.322 = 0.322 + 0 + 0
200	W8X31	0.261 = 0.261 + 0 + 0
201	W8X31	0.098 = 0.098 + 0 + 0
202	W8X31	0.07 = 0.07 + 0 + 0
203	W8X31	0.044 = 0.044 + 0 + 0
204	W8X31	0.043 = 0.043 + 0 + 0
205	W8X31	0.069 = 0.069 + 0 + 0
206	W8X31	0.097 = 0.097 + 0 + 0
207	W8X31	0.259 = 0.259 + 0 + 0
208	W8X31	0.319 = 0.319 + 0 + 0
209	W8X31	0.401 = 0.401 + 0 + 0
210	W8X31	0.472 = 0.472 + 0 + 0
211	W8X31	0.504 = 0.504 + 0 + 0
251	W10X68	0.05 = 0.024 + 0.009 + 0.017
252	W10X68	0.063 = 0.057 + 0.004 + 0.002
253	W10X68	0.066 = 0.03 + 0.009 + 0.027
256	W8X31	0.007 = 0.007 + 0 + 0
257	W10X68	0.321 = 0.301 + 0.015 + 0.005
258	W10X68	0.102 = 0.072 + 0.021 + 0.009
259	W10X68	0.346 = 0.325 + 0.017 + 0.005

260	W10X68	0.101 = 0.073 + 0.021 + 0.007
279	W8X31	0.092 = 0.092 + 0 + 0
280	W8X31	0.084 = 0.084 + 0 + 0
281	W8X31	0.071 = 0.071 + 0 + 0
282	W8X31	0.066 = 0.066 + 0 + 0
283	W8X31	0.054 = 0.054 + 0 + 0
284	W8X31	0.041 = 0.041 + 0 + 0
285	W8X31	0.027 = 0.027 + 0 + 0
286	W8X31	0.014 = 0.014 + 0 + 0
287	W8X31	0.014 = 0.014 + 0 + 0
288	W8X31	0.028 = 0.028 + 0 + 0
289	W8X31	0.042 = 0.042 + 0 + 0
290	W8X31	0.055 = 0.055 + 0 + 0
291	W8X31	0.067 = 0.067 + 0 + 0
292	W8X31	0.071 = 0.071 + 0 + 0
293	W8X31	0.085 = 0.085 + 0 + 0
294	W8X31	0.092 = 0.092 + 0 + 0
333	W12X136	0.626 = 0.432 + 0.182 + 0.012
334	W12X136	0.302 = 0.077 + 0.21 + 0.015
335	W12X136	0.61 = 0.418 + 0.183 + 0.01
336	W12X136	0.302 = 0.079 + 0.21 + 0.013
339	W8X31	0.57 = 0.57 + 0 + 0
344	W10X68	0.27 = 0.229 + 0.027 + 0.014
351	W8X31	0.571 = 0.571 + 0 + 0
361	W10X68	0.031 = 0.005 + 0.01 + 0.016
363	W8X31	0.696 = 0.696 + 0 + 0
364	W8X31	0.5 = 0.5 + 0 + 0
365	W8X31	0.407 = 0.407 + 0 + 0
366	W8X31	0.252 = 0.252 + 0 + 0
367	W8X31	0.093 = 0.093 + 0 + 0
368	W8X31	0.052 = 0.052 + 0 + 0
369	W8X31	0.017 = 0.017 + 0 + 0
370	W8X31	0.042 = 0.042 + 0 + 0
371	W8X31	0.073 = 0.073 + 0 + 0
372	W8X31	0.321 = 0.321 + 0 + 0
373	W8X31	0.074 = 0.074 + 0 + 0
374	W8X31	0.043 = 0.043 + 0 + 0
375	W8X31	0.018 = 0.018 + 0 + 0
376	W8X31	0.052 = 0.052 + 0 + 0
377	W8X31	0.092 = 0.092 + 0 + 0
378	W8X31	0.251 = 0.251 + 0 + 0
379	W8X31	0.406 = 0.406 + 0 + 0

380	W8X31	0.499 = 0.499 + 0 + 0
381	W8X31	0.704 = 0.704 + 0 + 0
404	W10X88	0.64 = 0.465 + 0.175 + 0
405	W10X88	0.395 = 0.201 + 0.194 + 0
406	W10X88	0.769 = 0.595 + 0.171 + 0.003
407	W10X88	0.288 = 0.09 + 0.196 + 0.003
410	W8X31	0.419 = 0.419 + 0 + 0
422	W8X31	0.415 = 0.415 + 0 + 0
434	W8X31	0.476 = 0.476 + 0 + 0
435	W8X31	0.439 = 0.439 + 0 + 0
436	W8X31	0.357 = 0.357 + 0 + 0
437	W8X31	0.229 = 0.229 + 0 + 0
438	W8X31	0.096 = 0.096 + 0 + 0
439	W8X31	0.055 = 0.055 + 0 + 0
440	W8X31	0.014 = 0.014 + 0 + 0
441	W8X31	0.036 = 0.036 + 0 + 0
442	W8X31	0.066 = 0.066 + 0 + 0
443	W8X31	0.236 = 0.236 + 0 + 0
444	W8X31	0.066 = 0.066 + 0 + 0
445	W8X31	0.036 = 0.036 + 0 + 0
446	W8X31	0.014 = 0.014 + 0 + 0
447	W8X31	0.055 = 0.055 + 0 + 0
448	W8X31	0.096 = 0.096 + 0 + 0
449	W8X31	0.23 = 0.23 + 0 + 0
450	W8X31	0.358 = 0.358 + 0 + 0
451	W8X31	0.441 = 0.441 + 0 + 0
452	W8X31	0.47 = 0.47 + 0 + 0
475	W12X136	0.63 = 0.437 + 0.182 + 0.012
476	W12X136	0.302 = 0.077 + 0.209 + 0.015
477	W12X136	0.647 = 0.449 + 0.182 + 0.016
478	W12X136	0.303 = 0.076 + 0.208 + 0.02
481	W8X31	0.57 = 0.57 + 0 + 0
493	W8X31	0.569 = 0.569 + 0 + 0
505	W8X31	0.697 = 0.697 + 0 + 0
506	W8X31	0.5 = 0.5 + 0 + 0
507	W8X31	0.407 = 0.407 + 0 + 0
508	W8X31	0.252 = 0.252 + 0 + 0
509	W8X31	0.092 = 0.092 + 0 + 0
510	W8X31	0.052 = 0.052 + 0 + 0
511	W8X31	0.017 = 0.017 + 0 + 0
512	W8X31	0.042 = 0.042 + 0 + 0
513	W8X31	0.073 = 0.073 + 0 + 0

514	W8X31	0.321 = 0.321 + 0 + 0
515	W8X31	0.073 = 0.073 + 0 + 0
516	W8X31	0.042 = 0.042 + 0 + 0
517	W8X31	0.017 = 0.017 + 0 + 0
518	W8X31	0.053 = 0.053 + 0 + 0
519	W8X31	0.093 = 0.093 + 0 + 0
520	W8X31	0.253 = 0.253 + 0 + 0
521	W8X31	0.408 = 0.408 + 0 + 0
522	W8X31	0.501 = 0.501 + 0 + 0
523	W8X31	0.696 = 0.696 + 0 + 0

Hangar - Column 10

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	C7	10	32.99167	DStlS2-1	Special Concentrically Braced Frame	W10X68	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
34.00000	0.858	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r/P_y$	$\alpha P_r/P_e$	τ_b	EA factor	EI factor
0.062	0.253	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft²)	J (ft⁴)	I_{33} (ft⁴)	I_{22} (ft⁴)	A_{v3} (ft²)	A_{v2} (ft²)
0.1382	0.000172	0.019001	0.006462	0.108	0.0339

Design Properties

S_{33} (ft³)	S_{22} (ft³)	Z_{33} (ft³)	Z_{22} (ft³)	r_{33} (ft)	r_{22} (ft)	C_w (ft⁶)
0.043848	0.015356	0.049363	0.023206	0.3708	0.21624	0.001027

Material Properties

E (lb/ft²)	f_y (lb/ft²)	R_y	C_{pr}	α
4176000000	7200000	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	32.99167	0.3708	88.974	200	88.974	300
Minor (2-2)	1	32.99167	0.21624	152.567	200	152.567	300

Stress Check Forces and Moments

Location (ft)	P_u (lb)	M_{u33} (lb-ft)	M_{u22} (lb-ft)	V_{u2} (lb)	V_{u3} (lb)	T_u (lb-ft)
32.99167	-61920.42	-6231.24	-1210.7	160.63	25	-2.08

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.97	1	1	1	1	0.6
Minor Bending	0.97	1	1	1	1	0.6

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.97	1	1.585

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.345 =	0.321 + 0.017 + 0.007

Axial Force and Capacities

P_u Force (lb)	ϕP_{nc} Capacity (lb)	ϕP_{nt} Capacity (lb)
61920.42	193140.59	895500

Moments and Capacities

	M_u Moment (lb-ft)	ϕM_n (lb-ft)	ϕM_n No LTB (lb-ft)	ϕM_n Cb=1 (lb-ft)
Major Bending	6231.24	319875	319875	227792.59
Minor Bending	1210.7	150375		

Shear Design

	V_u Force (lb)	ϕV_n Capacity (lb)	Stress Ratio
Major Shear	160.63	146640	0.001
Minor Shear	25	419958	5.954E-05

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	C116	333	32.975	DStlS2-1	Special Concentrically Braced Frame	W12X136	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
34.00000	0.4	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r/P_y$	$\alpha P_r/P_e$	τ_b	EA factor	EI factor
0.124	0.341	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft²)	J (ft⁴)	I_{33} (ft⁴)	I_{22} (ft⁴)	A_{v3} (ft²)	A_{v2} (ft²)
0.2771	0.000892	0.059799	0.019194	0.2153	0.0735

Design Properties

S_{33} (ft³)	S_{22} (ft³)	Z_{33} (ft³)	Z_{22} (ft³)	r_{33} (ft)	r_{22} (ft)	C_w (ft⁶)
0.107103	0.037149	0.123843	0.056713	0.46456	0.26319	0.004909

Material Properties

E (lb/ft²)	f_y (lb/ft²)	R_y	C_{pr}	α
4176000000	7200000	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	32.975	0.46456	70.981	200	70.981	300
Minor (2-2)	1	32.975	0.26319	125.288	200	125.288	300

Stress Check Forces and Moments

Location (ft)	P_u (lb)	M_{u33} (lb-ft)	M_{u22} (lb-ft)	V_{u2} (lb)	V_{u3} (lb)	T_u (lb-ft)
32.975	-247831.29	164567.27	4886.33	-4543.92	-70.55	64.85

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.97	1	1	1	1	0.6
Minor Bending	0.97	1	1	1	1	0.6

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.97	1	1.566

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.626 =	0.432 + 0.182 + 0.012

Axial Force and Capacities

P_u Force (lb)	ϕP_{nc} Capacity (lb)	ϕP_{nt} Capacity (lb)
247831.29	574236.38	1795500

Moments and Capacities

	M_u Moment (lb-ft)	ϕM_n (lb-ft)	ϕM_n No LTB (lb-ft)	ϕM_n Cb=1 (lb-ft)
Major Bending	164567.27	802500	802500	670448.6
Minor Bending	4886.33	367500		

Shear Design

	V_u Force (lb)	ϕV_n Capacity (lb)	Stress Ratio
Major Shear	4543.92	317580	0.014
Minor Shear	70.55	837000	8.429E-05

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	C171	406	32.975	DStlS2-1	Special Concentrically Braced Frame	W10X88	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
34.00000	0.418	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r/P_y$	$\alpha P_r/P_e$	τ_b	EA factor	EI factor
0.118	0.47	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft²)	J (ft⁴)	I_{33} (ft⁴)	I_{22} (ft⁴)	A_{v3} (ft²)	A_{v2} (ft²)
0.1806	0.000363	0.025752	0.008632	0.1416	0.0454

Design Properties

S_{33} (ft³)	S_{22} (ft³)	Z_{33} (ft³)	Z_{22} (ft³)	r_{33} (ft)	r_{22} (ft)	C_w (ft⁶)
0.057227	0.020114	0.065394	0.030729	0.37766	0.21865	0.001453

Material Properties

E (lb/ft²)	f_y (lb/ft²)	R_y	C_{pr}	α
4176000000	7200000	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	32.975	0.37766	87.314	200	87.314	300
Minor (2-2)	1	32.975	0.21865	150.809	200	150.809	300

Stress Check Forces and Moments

Location (ft)	P_u (lb)	M_{u33} (lb-ft)	M_{u22} (lb-ft)	V_{u2} (lb)	V_{u3} (lb)	T_u (lb-ft)
32.975	-153630.74	-81555.47	-617.81	2132.97	0.5	35.25

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.97	1	1	1	1	0.6
Minor Bending	0.97	1	1	1	1	0.6

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.97	1	1.521

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.769 =	0.595 + 0.171 + 0.003

Axial Force and Capacities

P_u Force (lb)	ϕP_{nc} Capacity (lb)	ϕP_{nt} Capacity (lb)
153630.74	258262.15	1170000

Moments and Capacities

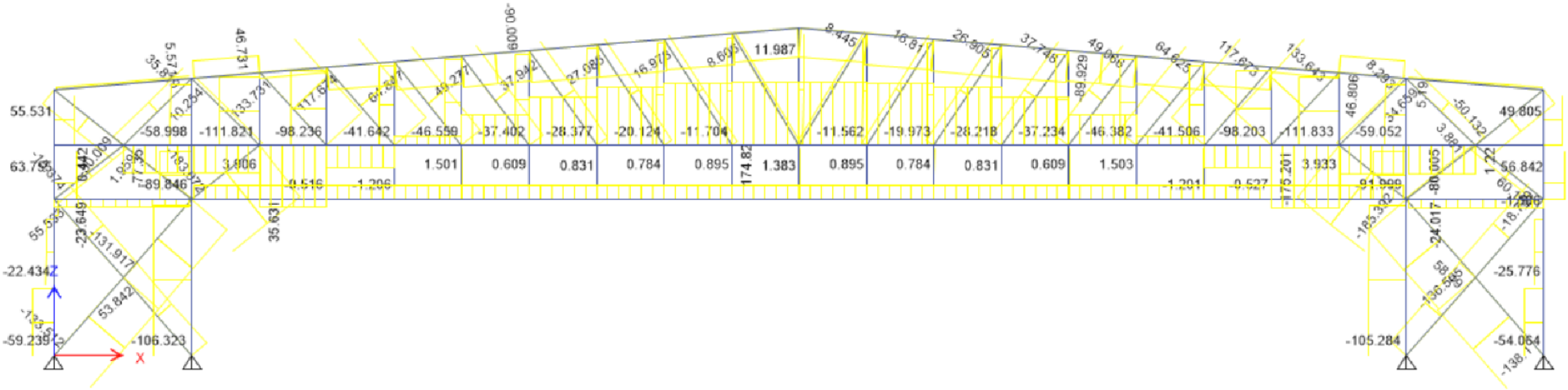
	M_u Moment (lb-ft)	ϕM_n (lb-ft)	ϕM_n No LTB (lb-ft)	ϕM_n Cb=1 (lb-ft)
Major Bending	81555.47	423750	423750	330608.17
Minor Bending	617.81	199125		

Shear Design

	V_u Force (lb)	ϕV_n Capacity (lb)	Stress Ratio
Major Shear	2132.97	196020	0.011
Minor Shear	0.5	550638	0

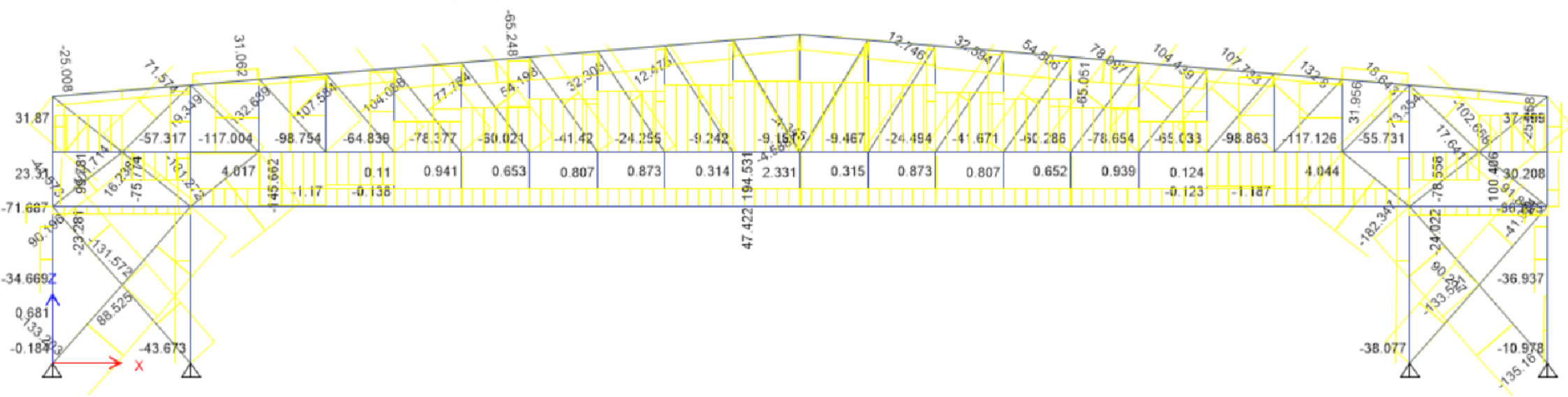
Hangar - Axial Load Distribution (1.2DL+1.6LL)

Elevation A.1



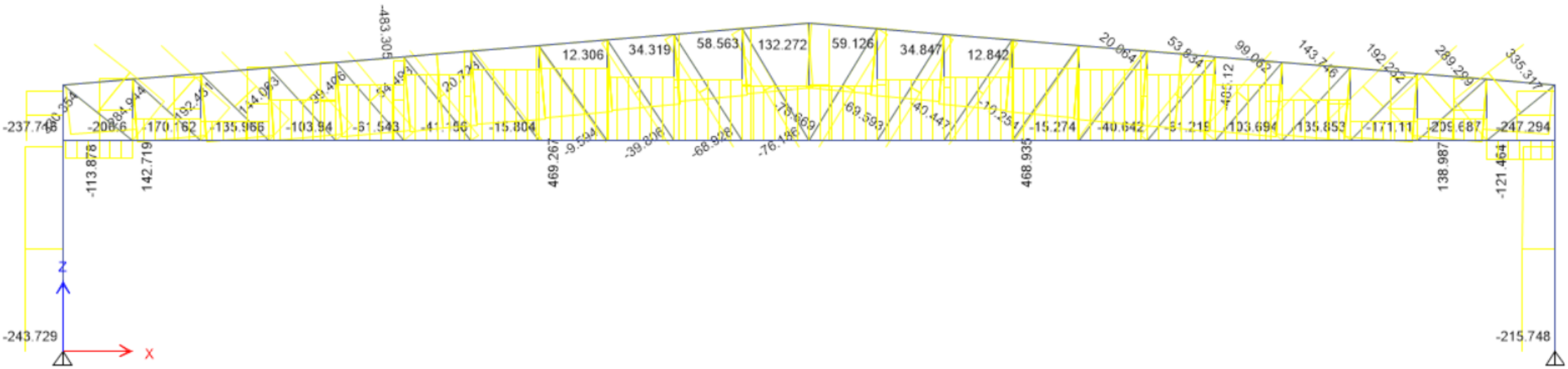
Hangar - Axial Load Distribution (1.2DL+1.6LL)

Elevation A.3



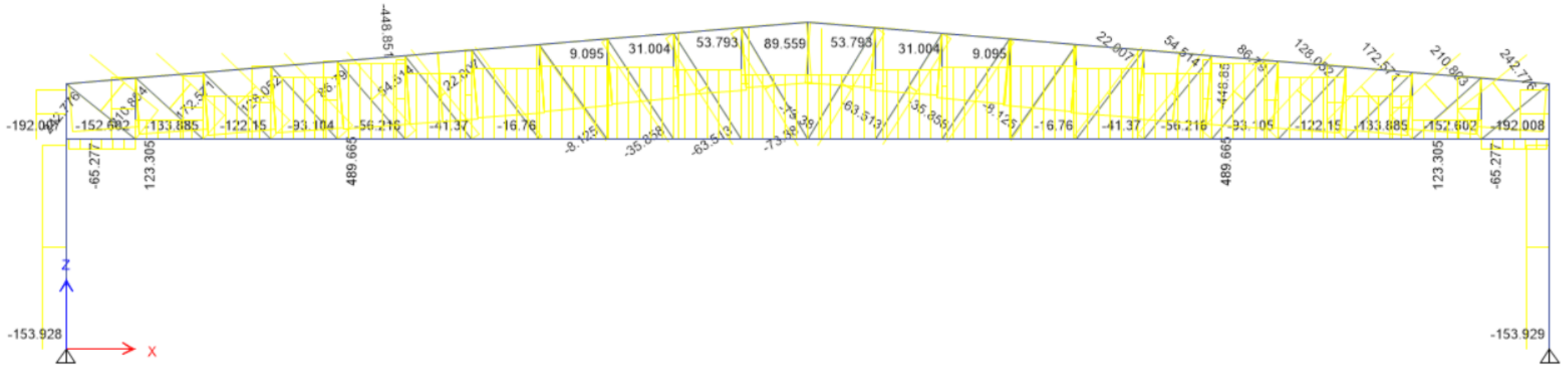
Hangar - Axial Load Distribution (1.2DL+1.6LL)

Elevation B



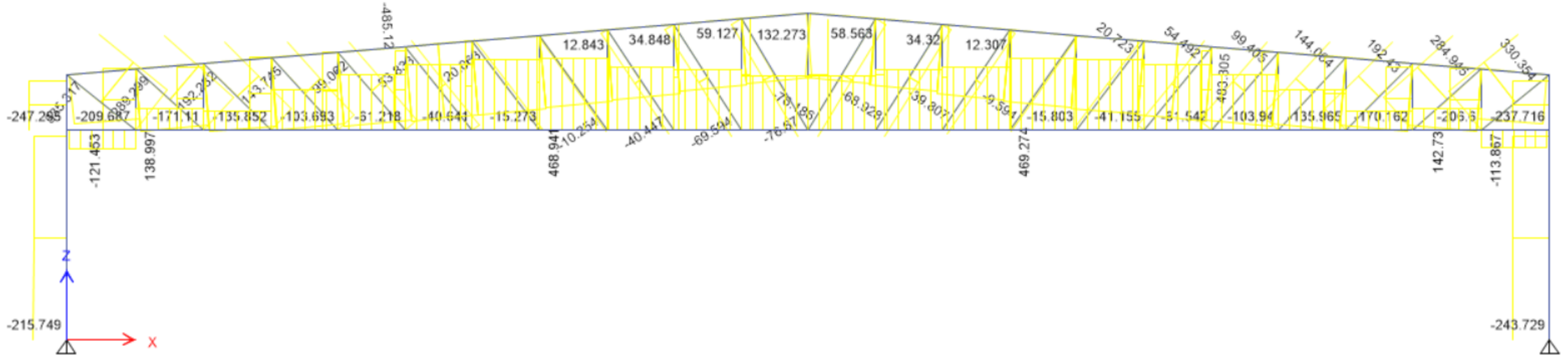
Hangar - Axial Load Distribution (1.2DL+1.6LL)

Elevation C



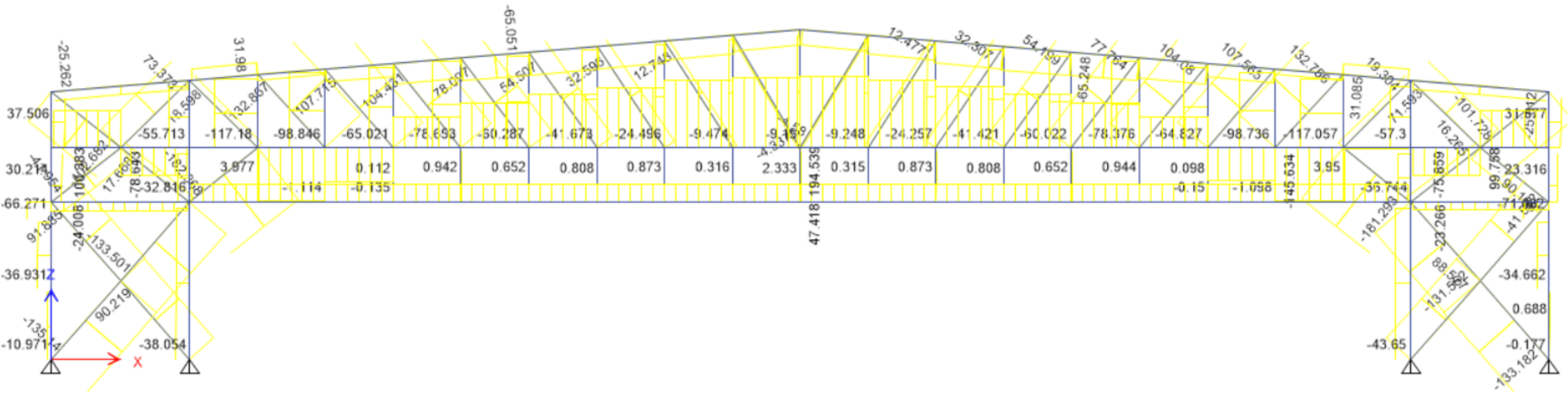
Hangar - Axial Load Distribution (1.2DL+1.6LL)

Elevation D



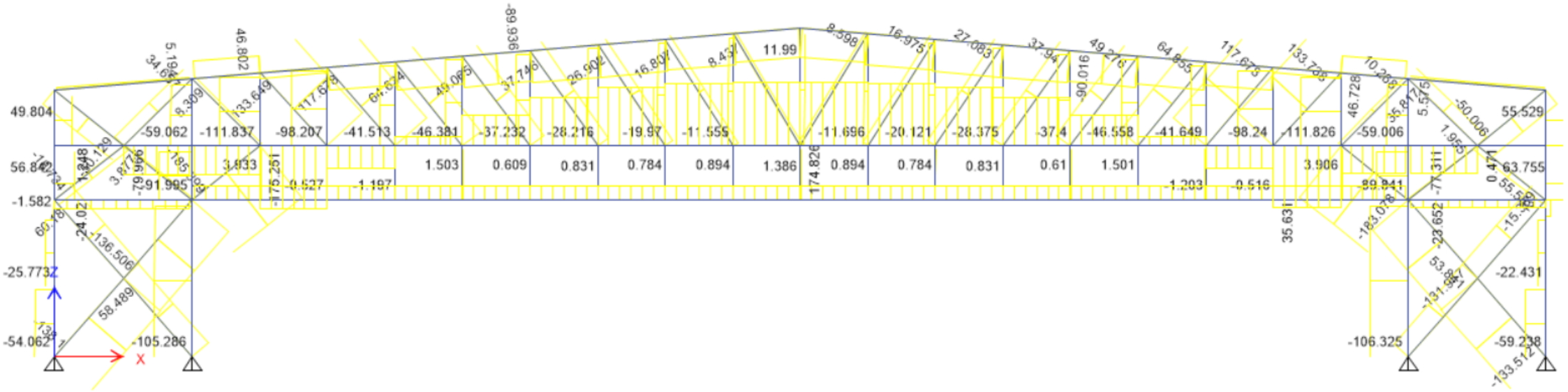
Hangar - Axial Load Distribution (1.2DL+1.6LL)

Elevation D.7

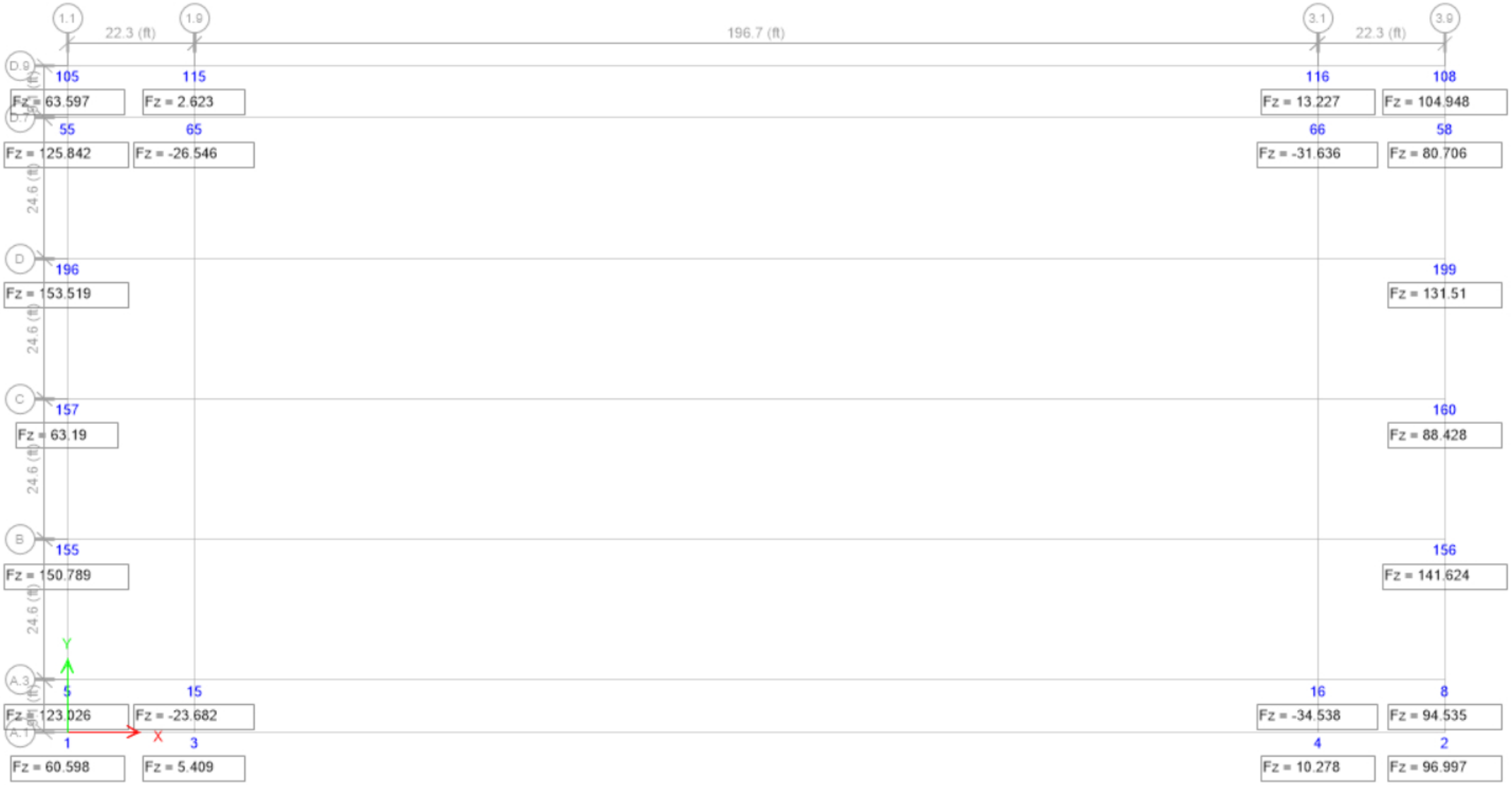


Hangar - Axial Load Distribution (1.2DL+1.6LL)

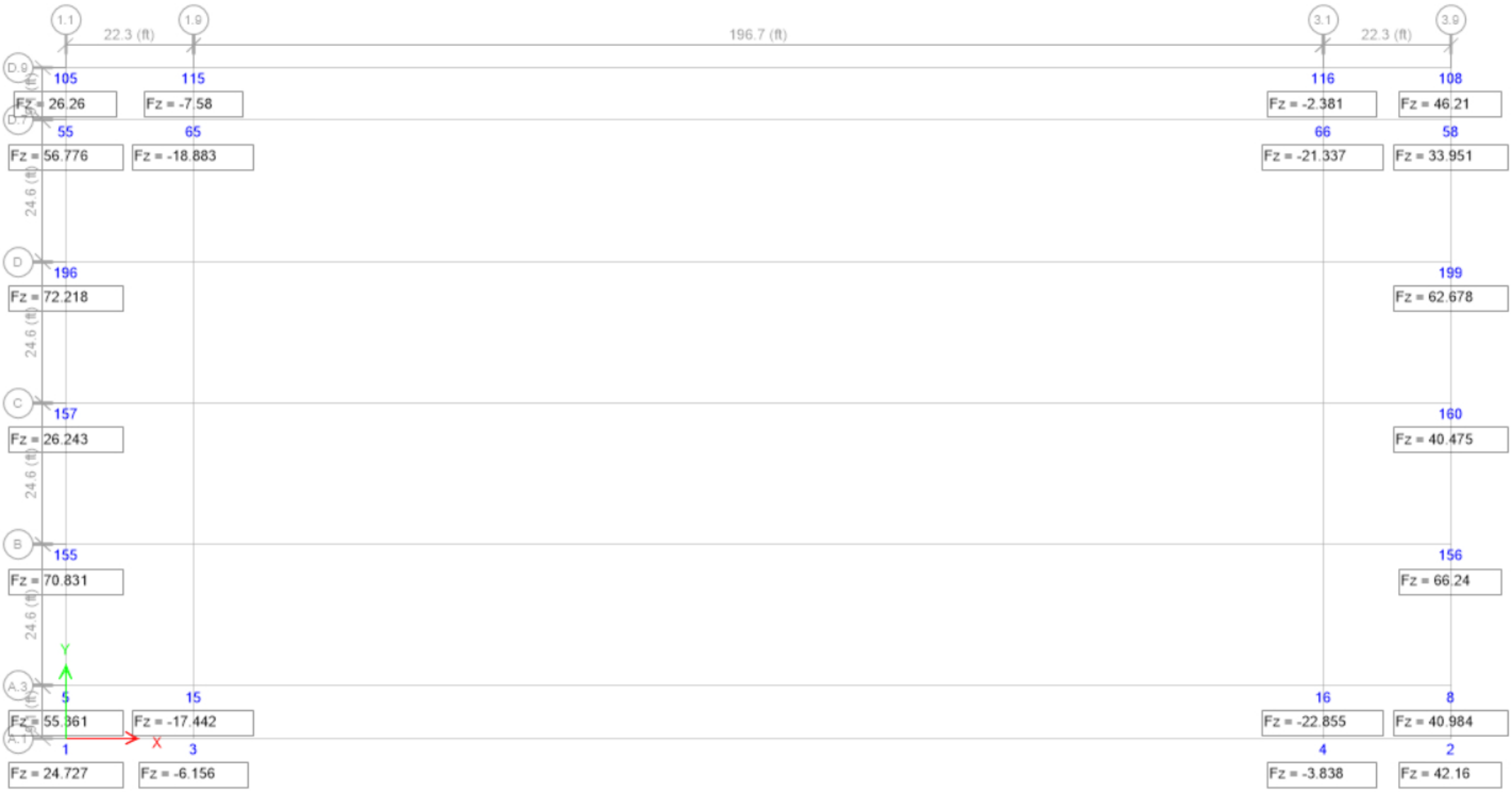
Elevation D.9



Hangar - Base Reactions (Dead)



Hangar - Base Reactions (Live)



Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Slender Wall

Project File: AOF.ec6

LIC# : KW-06019202, Build:20.24.10.03 Michael Baker International (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Exterior Masonry Walls (MW8)

Code References

Calculations per TMS 402-16, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-22 / IBC 2024

General Information

Calculations per TMS 402-16, IBC 2021, ASCE 7-16

Construction Type : Grouted Hollow Concrete Masonry

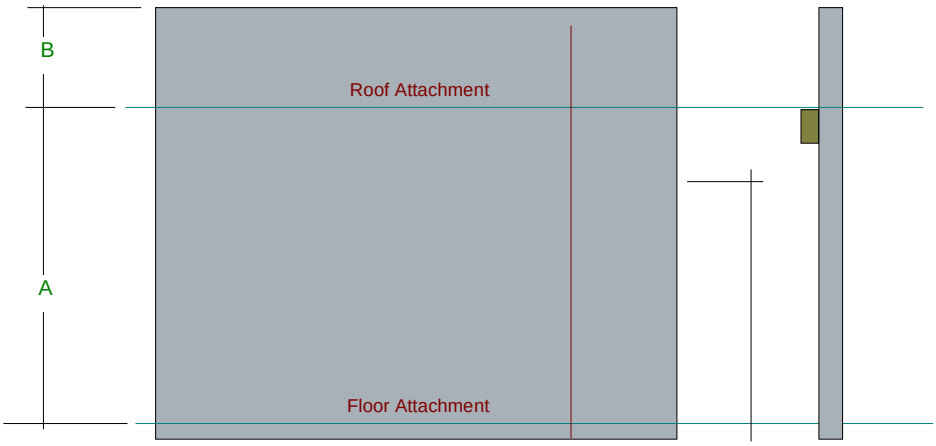
F'm	=	2.0 ksi	Nom. Wall Thickness	8	in	Temp Diff across thickness	=	deg F
Fy - Yield	=	60.0 ksi	Actual Thickness	7.625	in	Min Allow Out-of-plane Defl Ra	=	0.0
Fr - Rupture	=	61.0 psi	Rebar "d" distance	3.8125	in			
Em = f'm *	=	900.0	Lower Level Rebar . . .			Minimum Vertical Steel %	=	0.0020
Max % of ρ bal.	=	0.009228	Bar Size	# 5				
Grout Density	=	140 pcf	Bar Spacing	32	in			
Block Weight		Normal Weight						
Wall Weight	=	51.0 psf						

Wall is grouted at rebar cells only

One-Story Wall Dimensions

A	Clear Height	=	10.0 ft
B	Parapet height	=	ft

Wall Support Condition Top & Bottom Pinned



Vertical Loads

Vertical Uniform Loads . . . (Applied per foot of Strip Width)

			DL : Dead	Lr : Roof Live	Lf : Floor Live	S : Snow	W : Wind
Ledger Load	Eccentricity	in	1.0	1.0			k/ft
Concentric Load							k/ft

Lateral Loads

Wind Loads :

Full area WIND load 16.0 psf

Seismic Loads :

Wall Weight Seismic Load Input Method : ASCE seismic factors entered

SDS Value per ASCE 12.11.1 S_{DS} * I = 0.740

Fp = Wall Wt. * 0.2960 = 15.096 psf

Project Title:
Engineer:
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Project Descr:

Masonry Slender Wall

Project File: AOF.ec6

LIC# : KW-06019202, Build:20.24.10.03 Michael Baker International (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Exterior Masonry Walls (MW8)

DESIGN SUMMARY

Results reported for "Strip Width" of 12.0 in

	Governing Load Combination . . .	Actual Values . . .		Allowable Values . . .	
PASS	Moment Capacity Check +0.90D+W	Maximum Bending Stress Rat	**0.09920**		
		Max Mu	0.2001 k-ft	Phi * Mn	2.017 k-ft
PASS	Service Deflection Check W Only	Actual Defl. Ratio L/ Max. Deflection	20590 0.005828 in	Allowable Defl. Ratio	400.0
PASS	Axial Load Check +1.20D+0.50Lr+W	Max Pu / Ag Location	44.091 psi 4.833 ft	Max. Allow. Defl. 0.2 * f'm	0.30 in 400.0 psi
	Reinforcing Limit Check	Actual As/bd	0.002541	Max Allow As/bd	0.009228

Maximum Reactions *for Load Combination...*

Top Horizontal	W Only	0.080 k
Base Horizontal	W Only	0.080 k
Vertical Reaction	+D+Lr	2.510 k

Design Maximum Combinations - Moments

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Axial Load 0.2*f'm*b*t k	Moment Values Mcr k-ft	Mu k-ft	Phi	Phi Mn k-ft	As in^2	As Ratio	0.6 * rho bal	Bar 'd'
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
	0.000	0.000	0.00	0.00	0.00	0.00	0.000	0.0000	0.0000	0.00
+1.20D+1.60Lr+0.50W at 4.67 to 5.00	3.126	18.384	0.46	0.10	0.90	2.53	0.116	0.0025	0.0084	0.00
+1.20D+1.60Lr-0.50W at 4.67 to 5.00	3.126	18.384	0.46	0.10	0.90	2.53	0.116	0.0025	0.0084	0.00
+1.20D+0.50W at 4.67 to 5.00	1.526	18.384	0.46	0.10	0.90	2.12	0.116	0.0025	0.0090	0.00
+1.20D-0.50W at 4.67 to 5.00	1.526	18.384	0.46	0.10	0.90	2.12	0.116	0.0025	0.0090	0.00
+1.20D+0.50Lr+W at 4.67 to 5.00	2.026	18.384	0.46	0.20	0.90	2.25	0.116	0.0025	0.0088	0.00
+1.20D+0.50Lr-W at 4.67 to 5.00	2.026	18.384	0.46	0.20	0.90	2.25	0.116	0.0025	0.0088	0.00
+1.20D+W at 4.67 to 5.00	1.526	18.384	0.46	0.20	0.90	2.12	0.116	0.0025	0.0090	0.00
+1.20D-W at 4.67 to 5.00	1.526	18.384	0.46	0.20	0.90	2.12	0.116	0.0025	0.0090	0.00
+0.90D+W at 4.67 to 5.00	1.145	18.384	0.46	0.20	0.90	2.02	0.116	0.0025	0.0091	0.00
+0.90D-W at 4.67 to 5.00	1.145	18.384	0.46	0.20	0.90	2.02	0.116	0.0025	0.0091	0.00
+1.318D+E at 4.67 to 5.00	1.676	18.384	0.46	0.19	0.90	2.16	0.116	0.0025	0.0089	0.00
+1.318D-E at 4.67 to 5.00	1.676	18.384	0.46	0.19	0.90	2.16	0.116	0.0025	0.0089	0.00
+0.7820D+E at 4.67 to 5.00	0.995	18.384	0.46	0.19	0.90	1.98	0.116	0.0025	0.0092	0.00
+0.7820D-E at 4.67 to 5.00	0.995	18.384	0.46	0.19	0.90	1.98	0.116	0.0025	0.0092	0.00

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load Pu k	Moment Values Mcr k-ft	Moment Values Mactual k-ft	Stiffness I gross in^4	Stiffness I cracked in^4	Stiffness I effective in^4	Deflections Deflection in	Deflections Defl. Ratio
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
+D+0.60W at 4.67 to 5.00	1.272	0.46	0.12	342.40	21.51	342.400	0.004	34,214.4
+D-0.60W at 4.67 to 5.00	1.272	0.46	0.12	342.40	21.51	342.400	0.004	34,214.4
+D+0.750Lr+0.450W at 4.67 to 5.00	2.022	0.46	0.09	342.40	23.07	342.400	0.003	45,538.2
+D+0.750Lr-0.450W at 4.67 to 5.00	2.022	0.46	0.09	342.40	23.07	342.400	0.003	45,538.2
+D+0.450W at 4.67 to 5.00	1.272	0.46	0.09	342.40	21.51	342.400	0.003	45,619.2
+D-0.450W at 4.67 to 5.00	1.272	0.46	0.09	342.40	21.51	342.400	0.003	45,619.2

Project Title:
Engineer:
Project ID:
Project Descr:

Masonry Slender Wall

Project File: AOF.ec6

LIC# : KW-06019202, Build:20.24.10.03 Michael Baker International (c) ENERCALC, LLC 1982-2025

DESCRIPTION: Exterior Masonry Walls (MW8)

+0.60D+0.60W at 4.67 to 5.00	0.763	0.46	0.12	342.40	20.43	342.400	0.004	34,255.1
+0.60D-0.60W at 4.67 to 5.00	0.763	0.46	0.12	342.40	20.43	342.400	0.004	34,255.1
+D+0.70E at 4.67 to 5.00	1.272	0.46	0.13	342.40	21.51	342.400	0.004	31,082.8
+D-0.70E at 4.67 to 5.00	1.272	0.46	0.13	342.40	21.51	342.400	0.004	31,082.8

Design Maximum Combinations - Deflections

Results reported for "Strip Width" = 12 in.

Load Combination	Axial Load	Moment Values		Stiffness			Deflections	
	Pu	Mcr	Mactual	I gross	I cracked	I effective	Deflection	Defl. Ratio
	k	k-ft	k-ft	in^4	in^4	in^4	in	
+D+0.5250E at 4.67 to 5.00	1.272	0.46	0.10	342.40	21.51	342.400	0.003	41,443.8
+D-0.5250E at 4.67 to 5.00	1.272	0.46	0.10	342.40	21.51	342.400	0.003	41,443.8
+0.60D+0.70E at 4.67 to 5.00	0.763	0.46	0.13	342.40	20.43	342.400	0.004	31,119.8
+0.60D-0.70E at 4.67 to 5.00	0.763	0.46	0.13	342.40	20.43	342.400	0.004	31,119.8
	0.000	0.00	0.00	0.00	0.00	0.000	0.000	0.0
W Only at 5.00 to 5.33	0.000	0.46	0.20	342.40	18.78	342.400	0.006	20,589.7
-W at 5.00 to 5.33	0.000	0.46	0.20	342.40	18.78	342.400	0.006	20,589.7
E Only at 5.00 to 5.33	0.000	0.46	0.19	342.40	18.78	342.400	0.005	21,822.7
E Only * -1.0 at 5.00 to 5.33	0.000	0.46	0.19	342.40	18.78	342.400	0.005	21,822.7

Reactions - Vertical & Horizontal

Load Combination	Base Horizontal	Top Horizontal	Vertical @ Wall Base
D Only	0.0 k	0.00 k	1.510 k
+D+Lr	0.0 k	0.00 k	2.510 k
+D+0.750Lr	0.0 k	0.00 k	2.260 k
+D+0.60W	0.0 k	0.05 k	1.510 k
+D-0.60W	0.0 k	0.05 k	1.510 k
+D+0.750Lr+0.450W	0.0 k	0.04 k	2.260 k
+D+0.750Lr-0.450W	0.0 k	0.04 k	2.260 k
+D+0.450W	0.0 k	0.04 k	1.510 k
+D-0.450W	0.0 k	0.04 k	1.510 k
+0.60D+0.60W	0.0 k	0.05 k	0.906 k
+0.60D-0.60W	0.0 k	0.05 k	0.906 k
+D+0.70E	0.1 k	0.05 k	1.510 k
+D-0.70E	0.1 k	0.05 k	1.510 k
+D+0.5250E	0.0 k	0.04 k	1.510 k
+D-0.5250E	0.0 k	0.04 k	1.510 k
+0.60D+0.70E	0.1 k	0.05 k	0.906 k
+0.60D-0.70E	0.1 k	0.05 k	0.906 k
Lr Only	0.0 k	0.00 k	1.000 k
W Only	0.1 k	0.08 k	0.000 k
-W	0.1 k	0.08 k	0.000 k
E Only	0.1 k	0.08 k	0.000 k
E Only * -1.0	0.1 k	0.08 k	0.000 k

DESIGN MODULE - MASONRY WALL OPENING

Michael Baker INTERNATIONAL

HANGAR-CREECH PH 2

LOADS FROM ROOF/FLOOR

	ROOF	FLOOR
DEAD:	30.0 psf	0.0 psf
LIVE:	20.0 psf	0.0 psf
SNOW/Lr:	0.0 psf	0.0 psf

WALL DESIGN INFORMATION

f'm:	2000 psi
Fy:	60 ksi
Solid Opening:	Y
Interior Wall:	N

OUT OF PLANE LOADS

Wall Type	*Wall Weight (Finishes Only)*
1	45 psf
2	0 psf
3	0 psf
4	0 psf

W:	16 psf
S_{DS}:	0.59

LINTEL ID	WALL TYPE	WALL HEIGHT	WALL THICKNESS	ROOF DECK HEIGHT	ROOF TRIB	FLOOR TRIB	VERTICAL LOADING			# MATS HOR REINF
							DEAD	LIVE	SNOW/Lr	
ML1	1	10.0 ft	8" CMU	10.0 ft	6.0 ft	0.0 ft	180 plf	120 plf	0 plf	1
										1

MASONRY WALL DESIGN

LINTEL ID	WALL GROUT SPACING	OPENING		LINTEL				JAMB COLUMN			STATUS
		HEIGHT	WIDTH	BAR SIZE	DEPTH	BARS/ COURSE	# OF COURSES	BAR SIZE	WIDTH	BARS/ CELL	
ML1	24 o.c.	8.00 ft	3.50 ft	#5	16 in	2	1	#5	16 in	1	OK

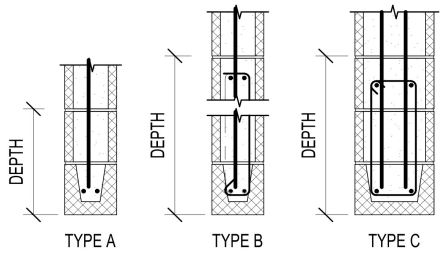
DESIGN MODULE - MASONRY WALL OPENING

HANGAR-CREECH PH 2

Michael Baker INTERNATIONAL

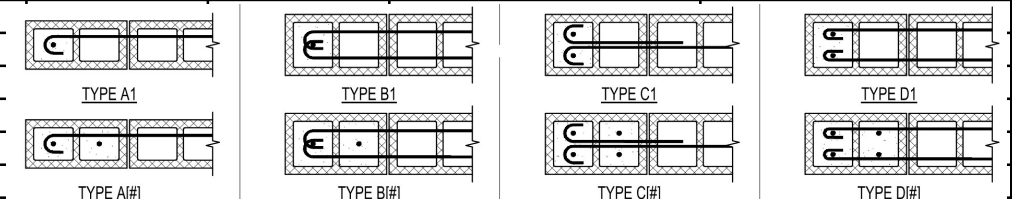
MASONRY LINTEL SCHEDULE

MARK	DEPTH	TYPE	LINTEL REINFORCING		SUPPORT COLUMN	COMMENTS
			HORIZONTAL	STIRRUPS		
ML1	16 "	TYPE A	(2) #5	--	ML1 C	



MASONRY JAMB COLUMN SCHEDULE

MARK	SIZE	TYPE	COLUMN REINFORCING		COMMENTS
			VERTICAL	TIES	
ML1 C	8x16	TYPE A2	(3) #5	--	



Michael Baker
INTERNATIONAL

Lateral System Calculations

Auto Seismic Loading

Load Pattern Definitions - Auto Seismic - ASCE 7-22 (Part 1 of 4)

Name	Period Used sec	Coeff Used	Weight Used kip	Base Shear kip	Is Auto Load	X Dir?	X Dir Plus Ecc?	X Dir Minus Ecc?	Y Dir?	Y Dir Plus Ecc?	Y Dir Minus Ecc?	Ecc Ratio
Exx					No	Yes	Yes	Yes	No	No	No	0.05
Exx(1/3)	0.393	0.122333	1232.4286	150.767	Yes	Yes	No	No	No	No	No	0.05
Exx(2/3)	0.393	0.122333	1232.4286	150.767	Yes	No	Yes	No	No	No	No	0.05
Exx(3/3)	0.393	0.122333	1232.4286	150.767	Yes	No	No	Yes	No	No	No	0.05
Eyy					No	No	No	No	Yes	Yes	Yes	0.05
Eyy(1/3)	0.393	0.122333	1232.4286	150.767	Yes	No	No	No	Yes	No	No	0.05
Eyy(2/3)	0.393	0.122333	1232.4286	150.767	Yes	No	No	No	No	Yes	No	0.05
Eyy(3/3)	0.393	0.122333	1232.4286	150.767	Yes	No	No	No	No	No	Yes	0.05

Load Pattern Definitions - Auto Seismic - ASCE 7-22 (Part 2 of 4)

Name	Period Used sec	Coeff Used	Weight Used kip	Base Shear kip	Top Story	Bottom Story	Period Type	Ct and X	MethodType	RSFunc
Exx					Story33	Base	Approximate	0.02; 0.75	Method 2	
Exx(1/3)	0.393	0.122333	1232.4286	150.767	Story33	Base	Approximate	0.02; 0.75	Method 2	
Exx(2/3)	0.393	0.122333	1232.4286	150.767	Story33	Base	Approximate	0.02; 0.75	Method 2	
Exx(3/3)	0.393	0.122333	1232.4286	150.767	Story33	Base	Approximate	0.02; 0.75	Method 2	
Eyy					Story33	Base	Approximate	0.02; 0.75	Method 2	
Eyy(1/3)	0.393	0.122333	1232.4286	150.767	Story33	Base	Approximate	0.02; 0.75	Method 2	
Eyy(2/3)	0.393	0.122333	1232.4286	150.767	Story33	Base	Approximate	0.02; 0.75	Method 2	
Eyy(3/3)	0.393	0.122333	1232.4286	150.767	Story33	Base	Approximate	0.02; 0.75	Method 2	

Load Pattern Definitions - Auto Seismic - ASCE 7-22 (Part 3 of 4)

Name	Period Used sec	Coeff Used	Weight Used kip	Base Shear kip	Smax	Tmax	Sds	Sd1	TL sec	Site Class	S1	R
Exx							0.5872	0.3281	6	D	0.23	6
Exx(1/3)	0.393	0.122333	1232.4286	150.767			0.5872	0.3281	6	D	0.23	6
Exx(2/3)	0.393	0.122333	1232.4286	150.767			0.5872	0.3281	6	D	0.23	6
Exx(3/3)	0.393	0.122333	1232.4286	150.767			0.5872	0.3281	6	D	0.23	6
Eyy							0.5872	0.3281	6	D	0.23	6
Eyy(1/3)	0.393	0.122333	1232.4286	150.767			0.5872	0.3281	6	D	0.23	6
Eyy(2/3)	0.393	0.122333	1232.4286	150.767			0.5872	0.3281	6	D	0.23	6
Eyy(3/3)	0.393	0.122333	1232.4286	150.767			0.5872	0.3281	6	D	0.23	6

Load Pattern Definitions - Auto Seismic - ASCE 7-22 (Part 4 of 4)

Name	Period Used sec	Coeff Used	Weight Used kip	Base Shear kip	I	Omega	Cd
Exx					1.25	2	5
Exx(1/3)	0.393	0.122333	1232.4286	150.767	1.25	2	5
Exx(2/3)	0.393	0.122333	1232.4286	150.767	1.25	2	5
Exx(3/3)	0.393	0.122333	1232.4286	150.767	1.25	2	5
Eyy					1.25	2	5
Eyy(1/3)	0.393	0.122333	1232.4286	150.767	1.25	2	5
Eyy(2/3)	0.393	0.122333	1232.4286	150.767	1.25	2	5
Eyy(3/3)	0.393	0.122333	1232.4286	150.767	1.25	2	5

ASCE 7-22 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Exx according to ASCE 7-22, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Approximate

Coefficient, C_t [ASCE Table 12.8-2]		
Coefficient, x [ASCE Table 12.8-2]		$C_{t=0.02ft}$
Structure Height Above Base, h_n		
Approximate Fundamental Period, T_a [ASCE 12.8.2.1 Eq. 12.8-8]	Ta=<Ct><hn>^{x}	$T_{a=0.393}$ sec
Long-Period Transition Period, T_L [ASCE 11.4.6]		$T_{L=6}$ sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	R=6
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	
Importance Factor, I [ASCE Table 1.5-2]	

Sds and Sd1 Source

Mapped MCE Spectral Response Acceleration, S_{ds} [ASCE 11.4.4]	
Mapped MCE Spectral Response Acceleration, S_{d1} [ASCE 11.4.4]	$S_{ds=0.5872g}$
Site Class [ASCE Table 20.2-1] = D - Medium dense sand or stiff clay	

Seismic Response

Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.5, 11.4.8]	
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.5, 11.4.6]	S_{DS} =0.5872g
Design Spectral Response Acceleration, S 1 [ASCE 11.4.5.1]	

Equivalent Lateral Forces

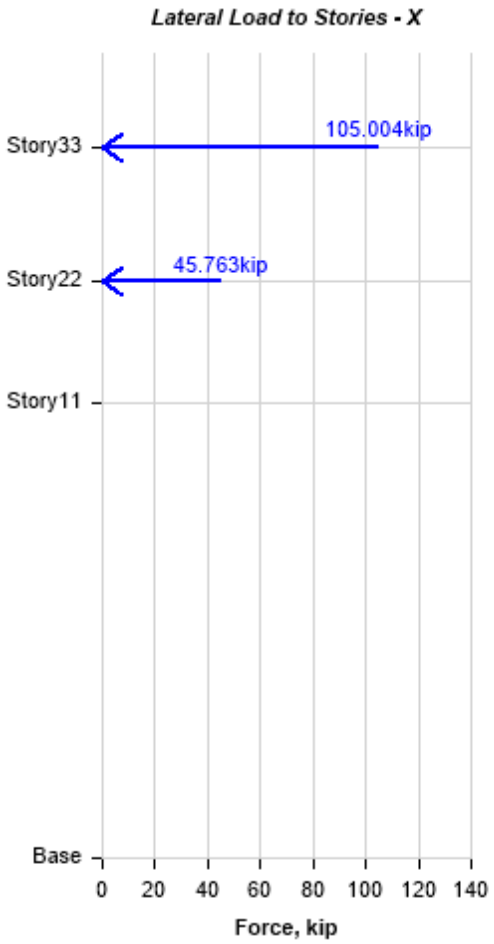
Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-3]	$C_S = S_{DS} / (R/I)$
[ASCE 12.8.1.1, Eq. 12.8-4]	$C_{S,max} = S_{D1} / T(R/I)$
[ASCE 12.8.1.1, Eq. 12.8-6]	$C_{S,min} = \max(0.044\ S_{DS} I, 0.01) = 0.032296$
[ASCE 12.8.1.1, Eq. 12.8-7]	$C_{S,min} = 0.5\ S_1 /(R/I)$ for $S_1 = 0.6g$
	$C_{S,min} \le C_s \le C_{S,max}$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kip)	V (kip)
X	0.393	0.122333	1232.4286	150.7671
X + Ecc. Y	0.393	0.122333	1232.4286	150.7671
X - Ecc. Y	0.393	0.122333	1232.4286	150.7671

Applied Story Forces

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story33	53	105.004	0
Story22	43	45.763	0
Story11	34	0	0
Base	0	0	0


Lateral Load to Stories - X
Story33
105.004kip
Story22
45.763kip
Story11
Base
0
20
40
60
80
100
120
140
Force, kip

Set 1

ASCE 7-22 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Eyy according to ASCE 7-22, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = Approximate

Coefficient, C_t [ASCE Table 12.8-2]		
Coefficient, x [ASCE Table 12.8-2]		$C_{t=0.02ft}$
Structure Height Above Base, h_n		
Approximate Fundamental Period, T_a [ASCE 12.8.2.1 Eq. 12.8-8]	Ta=<Ct><hn>^{x}	$T_{a=0.393}$ sec
Long-Period Transition Period, T_L [ASCE 11.4.6]		$T_{L=6}$ sec

Factors and Coefficients

Response Modification Factor, R [ASCE Table 12.2-1]	
System Overstrength Factor, Ω_0 [ASCE Table 12.2-1]	R=6
Deflection Amplification Factor, C_d [ASCE Table 12.2-1]	
Importance Factor, I [ASCE Table 1.5-2]	

Sds and Sd1 Source

Mapped MCE Spectral Response Acceleration, S_{ds} [ASCE 11.4.4]	
Mapped MCE Spectral Response Acceleration, S_{d1} [ASCE 11.4.4]	$S_{ds=0.5872g}$
Site Class [ASCE Table 20.2-1] = D - Medium dense sand or stiff clay	

Seismic Response

Design Spectral Response Acceleration, S_{DS} [ASCE 11.4.5, 11.4.8]	
Design Spectral Response Acceleration, S_{D1} [ASCE 11.4.5, 11.4.6]	S_{DS} =0.5872g
Design Spectral Response Acceleration, S 1 [ASCE 11.4.5.1]	

Equivalent Lateral Forces

Seismic Response Coefficient, C_S [ASCE 12.8.1.1, Eq. 12.8-3]	$C_S = S_{DS} / (R/I)$
[ASCE 12.8.1.1, Eq. 12.8-4]	$C_{S,max} = S_{D1} / T(R/I)$
[ASCE 12.8.1.1, Eq. 12.8-6]	$C_{S,min} = \max(0.044\ S_{DS} I, 0.01) = 0.032296$
[ASCE 12.8.1.1, Eq. 12.8-7]	$C_{S,min} = 0.5\ S_1 /(R/I)$ for $S_1 = 0.6g$
	$C_{S,min} \leq C_s \leq C_{S,max}$

Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kip)	V (kip)
Y	0.393	0.122333	1232.4286	150.7671
Y + Ecc. X	0.393	0.122333	1232.4286	150.7671
Y - Ecc. X	0.393	0.122333	1232.4286	150.7671

Applied Story Forces

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story33	53	0	105.004
Story22	43	0	45.763
Story11	34	0	0
Base	0	0	0

Lateral Load to Stories - Y
Story33
105.004kip
Story22
45.763kip
Story11
Base
0
20
40
60
80
100
120
140
Force, kip

Set 1

Structure Results

Base Reactions

Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Dead	LinStatic		0	0	1394.444	81222.3969	-167974.4189	-0.1083
Live	LinStatic		0	0	564.641	32919.8283	-68123.7611	-0.0556
Exx	LinStatic	Max	-150.767	0	0	0.0662	-7300.8034	8789.7183
Exx	LinStatic	Min	-150.767	0	0	0.0662	-7300.8034	8789.7183
Eyy	LinStatic	Max	0	-150.767	0	7318.6474	-0.0432	-18190.0017
Eyy	LinStatic	Min	0	-150.767	0	7318.6474	-0.0432	-18190.0017
Mass	LinStatic		0	0	1232.429	71853.3026	-148692.088	-0.1207
Wx	LinStatic		-184.41	0	-408.28	-23803.3973	44100.733	10751.1311
Wy	LinStatic		0	-374.555	-408.28	-12529.0104	49258.5665	-45189.9363
Wxx	LinStatic		0	0	0	0.0144	0.0086	-0.0004
Wyy	LinStatic		0	0	0	-0.0019	-0.0301	-0.0004
Sb	LinStatic		0	0	594.423	34656.1642	-71716.9102	-0.0584
Su1	LinStatic		0	0	361.482	21075.3422	-54561.5371	-0.0408
Su2	LinStatic		0	0	361.482	21075.044	-32663.7205	-0.0319

Applied Loads

Line Loads

Frame Loads Assignments - Distributed (Part 1 of 2)

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
343	Dead	Story33	B26	Force	Gravity	Relative	0	0.3333	0	7.4588
343	Dead	Story33	B26	Force	Gravity	Relative	0.3333	0.6667	7.4588	14.9176
343	Dead	Story33	B26	Force	Gravity	Relative	0.6667	1	14.9176	22.3765
66	Dead	Story33	B28	Force	Gravity	Relative	0	1	0	22.3765
362	Dead	Story33	B45	Force	Gravity	Relative	0	0.3333	0	7.4588
362	Dead	Story33	B45	Force	Gravity	Relative	0.3333	0.6667	7.4588	14.9176
362	Dead	Story33	B45	Force	Gravity	Relative	0.6667	1	14.9176	22.3765
149	Dead	Story33	B47	Force	Gravity	Relative	0	1	0	22.3765
230	Dead	Story33	B69	Force	Gravity	Relative	0	1	0	22.3765
232	Dead	Story33	B71	Force	Gravity	Relative	0	1	0	22.3765
313	Dead	Story33	B117	Force	Gravity	Relative	0	1	0	22.3765
315	Dead	Story33	B119	Force	Gravity	Relative	0	1	0	22.3765
429	Dead	Story33	B29	Force	Gravity	Relative	0	0.7778	0	76.7567
429	Dead	Story33	B29	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
431	Dead	Story33	B46	Force	Gravity	Relative	0	0.7778	0	76.7567
431	Dead	Story33	B46	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
1	Dead	Story33	B48	Force	Gravity	Relative	0	0.7778	0	76.7567
1	Dead	Story33	B48	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
84	Dead	Story33	B70	Force	Gravity	Relative	0	0.7778	0	76.7567
84	Dead	Story33	B70	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
148	Dead	Story33	B72	Force	Gravity	Relative	0	0.7778	0	76.7567
148	Dead	Story33	B72	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
167	Dead	Story33	B118	Force	Gravity	Relative	0	0.7778	0	76.7567
167	Dead	Story33	B118	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
233	Dead	Story33	B120	Force	Gravity	Relative	0	0.7778	0	76.7567
233	Dead	Story33	B120	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
314	Dead	Story33	B124	Force	Gravity	Relative	0	0.7778	0	76.7567
314	Dead	Story33	B124	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
416	Dead	Story33	B125	Force	Gravity	Relative	0	0.8189	0	99.1332
416	Dead	Story33	B125	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
418	Dead	Story33	B192	Force	Gravity	Relative	0	0.8189	0	99.1332
418	Dead	Story33	B192	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
420	Dead	Story33	B193	Force	Gravity	Relative	0	0.8189	0	99.1332
420	Dead	Story33	B193	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
423	Dead	Story33	B216	Force	Gravity	Relative	0	0.8189	0	99.1332
423	Dead	Story33	B216	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
425	Dead	Story33	B217	Force	Gravity	Relative	0	0.8189	0	99.1332
425	Dead	Story33	B217	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
427	Dead	Story33	B218	Force	Gravity	Relative	0	0.8189	0	99.1332
427	Dead	Story33	B218	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
343	Live	Story33	B26	Force	Gravity	Relative	0	0.3333	0	7.4588
343	Live	Story33	B26	Force	Gravity	Relative	0.3333	0.6667	7.4588	14.9176
343	Live	Story33	B26	Force	Gravity	Relative	0.6667	1	14.9176	22.3765
66	Live	Story33	B28	Force	Gravity	Relative	0	1	0	22.3765
362	Live	Story33	B45	Force	Gravity	Relative	0	0.3333	0	7.4588
362	Live	Story33	B45	Force	Gravity	Relative	0.3333	0.6667	7.4588	14.9176
362	Live	Story33	B45	Force	Gravity	Relative	0.6667	1	14.9176	22.3765
149	Live	Story33	B47	Force	Gravity	Relative	0	1	0	22.3765
230	Live	Story33	B69	Force	Gravity	Relative	0	1	0	22.3765

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
232	Live	Story33	B71	Force	Gravity	Relative	0	1	0	22.3765
313	Live	Story33	B117	Force	Gravity	Relative	0	1	0	22.3765
315	Live	Story33	B119	Force	Gravity	Relative	0	1	0	22.3765
429	Live	Story33	B29	Force	Gravity	Relative	0	0.7778	0	76.7567
429	Live	Story33	B29	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
431	Live	Story33	B46	Force	Gravity	Relative	0	0.7778	0	76.7567
431	Live	Story33	B46	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
1	Live	Story33	B48	Force	Gravity	Relative	0	0.7778	0	76.7567
1	Live	Story33	B48	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
84	Live	Story33	B70	Force	Gravity	Relative	0	0.7778	0	76.7567
84	Live	Story33	B70	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
148	Live	Story33	B72	Force	Gravity	Relative	0	0.7778	0	76.7567
148	Live	Story33	B72	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
167	Live	Story33	B118	Force	Gravity	Relative	0	0.7778	0	76.7567
167	Live	Story33	B118	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
233	Live	Story33	B120	Force	Gravity	Relative	0	0.7778	0	76.7567
233	Live	Story33	B120	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
314	Live	Story33	B124	Force	Gravity	Relative	0	0.7778	0	76.7567
314	Live	Story33	B124	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
416	Live	Story33	B125	Force	Gravity	Relative	0	0.8189	0	99.1332
416	Live	Story33	B125	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
418	Live	Story33	B192	Force	Gravity	Relative	0	0.8189	0	99.1332
418	Live	Story33	B192	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
420	Live	Story33	B193	Force	Gravity	Relative	0	0.8189	0	99.1332
420	Live	Story33	B193	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
423	Live	Story33	B216	Force	Gravity	Relative	0	0.8189	0	99.1332
423	Live	Story33	B216	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
425	Live	Story33	B217	Force	Gravity	Relative	0	0.8189	0	99.1332
425	Live	Story33	B217	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
427	Live	Story33	B218	Force	Gravity	Relative	0	0.8189	0	99.1332
427	Live	Story33	B218	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
343	Mass	Story33	B26	Force	Gravity	Relative	0	0.3333	0	7.4588
343	Mass	Story33	B26	Force	Gravity	Relative	0.3333	0.6667	7.4588	14.9176
343	Mass	Story33	B26	Force	Gravity	Relative	0.6667	1	14.9176	22.3765
66	Mass	Story33	B28	Force	Gravity	Relative	0	1	0	22.3765
362	Mass	Story33	B45	Force	Gravity	Relative	0	0.3333	0	7.4588
362	Mass	Story33	B45	Force	Gravity	Relative	0.3333	0.6667	7.4588	14.9176
362	Mass	Story33	B45	Force	Gravity	Relative	0.6667	1	14.9176	22.3765
149	Mass	Story33	B47	Force	Gravity	Relative	0	1	0	22.3765
230	Mass	Story33	B69	Force	Gravity	Relative	0	1	0	22.3765
232	Mass	Story33	B71	Force	Gravity	Relative	0	1	0	22.3765
313	Mass	Story33	B117	Force	Gravity	Relative	0	1	0	22.3765
315	Mass	Story33	B119	Force	Gravity	Relative	0	1	0	22.3765
429	Mass	Story33	B29	Force	Gravity	Relative	0	0.7778	0	76.7567
429	Mass	Story33	B29	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
431	Mass	Story33	B46	Force	Gravity	Relative	0	0.7778	0	76.7567
431	Mass	Story33	B46	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
1	Mass	Story33	B48	Force	Gravity	Relative	0	0.7778	0	76.7567
1	Mass	Story33	B48	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
84	Mass	Story33	B70	Force	Gravity	Relative	0	0.7778	0	76.7567
84	Mass	Story33	B70	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
148	Mass	Story33	B72	Force	Gravity	Relative	0	0.7778	0	76.7567
148	Mass	Story33	B72	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
167	Mass	Story33	B118	Force	Gravity	Relative	0	0.7778	0	76.7567

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
167	Mass	Story33	B118	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
233	Mass	Story33	B120	Force	Gravity	Relative	0	0.7778	0	76.7567
233	Mass	Story33	B120	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
314	Mass	Story33	B124	Force	Gravity	Relative	0	0.7778	0	76.7567
314	Mass	Story33	B124	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
416	Mass	Story33	B125	Force	Gravity	Relative	0	0.8189	0	99.1332
416	Mass	Story33	B125	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
418	Mass	Story33	B192	Force	Gravity	Relative	0	0.8189	0	99.1332
418	Mass	Story33	B192	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
420	Mass	Story33	B193	Force	Gravity	Relative	0	0.8189	0	99.1332
420	Mass	Story33	B193	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
423	Mass	Story33	B216	Force	Gravity	Relative	0	0.8189	0	99.1332
423	Mass	Story33	B216	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
425	Mass	Story33	B217	Force	Gravity	Relative	0	0.8189	0	99.1332
425	Mass	Story33	B217	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
427	Mass	Story33	B218	Force	Gravity	Relative	0	0.8189	0	99.1332
427	Mass	Story33	B218	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
343	Wx	Story33	B26	Force	Global-Y	Relative	0	0.3333	0	7.4588
343	Wx	Story33	B26	Force	Global-Y	Relative	0.3333	0.6667	7.4588	14.9176
343	Wx	Story33	B26	Force	Global-Y	Relative	0.6667	1	14.9176	22.3765
66	Wx	Story33	B28	Force	Global-Y	Relative	0	1	0	22.3765
362	Wx	Story33	B45	Force	Local-2	Relative	0	0.3333	0	7.4588
362	Wx	Story33	B45	Force	Local-2	Relative	0.3333	0.6667	7.4588	14.9176
362	Wx	Story33	B45	Force	Local-2	Relative	0.6667	1	14.9176	22.3765
149	Wx	Story33	B47	Force	Local-2	Relative	0	1	0	22.3765
230	Wx	Story33	B69	Force	Local-2	Relative	0	1	0	22.3765
232	Wx	Story33	B71	Force	Local-2	Relative	0	1	0	22.3765
313	Wx	Story33	B117	Force	Global-Y	Relative	0	1	0	22.3765
315	Wx	Story33	B119	Force	Global-Y	Relative	0	1	0	22.3765
429	Wx	Story33	B29	Force	Global-Y	Relative	0	0.7778	0	76.7567
429	Wx	Story33	B29	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
431	Wx	Story33	B46	Force	Global-Y	Relative	0	0.7778	0	76.7567
431	Wx	Story33	B46	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
1	Wx	Story33	B48	Force	Local-2	Relative	0	0.7778	0	76.7567
1	Wx	Story33	B48	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
84	Wx	Story33	B70	Force	Local-2	Relative	0	0.7778	0	76.7567
84	Wx	Story33	B70	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
148	Wx	Story33	B72	Force	Local-2	Relative	0	0.7778	0	76.7567
148	Wx	Story33	B72	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
167	Wx	Story33	B118	Force	Local-2	Relative	0	0.7778	0	76.7567
167	Wx	Story33	B118	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
233	Wx	Story33	B120	Force	Global-Y	Relative	0	0.7778	0	76.7567
233	Wx	Story33	B120	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
314	Wx	Story33	B124	Force	Global-Y	Relative	0	0.7778	0	76.7567
314	Wx	Story33	B124	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
416	Wx	Story33	B125	Force	Local-2	Relative	0	0.8189	0	99.1332
416	Wx	Story33	B125	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
418	Wx	Story33	B192	Force	Local-2	Relative	0	0.8189	0	99.1332
418	Wx	Story33	B192	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
420	Wx	Story33	B193	Force	Local-2	Relative	0	0.8189	0	99.1332
420	Wx	Story33	B193	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
423	Wx	Story33	B216	Force	Local-2	Relative	0	0.8189	0	99.1332
423	Wx	Story33	B216	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
425	Wx	Story33	B217	Force	Local-2	Relative	0	0.8189	0	99.1332

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
425	Wx	Story33	B217	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
427	Wx	Story33	B218	Force	Local-2	Relative	0	0.8189	0	99.1332
427	Wx	Story33	B218	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
9	Wx	Story33	C9	Force	Global-Y	Relative	0	1	0	10.8483
11	Wx	Story33	C10	Force	Global-Y	Relative	0	1	0	10.8483
258	Wx	Story33	C97	Force	Global-Y	Relative	0	1	0	10.8483
260	Wx	Story33	C99	Force	Global-Y	Relative	0	1	0	10.8483
552	Wx	Story22	B73	Force	Global-X	Relative	0	1	0	9.1
553	Wx	Story22	B74	Force	Global-X	Relative	0	1	0	24.6
554	Wx	Story22	B75	Force	Global-X	Relative	0	1	0	24.6
555	Wx	Story22	B76	Force	Global-X	Relative	0	1	0	24.6
556	Wx	Story22	B77	Force	Global-X	Relative	0	1	0	24.6
557	Wx	Story22	B78	Force	Global-X	Relative	0	1	0	9.1
600	Wx	Story22	B80	Force	Global-X	Relative	0	1	0	9.1
601	Wx	Story22	B81	Force	Global-X	Relative	0	1	0	24.6
602	Wx	Story22	B82	Force	Global-X	Relative	0	1	0	24.6
603	Wx	Story22	B83	Force	Global-X	Relative	0	1	0	24.6
604	Wx	Story22	B84	Force	Global-X	Relative	0	1	0	24.6
605	Wx	Story22	B85	Force	Global-X	Relative	0	1	0	9.1
2	Wx	Story22	C1	Force	Global-X	Relative	0	1	0	9
4	Wx	Story22	C2	Force	Global-X	Relative	0	1	0	9
85	Wx	Story22	C3	Force	Global-X	Relative	0	1	0	9
87	Wx	Story22	C6	Force	Global-X	Relative	0	1	0	9
168	Wx	Story22	C47	Force	Global-X	Relative	0	1	0	9
170	Wx	Story22	C48	Force	Global-X	Relative	0	1	0	9
251	Wx	Story22	C93	Force	Global-X	Relative	0	1	0	9
253	Wx	Story22	C94	Force	Global-X	Relative	0	1	0	9
334	Wx	Story22	C116	Force	Global-X	Relative	0	1	0	9
336	Wx	Story22	C117	Force	Global-X	Relative	0	1	0	9
405	Wx	Story22	C170	Force	Global-X	Relative	0	1	0	9
407	Wx	Story22	C171	Force	Global-X	Relative	0	1	0	9
476	Wx	Story22	C193	Force	Global-X	Relative	0	1	0	9
478	Wx	Story22	C194	Force	Global-X	Relative	0	1	0	9
178	Wx	Story11	B1	Force	Global-Y	Relative	0	0.3333	0	7.4333
178	Wx	Story11	B1	Force	Global-Y	Relative	0.3333	0.6667	7.4333	14.8667
178	Wx	Story11	B1	Force	Global-Y	Relative	0.6667	1	14.8667	22.3
6	Wx	Story11	B3	Force	Global-Y	Relative	0	1	0	22.3
254	Wx	Story11	B97	Force	Global-Y	Relative	0	1	0	22.3
255	Wx	Story11	B98	Force	Global-Y	Relative	0	1	0	22.3
546	Wx	Story11	B73	Force	Global-X	Relative	0	1	0	9.1
547	Wx	Story11	B74	Force	Global-X	Relative	0	1	0	24.6
548	Wx	Story11	B75	Force	Global-X	Relative	0	1	0	24.6
549	Wx	Story11	B76	Force	Global-X	Relative	0	1	0	24.6
550	Wx	Story11	B77	Force	Global-X	Relative	0	1	0	24.6
551	Wx	Story11	B78	Force	Global-X	Relative	0	1	0	9.1
594	Wx	Story11	B80	Force	Global-X	Relative	0	1	0	9.1
595	Wx	Story11	B81	Force	Global-X	Relative	0	1	0	24.6
596	Wx	Story11	B82	Force	Global-X	Relative	0	1	0	24.6
597	Wx	Story11	B83	Force	Global-X	Relative	0	1	0	24.6
598	Wx	Story11	B84	Force	Global-X	Relative	0	1	0	24.6
599	Wx	Story11	B85	Force	Global-X	Relative	0	1	0	9.1
13	Wx	Story11	B5	Force	Global-Y	Relative	0	1	0	10.9278
14	Wx	Story11	B6	Force	Global-Y	Relative	0	1	0	10.9278
15	Wx	Story11	B7	Force	Global-Y	Relative	0	1	0	10.9278

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
16	Wx	Story11	B8	Force	Global-Y	Relative	0	1	0	10.9278
17	Wx	Story11	B9	Force	Global-Y	Relative	0	1	0	10.9278
18	Wx	Story11	B10	Force	Global-Y	Relative	0	1	0	10.9278
19	Wx	Story11	B11	Force	Global-Y	Relative	0	1	0	10.9278
20	Wx	Story11	B12	Force	Global-Y	Relative	0	1	0	10.9278
21	Wx	Story11	B13	Force	Global-Y	Relative	0	1	0	10.9278
22	Wx	Story11	B14	Force	Global-Y	Relative	0	1	0	10.9278
23	Wx	Story11	B15	Force	Global-Y	Relative	0	1	0	10.9278
24	Wx	Story11	B16	Force	Global-Y	Relative	0	1	0	10.9278
25	Wx	Story11	B17	Force	Global-Y	Relative	0	1	0	10.9278
26	Wx	Story11	B18	Force	Global-Y	Relative	0	1	0	10.9278
27	Wx	Story11	B19	Force	Global-Y	Relative	0	1	0	10.9278
28	Wx	Story11	B20	Force	Global-Y	Relative	0	1	0	10.9278
29	Wx	Story11	B21	Force	Global-Y	Relative	0	1	0	10.9278
96	Wx	Story11	B22	Force	Global-Y	Relative	0	1	0	10.9278
263	Wx	Story11	B99	Force	Global-Y	Relative	0	1	0	10.9278
264	Wx	Story11	B100	Force	Global-Y	Relative	0	1	0	10.9278
265	Wx	Story11	B101	Force	Global-Y	Relative	0	1	0	10.9278
266	Wx	Story11	B102	Force	Global-Y	Relative	0	1	0	10.9278
267	Wx	Story11	B103	Force	Global-Y	Relative	0	1	0	10.9278
268	Wx	Story11	B104	Force	Global-Y	Relative	0	1	0	10.9278
269	Wx	Story11	B105	Force	Global-Y	Relative	0	1	0	10.9278
270	Wx	Story11	B106	Force	Global-Y	Relative	0	1	0	10.9278
271	Wx	Story11	B107	Force	Global-Y	Relative	0	1	0	10.9278
272	Wx	Story11	B108	Force	Global-Y	Relative	0	1	0	10.9278
273	Wx	Story11	B109	Force	Global-Y	Relative	0	1	0	10.9278
274	Wx	Story11	B110	Force	Global-Y	Relative	0	1	0	10.9278
275	Wx	Story11	B111	Force	Global-Y	Relative	0	1	0	10.9278
276	Wx	Story11	B112	Force	Global-Y	Relative	0	1	0	10.9278
277	Wx	Story11	B113	Force	Global-Y	Relative	0	1	0	10.9278
278	Wx	Story11	B114	Force	Global-Y	Relative	0	1	0	10.9278
341	Wx	Story11	B115	Force	Global-Y	Relative	0	1	0	10.9278
342	Wx	Story11	B116	Force	Global-Y	Relative	0	1	0	10.9278
5	Wx	Story11	C1	Force	Global-X	Relative	0	0.3333	0	11.3333
5	Wx	Story11	C1	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
5	Wx	Story11	C1	Force	Global-X	Relative	0.6667	1	22.6667	34
3	Wx	Story11	C2	Force	Global-X	Relative	0	1	0	34
8	Wx	Story11	C5	Force	Global-Y	Relative	0	1	0	34
10	Wx	Story11	C7	Force	Global-Y	Relative	0	1	0	34
147	Wx	Story11	C3	Force	Global-X	Relative	0	0.3333	0	11.3333
147	Wx	Story11	C3	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
147	Wx	Story11	C3	Force	Global-X	Relative	0.6667	1	22.6667	34
86	Wx	Story11	C6	Force	Global-X	Relative	0	1	0	34
344	Wx	Story11	C47	Force	Global-X	Relative	0	0.3333	0	11.3333
344	Wx	Story11	C47	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
344	Wx	Story11	C47	Force	Global-X	Relative	0.6667	1	22.6667	34
169	Wx	Story11	C48	Force	Global-X	Relative	0	1	0	34
361	Wx	Story11	C93	Force	Global-X	Relative	0	0.3333	0	11.3333
361	Wx	Story11	C93	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
361	Wx	Story11	C93	Force	Global-X	Relative	0.6667	1	22.6667	34
252	Wx	Story11	C94	Force	Global-X	Relative	0	1	0	34
257	Wx	Story11	C96	Force	Global-Y	Relative	0	1	0	34
259	Wx	Story11	C98	Force	Global-Y	Relative	0	1	0	34
333	Wx	Story11	C116	Force	Global-X	Relative	0	1	0	34

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
335	Wx	Story11	C117	Force	Global-X	Relative	0	1	0	34
404	Wx	Story11	C170	Force	Global-X	Relative	0	1	0	34
406	Wx	Story11	C171	Force	Global-X	Relative	0	1	0	34
475	Wx	Story11	C193	Force	Global-X	Relative	0	1	0	34
477	Wx	Story11	C194	Force	Global-X	Relative	0	1	0	34
343	Wy	Story33	B26	Force	Global-Y	Relative	0	0.3333	0	7.4588
343	Wy	Story33	B26	Force	Global-Y	Relative	0.3333	0.6667	7.4588	14.9176
343	Wy	Story33	B26	Force	Global-Y	Relative	0.6667	1	14.9176	22.3765
66	Wy	Story33	B28	Force	Global-Y	Relative	0	1	0	22.3765
362	Wy	Story33	B45	Force	Local-2	Relative	0	0.3333	0	7.4588
362	Wy	Story33	B45	Force	Local-2	Relative	0.3333	0.6667	7.4588	14.9176
362	Wy	Story33	B45	Force	Local-2	Relative	0.6667	1	14.9176	22.3765
149	Wy	Story33	B47	Force	Local-2	Relative	0	1	0	22.3765
230	Wy	Story33	B69	Force	Local-2	Relative	0	1	0	22.3765
232	Wy	Story33	B71	Force	Local-2	Relative	0	1	0	22.3765
313	Wy	Story33	B117	Force	Global-Y	Relative	0	1	0	22.3765
315	Wy	Story33	B119	Force	Global-Y	Relative	0	1	0	22.3765
429	Wy	Story33	B29	Force	Global-Y	Relative	0	0.7778	0	76.7567
429	Wy	Story33	B29	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
431	Wy	Story33	B46	Force	Global-Y	Relative	0	0.7778	0	76.7567
431	Wy	Story33	B46	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
1	Wy	Story33	B48	Force	Local-2	Relative	0	0.7778	0	76.7567
1	Wy	Story33	B48	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
84	Wy	Story33	B70	Force	Local-2	Relative	0	0.7778	0	76.7567
84	Wy	Story33	B70	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
148	Wy	Story33	B72	Force	Local-2	Relative	0	0.7778	0	76.7567
148	Wy	Story33	B72	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
167	Wy	Story33	B118	Force	Local-2	Relative	0	0.7778	0	76.7567
167	Wy	Story33	B118	Force	Local-2	Relative	0.7778	1	76.7567	98.6872
233	Wy	Story33	B120	Force	Global-Y	Relative	0	0.7778	0	76.7567
233	Wy	Story33	B120	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
314	Wy	Story33	B124	Force	Global-Y	Relative	0	0.7778	0	76.7567
314	Wy	Story33	B124	Force	Global-Y	Relative	0.7778	1	76.7567	98.6872
416	Wy	Story33	B125	Force	Local-2	Relative	0	0.8189	0	99.1332
416	Wy	Story33	B125	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
418	Wy	Story33	B192	Force	Local-2	Relative	0	0.8189	0	99.1332
418	Wy	Story33	B192	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
420	Wy	Story33	B193	Force	Local-2	Relative	0	0.8189	0	99.1332
420	Wy	Story33	B193	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
423	Wy	Story33	B216	Force	Local-2	Relative	0	0.8189	0	99.1332
423	Wy	Story33	B216	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
425	Wy	Story33	B217	Force	Local-2	Relative	0	0.8189	0	99.1332
425	Wy	Story33	B217	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
427	Wy	Story33	B218	Force	Local-2	Relative	0	0.8189	0	99.1332
427	Wy	Story33	B218	Force	Local-2	Relative	0.8189	1	99.1332	121.0637
9	Wy	Story33	C9	Force	Global-Y	Relative	0	1	0	10.8483
11	Wy	Story33	C10	Force	Global-Y	Relative	0	1	0	10.8483
258	Wy	Story33	C97	Force	Global-Y	Relative	0	1	0	10.8483
260	Wy	Story33	C99	Force	Global-Y	Relative	0	1	0	10.8483
552	Wy	Story22	B73	Force	Global-X	Relative	0	1	0	9.1
553	Wy	Story22	B74	Force	Global-X	Relative	0	1	0	24.6
554	Wy	Story22	B75	Force	Global-X	Relative	0	1	0	24.6
555	Wy	Story22	B76	Force	Global-X	Relative	0	1	0	24.6
556	Wy	Story22	B77	Force	Global-X	Relative	0	1	0	24.6

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
557	Wy	Story22	B78	Force	Global-X	Relative	0	1	0	9.1
600	Wy	Story22	B80	Force	Global-X	Relative	0	1	0	9.1
601	Wy	Story22	B81	Force	Global-X	Relative	0	1	0	24.6
602	Wy	Story22	B82	Force	Global-X	Relative	0	1	0	24.6
603	Wy	Story22	B83	Force	Global-X	Relative	0	1	0	24.6
604	Wy	Story22	B84	Force	Global-X	Relative	0	1	0	24.6
605	Wy	Story22	B85	Force	Global-X	Relative	0	1	0	9.1
2	Wy	Story22	C1	Force	Global-Y	Relative	0	1	0	9
4	Wy	Story22	C2	Force	Global-Y	Relative	0	1	0	9
85	Wy	Story22	C3	Force	Global-X	Relative	0	1	0	9
87	Wy	Story22	C6	Force	Global-X	Relative	0	1	0	9
168	Wy	Story22	C47	Force	Global-X	Relative	0	1	0	9
170	Wy	Story22	C48	Force	Global-X	Relative	0	1	0	9
251	Wy	Story22	C93	Force	Global-Y	Relative	0	1	0	9
253	Wy	Story22	C94	Force	Global-Y	Relative	0	1	0	9
334	Wy	Story22	C116	Force	Global-X	Relative	0	1	0	9
336	Wy	Story22	C117	Force	Global-X	Relative	0	1	0	9
405	Wy	Story22	C170	Force	Global-X	Relative	0	1	0	9
407	Wy	Story22	C171	Force	Global-X	Relative	0	1	0	9
476	Wy	Story22	C193	Force	Global-X	Relative	0	1	0	9
478	Wy	Story22	C194	Force	Global-X	Relative	0	1	0	9
178	Wy	Story11	B1	Force	Global-Y	Relative	0	0.3333	0	7.4333
178	Wy	Story11	B1	Force	Global-Y	Relative	0.3333	0.6667	7.4333	14.8667
178	Wy	Story11	B1	Force	Global-Y	Relative	0.6667	1	14.8667	22.3
6	Wy	Story11	B3	Force	Global-Y	Relative	0	1	0	22.3
254	Wy	Story11	B97	Force	Global-Y	Relative	0	1	0	22.3
255	Wy	Story11	B98	Force	Global-Y	Relative	0	1	0	22.3
546	Wy	Story11	B73	Force	Global-X	Relative	0	1	0	9.1
547	Wy	Story11	B74	Force	Global-X	Relative	0	1	0	24.6
548	Wy	Story11	B75	Force	Global-X	Relative	0	1	0	24.6
549	Wy	Story11	B76	Force	Global-X	Relative	0	1	0	24.6
550	Wy	Story11	B77	Force	Global-X	Relative	0	1	0	24.6
551	Wy	Story11	B78	Force	Global-X	Relative	0	1	0	9.1
594	Wy	Story11	B80	Force	Global-X	Relative	0	1	0	9.1
595	Wy	Story11	B81	Force	Global-X	Relative	0	1	0	24.6
596	Wy	Story11	B82	Force	Global-X	Relative	0	1	0	24.6
597	Wy	Story11	B83	Force	Global-X	Relative	0	1	0	24.6
598	Wy	Story11	B84	Force	Global-X	Relative	0	1	0	24.6
599	Wy	Story11	B85	Force	Global-X	Relative	0	1	0	9.1
13	Wy	Story11	B5	Force	Global-Y	Relative	0	1	0	10.9278
14	Wy	Story11	B6	Force	Global-Y	Relative	0	1	0	10.9278
15	Wy	Story11	B7	Force	Global-Y	Relative	0	1	0	10.9278
16	Wy	Story11	B8	Force	Global-Y	Relative	0	1	0	10.9278
17	Wy	Story11	B9	Force	Global-Y	Relative	0	1	0	10.9278
18	Wy	Story11	B10	Force	Global-Y	Relative	0	1	0	10.9278
19	Wy	Story11	B11	Force	Global-Y	Relative	0	1	0	10.9278
20	Wy	Story11	B12	Force	Global-Y	Relative	0	1	0	10.9278
21	Wy	Story11	B13	Force	Global-Y	Relative	0	1	0	10.9278
22	Wy	Story11	B14	Force	Global-Y	Relative	0	1	0	10.9278
23	Wy	Story11	B15	Force	Global-Y	Relative	0	1	0	10.9278
24	Wy	Story11	B16	Force	Global-Y	Relative	0	1	0	10.9278
25	Wy	Story11	B17	Force	Global-Y	Relative	0	1	0	10.9278
26	Wy	Story11	B18	Force	Global-Y	Relative	0	1	0	10.9278
27	Wy	Story11	B19	Force	Global-Y	Relative	0	1	0	10.9278

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
28	Wy	Story11	B20	Force	Global-Y	Relative	0	1	0	10.9278
29	Wy	Story11	B21	Force	Global-Y	Relative	0	1	0	10.9278
96	Wy	Story11	B22	Force	Global-Y	Relative	0	1	0	10.9278
263	Wy	Story11	B99	Force	Global-Y	Relative	0	1	0	10.9278
264	Wy	Story11	B100	Force	Global-Y	Relative	0	1	0	10.9278
265	Wy	Story11	B101	Force	Global-Y	Relative	0	1	0	10.9278
266	Wy	Story11	B102	Force	Global-Y	Relative	0	1	0	10.9278
267	Wy	Story11	B103	Force	Global-Y	Relative	0	1	0	10.9278
268	Wy	Story11	B104	Force	Global-Y	Relative	0	1	0	10.9278
269	Wy	Story11	B105	Force	Global-Y	Relative	0	1	0	10.9278
270	Wy	Story11	B106	Force	Global-Y	Relative	0	1	0	10.9278
271	Wy	Story11	B107	Force	Global-Y	Relative	0	1	0	10.9278
272	Wy	Story11	B108	Force	Global-Y	Relative	0	1	0	10.9278
273	Wy	Story11	B109	Force	Global-Y	Relative	0	1	0	10.9278
274	Wy	Story11	B110	Force	Global-Y	Relative	0	1	0	10.9278
275	Wy	Story11	B111	Force	Global-Y	Relative	0	1	0	10.9278
276	Wy	Story11	B112	Force	Global-Y	Relative	0	1	0	10.9278
277	Wy	Story11	B113	Force	Global-Y	Relative	0	1	0	10.9278
278	Wy	Story11	B114	Force	Global-Y	Relative	0	1	0	10.9278
341	Wy	Story11	B115	Force	Global-Y	Relative	0	1	0	10.9278
342	Wy	Story11	B116	Force	Global-Y	Relative	0	1	0	10.9278
5	Wy	Story11	C1	Force	Global-Y	Relative	0	0.3333	0	11.3333
5	Wy	Story11	C1	Force	Global-Y	Relative	0.3333	0.6667	11.3333	22.6667
5	Wy	Story11	C1	Force	Global-Y	Relative	0.6667	1	22.6667	34
3	Wy	Story11	C2	Force	Global-Y	Relative	0	1	0	34
8	Wy	Story11	C5	Force	Global-Y	Relative	0	1	0	34
10	Wy	Story11	C7	Force	Global-Y	Relative	0	1	0	34
147	Wy	Story11	C3	Force	Global-X	Relative	0	0.3333	0	11.3333
147	Wy	Story11	C3	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
147	Wy	Story11	C3	Force	Global-X	Relative	0.6667	1	22.6667	34
86	Wy	Story11	C6	Force	Global-X	Relative	0	1	0	34
344	Wy	Story11	C47	Force	Global-X	Relative	0	0.3333	0	11.3333
344	Wy	Story11	C47	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
344	Wy	Story11	C47	Force	Global-X	Relative	0.6667	1	22.6667	34
169	Wy	Story11	C48	Force	Global-X	Relative	0	1	0	34
361	Wy	Story11	C93	Force	Global-Y	Relative	0	0.3333	0	11.3333
361	Wy	Story11	C93	Force	Global-Y	Relative	0.3333	0.6667	11.3333	22.6667
361	Wy	Story11	C93	Force	Global-Y	Relative	0.6667	1	22.6667	34
252	Wy	Story11	C94	Force	Global-Y	Relative	0	1	0	34
257	Wy	Story11	C96	Force	Global-Y	Relative	0	1	0	34
259	Wy	Story11	C98	Force	Global-Y	Relative	0	1	0	34
333	Wy	Story11	C116	Force	Global-X	Relative	0	1	0	34
335	Wy	Story11	C117	Force	Global-X	Relative	0	1	0	34
404	Wy	Story11	C170	Force	Global-X	Relative	0	1	0	34
406	Wy	Story11	C171	Force	Global-X	Relative	0	1	0	34
475	Wy	Story11	C193	Force	Global-X	Relative	0	1	0	34
477	Wy	Story11	C194	Force	Global-X	Relative	0	1	0	34
2	Wxx	Story22	C1	Force	Global-Y	Relative	0	1	0	9
4	Wxx	Story22	C2	Force	Global-Y	Relative	0	1	0	9
251	Wxx	Story22	C93	Force	Global-Y	Relative	0	1	0	9
253	Wxx	Story22	C94	Force	Global-Y	Relative	0	1	0	9
5	Wxx	Story11	C1	Force	Global-Y	Relative	0	0.3333	0	11.3333
5	Wxx	Story11	C1	Force	Global-Y	Relative	0.3333	0.6667	11.3333	22.6667
5	Wxx	Story11	C1	Force	Global-Y	Relative	0.6667	1	22.6667	34

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
3	Wxx	Story11	C2	Force	Global-Y	Relative	0	1	0	34
361	Wxx	Story11	C93	Force	Global-Y	Relative	0	0.3333	0	11.3333
361	Wxx	Story11	C93	Force	Global-Y	Relative	0.3333	0.6667	11.3333	22.6667
361	Wxx	Story11	C93	Force	Global-Y	Relative	0.6667	1	22.6667	34
252	Wxx	Story11	C94	Force	Global-Y	Relative	0	1	0	34
2	Wyy	Story22	C1	Force	Global-X	Relative	0	1	0	9
4	Wyy	Story22	C2	Force	Global-X	Relative	0	1	0	9
251	Wyy	Story22	C93	Force	Global-X	Relative	0	1	0	9
253	Wyy	Story22	C94	Force	Global-X	Relative	0	1	0	9
5	Wyy	Story11	C1	Force	Global-X	Relative	0	0.3333	0	11.3333
5	Wyy	Story11	C1	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
5	Wyy	Story11	C1	Force	Global-X	Relative	0.6667	1	22.6667	34
3	Wyy	Story11	C2	Force	Global-X	Relative	0	1	0	34
361	Wyy	Story11	C93	Force	Global-X	Relative	0	0.3333	0	11.3333
361	Wyy	Story11	C93	Force	Global-X	Relative	0.3333	0.6667	11.3333	22.6667
361	Wyy	Story11	C93	Force	Global-X	Relative	0.6667	1	22.6667	34
252	Wyy	Story11	C94	Force	Global-X	Relative	0	1	0	34
343	Sb	Story33	B26	Force	Gravity	Relative	0	1	0	22.3765
66	Sb	Story33	B28	Force	Gravity	Relative	0	1	0	22.3765
362	Sb	Story33	B45	Force	Gravity	Relative	0	1	0	22.3765
149	Sb	Story33	B47	Force	Gravity	Relative	0	1	0	22.3765
230	Sb	Story33	B69	Force	Gravity	Relative	0	1	0	22.3765
232	Sb	Story33	B71	Force	Gravity	Relative	0	1	0	22.3765
313	Sb	Story33	B117	Force	Gravity	Relative	0	1	0	22.3765
315	Sb	Story33	B119	Force	Gravity	Relative	0	1	0	22.3765
429	Sb	Story33	B29	Force	Gravity	Relative	0	0.7778	0	76.7567
429	Sb	Story33	B29	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
431	Sb	Story33	B46	Force	Gravity	Relative	0	0.7778	0	76.7567
431	Sb	Story33	B46	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
1	Sb	Story33	B48	Force	Gravity	Relative	0	0.7778	0	76.7567
1	Sb	Story33	B48	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
84	Sb	Story33	B70	Force	Gravity	Relative	0	0.7778	0	76.7567
84	Sb	Story33	B70	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
148	Sb	Story33	B72	Force	Gravity	Relative	0	0.7778	0	76.7567
148	Sb	Story33	B72	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
167	Sb	Story33	B118	Force	Gravity	Relative	0	0.7778	0	76.7567
167	Sb	Story33	B118	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
233	Sb	Story33	B120	Force	Gravity	Relative	0	0.7778	0	76.7567
233	Sb	Story33	B120	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
314	Sb	Story33	B124	Force	Gravity	Relative	0	0.7778	0	76.7567
314	Sb	Story33	B124	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
416	Sb	Story33	B125	Force	Gravity	Relative	0	0.8189	0	99.1332
416	Sb	Story33	B125	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
418	Sb	Story33	B192	Force	Gravity	Relative	0	0.8189	0	99.1332
418	Sb	Story33	B192	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
420	Sb	Story33	B193	Force	Gravity	Relative	0	0.8189	0	99.1332
420	Sb	Story33	B193	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
423	Sb	Story33	B216	Force	Gravity	Relative	0	0.8189	0	99.1332
423	Sb	Story33	B216	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
425	Sb	Story33	B217	Force	Gravity	Relative	0	0.8189	0	99.1332
425	Sb	Story33	B217	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
427	Sb	Story33	B218	Force	Gravity	Relative	0	0.8189	0	99.1332
427	Sb	Story33	B218	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
343	Su1	Story33	B26	Force	Gravity	Relative	0	1	0	22.3765

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
66	Su1	Story33	B28	Force	Gravity	Relative	0	1	0	22.3765
362	Su1	Story33	B45	Force	Gravity	Relative	0	1	0	22.3765
149	Su1	Story33	B47	Force	Gravity	Relative	0	1	0	22.3765
230	Su1	Story33	B69	Force	Gravity	Relative	0	1	0	22.3765
232	Su1	Story33	B71	Force	Gravity	Relative	0	1	0	22.3765
313	Su1	Story33	B117	Force	Gravity	Relative	0	1	0	22.3765
315	Su1	Story33	B119	Force	Gravity	Relative	0	1	0	22.3765
429	Su1	Story33	B29	Force	Gravity	Relative	0	0.7778	0	76.7567
429	Su1	Story33	B29	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
431	Su1	Story33	B46	Force	Gravity	Relative	0	0.7778	0	76.7567
431	Su1	Story33	B46	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
1	Su1	Story33	B48	Force	Gravity	Relative	0	0.7778	0	76.7567
1	Su1	Story33	B48	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
84	Su1	Story33	B70	Force	Gravity	Relative	0	0.7778	0	76.7567
84	Su1	Story33	B70	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
148	Su1	Story33	B72	Force	Gravity	Relative	0	0.7778	0	76.7567
148	Su1	Story33	B72	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
167	Su1	Story33	B118	Force	Gravity	Relative	0	0.7778	0	76.7567
167	Su1	Story33	B118	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
233	Su1	Story33	B120	Force	Gravity	Relative	0	0.7778	0	76.7567
233	Su1	Story33	B120	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
314	Su1	Story33	B124	Force	Gravity	Relative	0	0.7778	0	76.7567
314	Su1	Story33	B124	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
416	Su1	Story33	B125	Force	Gravity	Relative	0	0.8189	0	99.1332
416	Su1	Story33	B125	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
418	Su1	Story33	B192	Force	Gravity	Relative	0	0.8189	0	99.1332
418	Su1	Story33	B192	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
420	Su1	Story33	B193	Force	Gravity	Relative	0	0.8189	0	99.1332
420	Su1	Story33	B193	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
423	Su1	Story33	B216	Force	Gravity	Relative	0	0.8189	0	99.1332
423	Su1	Story33	B216	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
425	Su1	Story33	B217	Force	Gravity	Relative	0	0.8189	0	99.1332
425	Su1	Story33	B217	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
427	Su1	Story33	B218	Force	Gravity	Relative	0	0.8189	0	99.1332
427	Su1	Story33	B218	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
343	Su2	Story33	B26	Force	Gravity	Relative	0	1	0	22.3765
66	Su2	Story33	B28	Force	Gravity	Relative	0	1	0	22.3765
362	Su2	Story33	B45	Force	Gravity	Relative	0	1	0	22.3765
149	Su2	Story33	B47	Force	Gravity	Relative	0	1	0	22.3765
230	Su2	Story33	B69	Force	Gravity	Relative	0	1	0	22.3765
232	Su2	Story33	B71	Force	Gravity	Relative	0	1	0	22.3765
313	Su2	Story33	B117	Force	Gravity	Relative	0	1	0	22.3765
315	Su2	Story33	B119	Force	Gravity	Relative	0	1	0	22.3765
429	Su2	Story33	B29	Force	Gravity	Relative	0	0.7778	0	76.7567
429	Su2	Story33	B29	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
431	Su2	Story33	B46	Force	Gravity	Relative	0	0.7778	0	76.7567
431	Su2	Story33	B46	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
1	Su2	Story33	B48	Force	Gravity	Relative	0	0.7778	0	76.7567
1	Su2	Story33	B48	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
84	Su2	Story33	B70	Force	Gravity	Relative	0	0.7778	0	76.7567
84	Su2	Story33	B70	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
148	Su2	Story33	B72	Force	Gravity	Relative	0	0.7778	0	76.7567
148	Su2	Story33	B72	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
167	Su2	Story33	B118	Force	Gravity	Relative	0	0.7778	0	76.7567

Unique Name	Load Pattern	Story	Label	Load Type	Direction	Distance Type	Relative Distance A	Relative Distance B	Absolute Distance A ft	Absolute Distance B ft
167	Su2	Story33	B118	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
233	Su2	Story33	B120	Force	Gravity	Relative	0	0.7778	0	76.7567
233	Su2	Story33	B120	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
314	Su2	Story33	B124	Force	Gravity	Relative	0	0.7778	0	76.7567
314	Su2	Story33	B124	Force	Gravity	Relative	0.7778	1	76.7567	98.6872
416	Su2	Story33	B125	Force	Gravity	Relative	0	0.8189	0	99.1332
416	Su2	Story33	B125	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
418	Su2	Story33	B192	Force	Gravity	Relative	0	0.8189	0	99.1332
418	Su2	Story33	B192	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
420	Su2	Story33	B193	Force	Gravity	Relative	0	0.8189	0	99.1332
420	Su2	Story33	B193	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
423	Su2	Story33	B216	Force	Gravity	Relative	0	0.8189	0	99.1332
423	Su2	Story33	B216	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
425	Su2	Story33	B217	Force	Gravity	Relative	0	0.8189	0	99.1332
425	Su2	Story33	B217	Force	Gravity	Relative	0.8189	1	99.1332	121.0637
427	Su2	Story33	B218	Force	Gravity	Relative	0	0.8189	0	99.1332
427	Su2	Story33	B218	Force	Gravity	Relative	0.8189	1	99.1332	121.0637

Frame Loads Assignments - Distributed (Part 2 of 2)

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
343	Dead	0.114	0.114
343	Dead	0.114	0.114
343	Dead	0.114	0.114
66	Dead	0.114	0.114
362	Dead	0.421	0.421
362	Dead	0.421	0.421
362	Dead	0.421	0.421
149	Dead	0.421	0.421
230	Dead	0.421	0.421
232	Dead	0.421	0.421
313	Dead	0.114	0.114
315	Dead	0.114	0.114
429	Dead	0.114	0.114
429	Dead	0.114	0.114
431	Dead	0.114	0.114
431	Dead	0.114	0.114
1	Dead	0.421	0.421
1	Dead	0.421	0.421
84	Dead	0.421	0.421
84	Dead	0.421	0.421
148	Dead	0.421	0.421
148	Dead	0.421	0.421
167	Dead	0.421	0.421
167	Dead	0.421	0.421
233	Dead	0.114	0.114
233	Dead	0.114	0.114
314	Dead	0.114	0.114
314	Dead	0.114	0.114
416	Dead	0.615	0.615
416	Dead	0.615	0.615
418	Dead	0.615	0.615

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
418	Dead	0.615	0.615
420	Dead	0.615	0.615
420	Dead	0.615	0.615
423	Dead	0.615	0.615
423	Dead	0.615	0.615
425	Dead	0.615	0.615
425	Dead	0.615	0.615
427	Dead	0.615	0.615
427	Dead	0.615	0.615
343	Live	0.091	0.091
343	Live	0.091	0.091
343	Live	0.091	0.091
66	Live	0.091	0.091
362	Live	0.337	0.337
362	Live	0.337	0.337
362	Live	0.337	0.337
149	Live	0.337	0.337
230	Live	0.337	0.337
232	Live	0.337	0.337
313	Live	0.091	0.091
315	Live	0.091	0.091
429	Live	0.091	0.091
429	Live	0.091	0.091
431	Live	0.091	0.091
431	Live	0.091	0.091
1	Live	0.337	0.337
1	Live	0.337	0.337
84	Live	0.337	0.337
84	Live	0.337	0.337
148	Live	0.337	0.337
148	Live	0.337	0.337
167	Live	0.337	0.337
167	Live	0.337	0.337
233	Live	0.091	0.091
233	Live	0.091	0.091
314	Live	0.091	0.091
314	Live	0.091	0.091
416	Live	0.492	0.492
416	Live	0.492	0.492
418	Live	0.492	0.492
418	Live	0.492	0.492
420	Live	0.492	0.492
420	Live	0.492	0.492
423	Live	0.492	0.492
423	Live	0.492	0.492
425	Live	0.492	0.492
425	Live	0.492	0.492
427	Live	0.492	0.492
427	Live	0.492	0.492
343	Mass	0.205	0.205
343	Mass	0.205	0.205
343	Mass	0.205	0.205
66	Mass	0.205	0.205
362	Mass	0.735	0.735
362	Mass	0.735	0.735

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
362	Mass	0.735	0.735
149	Mass	0.735	0.735
230	Mass	0.735	0.735
232	Mass	0.735	0.735
313	Mass	0.205	0.205
315	Mass	0.205	0.205
429	Mass	0.205	0.205
429	Mass	0.205	0.205
431	Mass	0.205	0.205
431	Mass	0.205	0.205
1	Mass	0.735	0.735
1	Mass	0.735	0.735
84	Mass	0.735	0.735
84	Mass	0.735	0.735
148	Mass	0.735	0.735
148	Mass	0.735	0.735
167	Mass	0.735	0.735
167	Mass	0.735	0.735
233	Mass	0.205	0.205
233	Mass	0.205	0.205
314	Mass	0.205	0.205
314	Mass	0.205	0.205
416	Mass	1.07	1.07
416	Mass	1.07	1.07
418	Mass	1.07	1.07
418	Mass	1.07	1.07
420	Mass	1.07	1.07
420	Mass	1.07	1.07
423	Mass	1.07	1.07
423	Mass	1.07	1.07
425	Mass	1.07	1.07
425	Mass	1.07	1.07
427	Mass	1.07	1.07
427	Mass	1.07	1.07
343	Wx	-0.06	-0.06
343	Wx	-0.06	-0.06
343	Wx	-0.06	-0.06
66	Wx	-0.06	-0.06
362	Wx	0.27	0.27
362	Wx	0.27	0.27
362	Wx	0.27	0.27
149	Wx	0.27	0.27
230	Wx	0.27	0.27
232	Wx	0.27	0.27
313	Wx	0.06	0.06
315	Wx	0.06	0.06
429	Wx	-0.06	-0.06
429	Wx	-0.06	-0.06
431	Wx	-0.06	-0.06
431	Wx	-0.06	-0.06
1	Wx	0.27	0.27
1	Wx	0.27	0.27
84	Wx	0.27	0.27
84	Wx	0.27	0.27
148	Wx	0.27	0.27

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
148	Wx	0.27	0.27
167	Wx	0.27	0.27
167	Wx	0.27	0.27
233	Wx	0.06	0.06
233	Wx	0.06	0.06
314	Wx	0.06	0.06
314	Wx	0.06	0.06
416	Wx	0.384	0.384
416	Wx	0.384	0.384
418	Wx	0.384	0.384
418	Wx	0.384	0.384
420	Wx	0.384	0.384
420	Wx	0.384	0.384
423	Wx	0.384	0.384
423	Wx	0.384	0.384
425	Wx	0.384	0.384
425	Wx	0.384	0.384
427	Wx	0.384	0.384
427	Wx	0.384	0.384
9	Wx	-0.85	-0.85
11	Wx	-0.85	-0.85
258	Wx	0.85	0.85
260	Wx	0.85	0.85
552	Wx	0.05	0.05
553	Wx	0.05	0.05
554	Wx	0.05	0.05
555	Wx	0.05	0.05
556	Wx	0.05	0.05
557	Wx	0.05	0.05
600	Wx	0.05	0.05
601	Wx	0.05	0.05
602	Wx	0.05	0.05
603	Wx	0.05	0.05
604	Wx	0.05	0.05
605	Wx	0.05	0.05
2	Wx	0.05	0.05
4	Wx	0.05	0.05
85	Wx	0.15	0.15
87	Wx	0.15	0.15
168	Wx	0.15	0.15
170	Wx	0.15	0.15
251	Wx	0.05	0.05
253	Wx	0.05	0.05
334	Wx	0.247	0.247
336	Wx	0.247	0.247
405	Wx	0.247	0.247
407	Wx	0.247	0.247
476	Wx	0.247	0.247
478	Wx	0.247	0.247
178	Wx	-0.31	-0.31
178	Wx	-0.31	-0.31
178	Wx	-0.31	-0.31
6	Wx	-0.31	-0.31
254	Wx	0.31	0.31
255	Wx	0.31	0.31

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
546	Wx	0.32	0.32
547	Wx	0.32	0.32
548	Wx	0.32	0.32
549	Wx	0.32	0.32
550	Wx	0.32	0.32
551	Wx	0.32	0.32
594	Wx	0.32	0.32
595	Wx	0.32	0.32
596	Wx	0.32	0.32
597	Wx	0.32	0.32
598	Wx	0.32	0.32
599	Wx	0.32	0.32
13	Wx	-0.31	-0.31
14	Wx	-0.31	-0.31
15	Wx	-0.31	-0.31
16	Wx	-0.31	-0.31
17	Wx	-0.31	-0.31
18	Wx	-0.31	-0.31
19	Wx	-0.31	-0.31
20	Wx	-0.31	-0.31
21	Wx	-0.31	-0.31
22	Wx	-0.31	-0.31
23	Wx	-0.31	-0.31
24	Wx	-0.31	-0.31
25	Wx	-0.31	-0.31
26	Wx	-0.31	-0.31
27	Wx	-0.31	-0.31
28	Wx	-0.31	-0.31
29	Wx	-0.31	-0.31
96	Wx	-0.31	-0.31
263	Wx	0.31	0.31
264	Wx	0.31	0.31
265	Wx	0.31	0.31
266	Wx	0.31	0.31
267	Wx	0.31	0.31
268	Wx	0.31	0.31
269	Wx	0.31	0.31
270	Wx	0.31	0.31
271	Wx	0.31	0.31
272	Wx	0.31	0.31
273	Wx	0.31	0.31
274	Wx	0.31	0.31
275	Wx	0.31	0.31
276	Wx	0.31	0.31
277	Wx	0.31	0.31
278	Wx	0.31	0.31
341	Wx	0.31	0.31
342	Wx	0.31	0.31
5	Wx	0.05	0.05
5	Wx	0.05	0.05
5	Wx	0.05	0.05
3	Wx	0.05	0.05
8	Wx	-0.85	-0.85
10	Wx	-0.85	-0.85
147	Wx	0.15	0.15

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
147	Wx	0.15	0.15
147	Wx	0.15	0.15
86	Wx	0.15	0.15
344	Wx	0.15	0.15
344	Wx	0.15	0.15
344	Wx	0.15	0.15
169	Wx	0.15	0.15
361	Wx	0.05	0.05
361	Wx	0.05	0.05
361	Wx	0.05	0.05
252	Wx	0.05	0.05
257	Wx	0.85	0.85
259	Wx	0.85	0.85
333	Wx	0.247	0.247
335	Wx	0.247	0.247
404	Wx	0.247	0.247
406	Wx	0.247	0.247
475	Wx	0.247	0.247
477	Wx	0.247	0.247
343	Wy	0.06	0.06
343	Wy	0.06	0.06
343	Wy	0.06	0.06
66	Wy	0.06	0.06
362	Wy	0.27	0.27
362	Wy	0.27	0.27
362	Wy	0.27	0.27
149	Wy	0.27	0.27
230	Wy	0.27	0.27
232	Wy	0.27	0.27
313	Wy	0.06	0.06
315	Wy	0.06	0.06
429	Wy	0.06	0.06
429	Wy	0.06	0.06
431	Wy	0.06	0.06
431	Wy	0.06	0.06
1	Wy	0.27	0.27
1	Wy	0.27	0.27
84	Wy	0.27	0.27
84	Wy	0.27	0.27
148	Wy	0.27	0.27
148	Wy	0.27	0.27
167	Wy	0.27	0.27
167	Wy	0.27	0.27
233	Wy	0.06	0.06
233	Wy	0.06	0.06
314	Wy	0.06	0.06
314	Wy	0.06	0.06
416	Wy	0.384	0.384
416	Wy	0.384	0.384
418	Wy	0.384	0.384
418	Wy	0.384	0.384
420	Wy	0.384	0.384
420	Wy	0.384	0.384
423	Wy	0.384	0.384
423	Wy	0.384	0.384

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
425	Wy	0.384	0.384
425	Wy	0.384	0.384
427	Wy	0.384	0.384
427	Wy	0.384	0.384
9	Wy	0.85	0.85
11	Wy	0.85	0.85
258	Wy	0.85	0.85
260	Wy	0.85	0.85
552	Wy	-0.05	-0.05
553	Wy	-0.05	-0.05
554	Wy	-0.05	-0.05
555	Wy	-0.05	-0.05
556	Wy	-0.05	-0.05
557	Wy	-0.05	-0.05
600	Wy	0.05	0.05
601	Wy	0.05	0.05
602	Wy	0.05	0.05
603	Wy	0.05	0.05
604	Wy	0.05	0.05
605	Wy	0.05	0.05
2	Wy	0.07	0.07
4	Wy	0.07	0.07
85	Wy	-0.14	-0.14
87	Wy	0.14	0.14
168	Wy	-0.14	-0.14
170	Wy	0.14	0.14
251	Wy	0.07	0.07
253	Wy	0.07	0.07
334	Wy	-0.2	-0.2
336	Wy	0.2	0.2
405	Wy	-0.2	-0.2
407	Wy	0.2	0.2
476	Wy	-0.2	-0.2
478	Wy	0.2	0.2
178	Wy	0.375	0.375
178	Wy	0.375	0.375
178	Wy	0.375	0.375
6	Wy	0.375	0.375
254	Wy	0.375	0.375
255	Wy	0.375	0.375
546	Wy	-0.28	-0.28
547	Wy	-0.28	-0.28
548	Wy	-0.28	-0.28
549	Wy	-0.28	-0.28
550	Wy	-0.28	-0.28
551	Wy	-0.28	-0.28
594	Wy	0.28	0.28
595	Wy	0.28	0.28
596	Wy	0.28	0.28
597	Wy	0.28	0.28
598	Wy	0.28	0.28
599	Wy	0.28	0.28
13	Wy	0.375	0.375
14	Wy	0.375	0.375
15	Wy	0.375	0.375

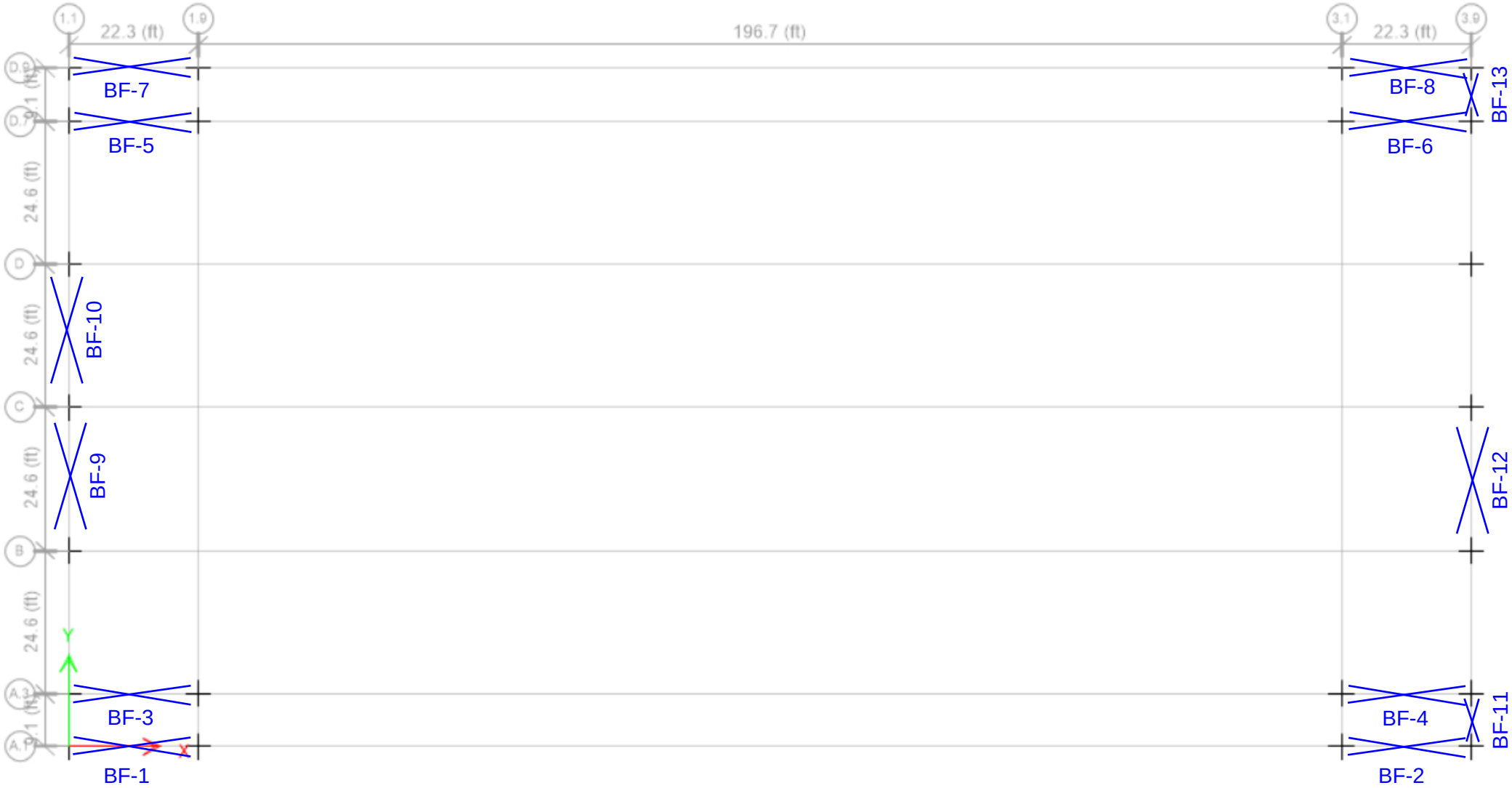
Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
16	Wy	0.375	0.375
17	Wy	0.375	0.375
18	Wy	0.375	0.375
19	Wy	0.375	0.375
20	Wy	0.375	0.375
21	Wy	0.375	0.375
22	Wy	0.375	0.375
23	Wy	0.375	0.375
24	Wy	0.375	0.375
25	Wy	0.375	0.375
26	Wy	0.375	0.375
27	Wy	0.375	0.375
28	Wy	0.375	0.375
29	Wy	0.375	0.375
96	Wy	0.375	0.375
263	Wy	0.375	0.375
264	Wy	0.375	0.375
265	Wy	0.375	0.375
266	Wy	0.375	0.375
267	Wy	0.375	0.375
268	Wy	0.375	0.375
269	Wy	0.375	0.375
270	Wy	0.375	0.375
271	Wy	0.375	0.375
272	Wy	0.375	0.375
273	Wy	0.375	0.375
274	Wy	0.375	0.375
275	Wy	0.375	0.375
276	Wy	0.375	0.375
277	Wy	0.375	0.375
278	Wy	0.375	0.375
341	Wy	0.375	0.375
342	Wy	0.375	0.375
5	Wy	0.07	0.07
5	Wy	0.07	0.07
5	Wy	0.07	0.07
3	Wy	0.07	0.07
8	Wy	0.85	0.85
10	Wy	0.85	0.85
147	Wy	-0.14	-0.14
147	Wy	-0.14	-0.14
147	Wy	-0.14	-0.14
86	Wy	0.14	0.14
344	Wy	-0.14	-0.14
344	Wy	-0.14	-0.14
344	Wy	-0.14	-0.14
169	Wy	0.14	0.14
361	Wy	0.07	0.07
361	Wy	0.07	0.07
361	Wy	0.07	0.07
252	Wy	0.07	0.07
257	Wy	0.85	0.85
259	Wy	0.85	0.85
333	Wy	-0.2	-0.2
335	Wy	0.2	0.2

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
404	Wy	-0.2	-0.2
406	Wy	0.2	0.2
475	Wy	-0.2	-0.2
477	Wy	0.2	0.2
2	Wxx	-0.07	-0.07
4	Wxx	-0.07	-0.07
251	Wxx	0.07	0.07
253	Wxx	0.07	0.07
5	Wxx	-0.07	-0.07
5	Wxx	-0.07	-0.07
5	Wxx	-0.07	-0.07
3	Wxx	-0.07	-0.07
361	Wxx	0.07	0.07
361	Wxx	0.07	0.07
361	Wxx	0.07	0.07
252	Wxx	0.07	0.07
2	Wyy	-0.05	-0.05
4	Wyy	0.05	0.05
251	Wyy	-0.05	-0.05
253	Wyy	0.05	0.05
5	Wyy	-0.05	-0.05
5	Wyy	-0.05	-0.05
5	Wyy	-0.05	-0.05
3	Wyy	0.05	0.05
361	Wyy	-0.05	-0.05
361	Wyy	-0.05	-0.05
361	Wyy	-0.05	-0.05
252	Wyy	0.05	0.05
343	Sb	0.095	0.095
66	Sb	0.095	0.095
362	Sb	0.357	0.357
149	Sb	0.357	0.357
230	Sb	0.357	0.357
232	Sb	0.357	0.357
313	Sb	0.095	0.095
315	Sb	0.095	0.095
429	Sb	0.095	0.095
429	Sb	0.095	0.095
431	Sb	0.095	0.095
431	Sb	0.095	0.095
1	Sb	0.357	0.357
1	Sb	0.357	0.357
84	Sb	0.357	0.357
84	Sb	0.357	0.357
148	Sb	0.357	0.357
148	Sb	0.357	0.357
167	Sb	0.357	0.357
167	Sb	0.357	0.357
233	Sb	0.095	0.095
233	Sb	0.095	0.095
314	Sb	0.095	0.095
314	Sb	0.095	0.095
416	Sb	0.517	0.517
416	Sb	0.517	0.517
418	Sb	0.517	0.517

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
418	Sb	0.517	0.517
420	Sb	0.517	0.517
420	Sb	0.517	0.517
423	Sb	0.517	0.517
423	Sb	0.517	0.517
425	Sb	0.517	0.517
425	Sb	0.517	0.517
427	Sb	0.517	0.517
427	Sb	0.517	0.517
343	Su1	0.024	0.024
66	Su1	0.08	0.08
362	Su1	0.089	0.089
149	Su1	0.298	0.298
230	Su1	0.089	0.089
232	Su1	0.298	0.298
313	Su1	0.024	0.024
315	Su1	0.08	0.08
429	Su1	0.024	0.024
429	Su1	0.024	0.024
431	Su1	0.08	0.08
431	Su1	0.151	0.151
1	Su1	0.089	0.089
1	Su1	0.089	0.089
84	Su1	0.298	0.298
84	Su1	0.561	0.561
148	Su1	0.089	0.089
148	Su1	0.089	0.089
167	Su1	0.298	0.298
167	Su1	0.561	0.561
233	Su1	0.024	0.024
233	Su1	0.024	0.024
314	Su1	0.08	0.08
314	Su1	0.151	0.151
416	Su1	0.128	0.128
416	Su1	0.128	0.128
418	Su1	0.431	0.431
418	Su1	0.812	0.812
420	Su1	0.128	0.128
420	Su1	0.128	0.128
423	Su1	0.431	0.431
423	Su1	0.812	0.812
425	Su1	0.128	0.128
425	Su1	0.128	0.128
427	Su1	0.431	0.431
427	Su1	0.812	0.812
343	Su2	0.08	0.08
66	Su2	0.024	0.024
362	Su2	0.298	0.298
149	Su2	0.089	0.089
230	Su2	0.298	0.298
232	Su2	0.089	0.089
313	Su2	0.08	0.08
315	Su2	0.024	0.024
429	Su2	0.08	0.08
429	Su2	0.151	0.151

Unique Name	Load Pattern	Force A kip/ft	Force B kip/ft
431	Su2	0.024	0.024
431	Su2	0.024	0.024
1	Su2	0.298	0.298
1	Su2	0.561	0.561
84	Su2	0.089	0.089
84	Su2	0.089	0.089
148	Su2	0.298	0.298
148	Su2	0.561	0.561
167	Su2	0.089	0.089
167	Su2	0.089	0.089
233	Su2	0.08	0.08
233	Su2	0.151	0.151
314	Su2	0.024	0.024
314	Su2	0.024	0.024
416	Su2	0.431	0.431
416	Su2	0.812	0.812
418	Su2	0.128	0.128
418	Su2	0.128	0.128
420	Su2	0.431	0.431
420	Su2	0.812	0.812
423	Su2	0.128	0.128
423	Su2	0.128	0.128
425	Su2	0.431	0.431
425	Su2	0.812	0.812
427	Su2	0.128	0.128
427	Su2	0.128	0.128

Lateral System Overview



S.O. No. _____

Subject: Creech Part 2 - Hangar - Diaphragm Loads

Sheet No. _____ of _____

Drawing No. _____

Computed by BT Checked by _____ Date 10/09/2025

Michael Baker INTERNATIONAL

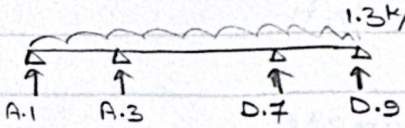
N/S → $V_{seismic} = 151^k$

$V_{ax\ 1.1} = \frac{151}{2} = 75.5^k$ $\quad V_{ax\ 3.9} = 75.5^k$

$V_{BF-9} = V_{BF-10} = \frac{75.5 \times 1000}{2 \times 24.6} = 1535$ plf

$V_{BF-11} = V_{BF-12} = V_{BF-13} = 1764$ plf

E/W → $V_{seismic} = 151^k$



$V_{ax\ A.1, D.9} = 1.3 \times 9.1/2 = 6^k$

$V_{ax\ A.3, D.7} = 1.3\left(\frac{9.1}{2} + 49.2\right) = 70^k$

$V_{BF-1} = V_{BF-2} = V_{BF-7} = V_{BF-8} = \frac{6 \times 1000}{2 \times 22.3} = 135$ plf

$V_{BF-3} = V_{BF-4} = V_{BF-5} = V_{BF-6} = \frac{70 \times 1000}{2 \times 22.3} = 1570$ plf

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N/S → $V_{wind} = 372^k$

$V_{ax\ 1.1} = \frac{372}{2} = 186^k$ $\quad V_{ax\ 3.9} = 186^k$

$V_{BF-9} = V_{BF-10} = \frac{186 \times 1000}{2 \times 24.6} = 3781$ plf

$V_{BF-11} = V_{BF-12} = V_{BF-13} = 4346$ plf

E/W → $V_{wind} = 180^k$

$V_{ax\ A.1} = V_{ax\ D.9} = 1.54 \times 9.1/2 = 7^k$

$V_{ax\ A.3} = V_{ax\ D.7} = 1.54 \times (9.1/2 + 49.2) = 83^k$

$V_{BF-1} = V_{BF-2} = V_{BF-7} = V_{BF-8} = \frac{7 \times 1000}{2 \times 22.3} = 157$ plf

$V_{BF-3} = V_{BF-4} = V_{BF-5} = V_{BF-6} = \frac{83 \times 1000}{2 \times 22.3} = 1861$ plf

20 ga 1.5PLB-36 Grade 50 Roof Deck

Diaphragm Shear & Wind Uplift Interaction

For Both Ends Lapped Deck
with MWFRS Design Net Wind Uplift (LRFD) of 16 psf

Hilti X-HSN24 PAF Connections to Supports
36 / 7 / 4 Perpendicular Connection Pattern to Supports
PunchLok II Connection Sidelap Connections

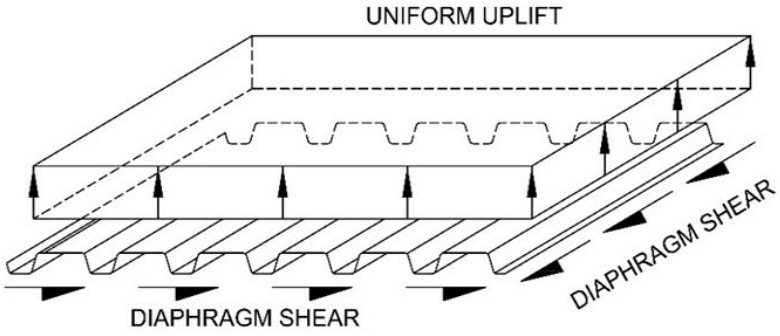
A572 GR50 Support Member or Equivalent
0.188 ≤ Support Thickness (in.) ≤ 0.375
6 in. Minimum Deck End Bearing Length

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

Sidelap Connection Spacing (in.)	Span 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
4	2559	2533	2507	2481	2377	2105	1878	1685	1521
6	2559	2533	2507	2481	2377	2105	1878	1685	1521
8	2361	2359	2354	2347	2277	2105	1878	1685	1521
12	1999	2048	1957	2000	1921	1958	1878	1685	1521
18	1612	1724	1633	1549	1644	1571	1503	1583	1521
24	1360	1515	1428	1348	1276	1402	1337	1277	1222
36	1065	1270	1190	1119	1055	997	945	1096	1046

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

Sidelap Connection Spacing (in.)	Span 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
4	6	6	6	6	6	6	6	6	6
6	7	7	7	6	6	7	8	9	9
8	7	7	7	7	7	7	8	9	10
12	8	8	8	8	8	8	8	9	10
18	10	10	11	10	10	10	10	10	10
24	12	11	12	13	12	11	12	11	12
36	18	13	14	15	16	17	18	14	15



20 ga 1.5PLB-36 Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck

NUCOR VULCRAFT GROUP

Hilti X-HSN24 PAF Connections to Supports
36 / 7 / 4 Perpendicular Connection Pattern to Supports
PunchLok II Connection Sidelap Connections

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

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

Sidelap Connection Spacing (in.)	Span 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
4	228	229	228	228	227	228	227	227	226
6	208	207	207	207	206	206	206	205	205
8	192	193	193	194	190	190	191	192	188
12	169	172	167	170	165	168	164	167	163
18	147	153	148	143	148	144	140	145	141
24	132	141	136	131	126	134	129	126	122
36	115	127	121	117	112	108	104	114	110

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

Sidelap Connection Spacing (in.)	Span 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
4	2513	2513	2513	2513	2377	2105	1878	1685	1521
6	2445	2440	2436	2433	2377	2105	1878	1685	1521
8	2224	2238	2249	2259	2199	2105	1878	1685	1521
12	1854	1912	1830	1884	1812	1861	1797	1685	1521
18	1480	1592	1510	1434	1531	1464	1402	1488	1431
24	1245	1393	1314	1243	1178	1300	1241	1187	1136
36	977	1165	1094	1030	973	922	875	1016	971

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

Sidelap Connection Spacing (in.)	Span 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
4	6	6	6	6	6	6	6	6	6
6	7	7	7	6	7	7	8	9	9
8	7	7	7	7	7	7	8	10	11
12	9	9	8	8	9	9	9	10	11
18	12	10	11	11	11	11	12	10	11
24	14	11	12	13	14	13	14	14	15
36	18	16	17	18	16	17	18	16	17

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: 5/13/2025

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Deck-Slab Diaphragm Shear Strength

20 ga 1.5VL-36 Grade 50 Composite Deck

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

Single Mat of Rebar #4 x #4 - 12 x 12 in o.c.

NUCOR® VULCRAFT

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

Perpendicular Connection Pattern[1]			2 per rib	
Parallel Connection Attachment (maximum)	1	row at	3	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

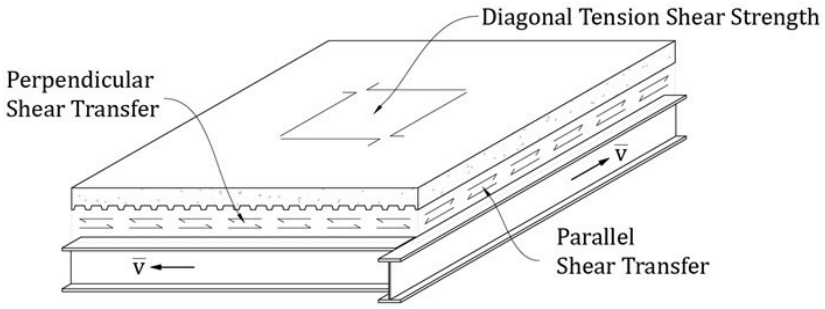
Available Diaphragm Design Strength Controlled by Connections to Chords and Collectors	**Va = (ΦPn)N =**	**5028**	**plf**
Deck-Slab Diaphragm Design Shear Stiffness	G' =	1050	kip/in

Deck-Slab Shear Strength

Reinforced Concrete Slab Design Shear Strength	Vu =	11384	plf

Chords & Collector Shear Transfer Strength

Chord & Collectors Design Shear Transfer Strength	(ΦPn)N =	5028	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 in accordance with: Date: 5/13/2025

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

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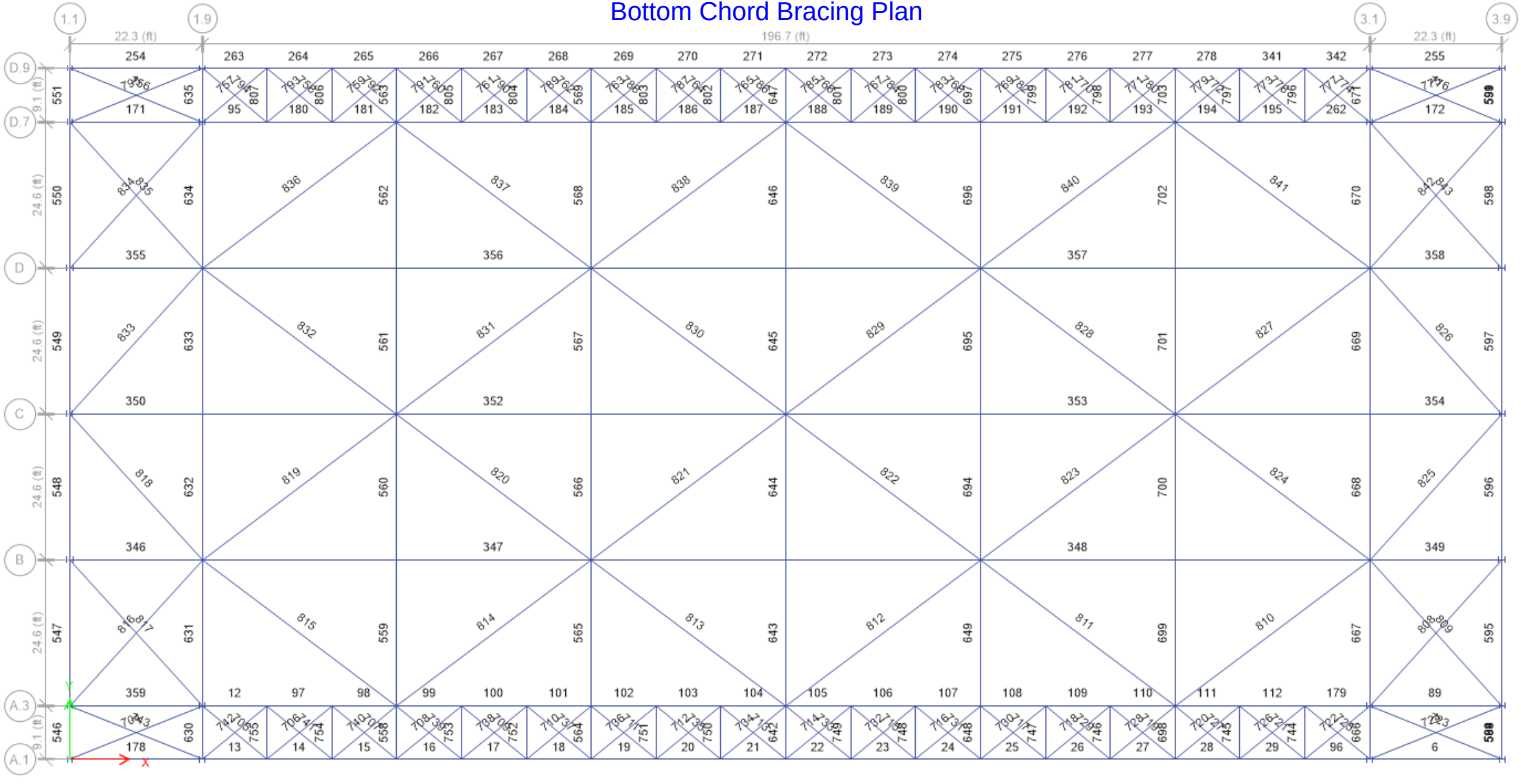
Bottom Chord Bracing Plan



Hangar – Horizontal bottom chord beams and braces showing DC ratio (designed with omega seismic load combinations)

Label	Section	D/C Ratio	
6	W12X45	0.354 = 0.337 + 0.017 + 0	DStlS44-1_O
12	W12X53	0.22 = 0.217 + 0.003 + 0	DStlS45-1_O
13	W12X53	0.243 = 0.24 + 0.003 + 0	DStlS45-1_O
14	W12X53	0.062 = 0.058 + 0.004 + 0	DStlS45-1_O
15	W12X53	0.008 = 0.004 + 0.004 + 0	DStlS45-2_O
16	W12X53	0.035 = 0.031 + 0.004 + 0	DStlS44-1_O
17	W12X53	0.064 = 0.06 + 0.004 + 0	DStlS44-1_O
18	W12X53	0.089 = 0.085 + 0.004 + 0	DStlS47-1_O
19	W12X53	0.102 = 0.098 + 0.004 + 0	DStlS47-1_O
20	W12X53	0.204 = 0.2 + 0.003 + 0	DStlS47-1_O
21	W12X53	0.103 = 0.099 + 0.004 + 0	DStlS47-1_O
22	W12X53	0.103 = 0.099 + 0.004 + 0	DStlS47-1_O
23	W12X53	0.203 = 0.2 + 0.003 + 0	DStlS47-1_O
24	W12X53	0.102 = 0.098 + 0.004 + 0	DStlS47-1_O
25	W12X53	0.089 = 0.085 + 0.004 + 0	DStlS47-1_O
26	W12X53	0.064 = 0.06 + 0.004 + 0	DStlS47-1_O
27	W12X53	0.035 = 0.031 + 0.004 + 0	DStlS45-1_O
28	W12X53	0.008 = 0.004 + 0.004 + 0	DStlS44-3_O
29	W12X53	0.063 = 0.059 + 0.004 + 0	DStlS44-1_O
89	W12X45	0.073 = 0.055 + 0.019 + 0	DStlS47-1_O
95	W12X53	0.222 = 0.219 + 0.003 + 0	DStlS45-1_O
96	W12X53	0.244 = 0.242 + 0.003 + 0	DStlS44-1_O
97	W12X53	0.056 = 0.053 + 0.004 + 0	DStlS45-1_O
98	W12X53	0.017 = 0.014 + 0.004 + 0	DStlS45-2_O
99	W12X53	0.069 = 0.066 + 0.004 + 0	DStlS44-1_O
100	W12X53	0.086 = 0.082 + 0.004 + 0	DStlS47-1_O
101	W12X53	0.093 = 0.09 + 0.004 + 0	DStlS47-1_O
102	W12X53	0.102 = 0.098 + 0.004 + 0	DStlS47-1_O
103	W12X53	0.222 = 0.219 + 0.003 + 0	DStlS47-1_O
104	W12X53	0.233 = 0.23 + 0.003 + 0	DStlS47-1_O
105	W12X53	0.232 = 0.229 + 0.003 + 0	DStlS47-1_O
106	W12X53	0.222 = 0.219 + 0.003 + 0	DStlS47-1_O
107	W12X53	0.102 = 0.098 + 0.004 + 0	DStlS47-1_O
108	W12X53	0.093 = 0.09 + 0.004 + 0	DStlS47-1_O
109	W12X53	0.086 = 0.082 + 0.004 + 0	DStlS47-1_O
110	W12X53	0.069 = 0.066 + 0.004 + 0	DStlS45-1_O
111	W12X53	0.018 = 0.014 + 0.004 + 0	DStlS44-3_O
112	W12X53	0.057 = 0.053 + 0.004 + 0	DStlS44-1_O

171	W12X45	0.091 = 0.072 + 0.019 + 0	DStlS46-1_O
172	W12X45	0.088 = 0.069 + 0.019 + 0	DStlS46-1_O
178	W12X45	0.366 = 0.35 + 0.017 + 0	DStlS45-1_O
179	W12X53	0.221 = 0.218 + 0.003 + 0	DStlS44-1_O
180	W12X53	0.057 = 0.054 + 0.004 + 0	DStlS45-1_O
181	W12X53	0.018 = 0.015 + 0.004 + 0	DStlS45-2_O
182	W12X53	0.069 = 0.066 + 0.004 + 0	DStlS44-1_O
183	W12X53	0.085 = 0.082 + 0.004 + 0	DStlS46-1_O
184	W12X53	0.093 = 0.089 + 0.004 + 0	DStlS46-1_O
185	W12X53	0.102 = 0.098 + 0.004 + 0	DStlS46-1_O
186	W12X53	0.221 = 0.218 + 0.003 + 0	DStlS46-1_O
187	W12X53	0.232 = 0.229 + 0.003 + 0	DStlS46-1_O
188	W12X53	0.233 = 0.23 + 0.003 + 0	DStlS46-1_O
189	W12X53	0.222 = 0.219 + 0.003 + 0	DStlS46-1_O
190	W12X53	0.102 = 0.099 + 0.004 + 0	DStlS46-1_O
191	W12X53	0.093 = 0.09 + 0.004 + 0	DStlS46-1_O
192	W12X53	0.086 = 0.082 + 0.004 + 0	DStlS46-1_O
193	W12X53	0.07 = 0.066 + 0.004 + 0	DStlS46-1_O
194	W12X53	0.017 = 0.014 + 0.004 + 0	DStlS44-3_O
195	W12X53	0.057 = 0.053 + 0.004 + 0	DStlS44-1_O
254	W12X45	0.37 = 0.353 + 0.017 + 0	DStlS45-1_O
255	W12X45	0.344 = 0.328 + 0.017 + 0	DStlS44-1_O
262	W12X53	0.223 = 0.22 + 0.003 + 0	DStlS44-1_O
263	W12X53	0.246 = 0.243 + 0.003 + 0	DStlS45-1_O
264	W12X53	0.063 = 0.06 + 0.004 + 0	DStlS45-1_O
265	W12X53	0.008 = 0.005 + 0.004 + 0	DStlS45-2_O
266	W12X53	0.034 = 0.031 + 0.004 + 0	DStlS44-1_O
267	W12X53	0.063 = 0.06 + 0.004 + 0	DStlS44-1_O
268	W12X53	0.088 = 0.085 + 0.004 + 0	DStlS46-1_O
269	W12X53	0.102 = 0.098 + 0.004 + 0	DStlS46-1_O
270	W12X53	0.104 = 0.1 + 0.004 + 0	DStlS46-1_O
271	W12X53	0.103 = 0.099 + 0.004 + 0	DStlS46-1_O
272	W12X53	0.103 = 0.099 + 0.004 + 0	DStlS46-1_O
273	W12X53	0.203 = 0.2 + 0.003 + 0	DStlS46-1_O
274	W12X53	0.102 = 0.098 + 0.004 + 0	DStlS46-1_O
275	W12X53	0.089 = 0.085 + 0.004 + 0	DStlS46-1_O
276	W12X53	0.064 = 0.06 + 0.004 + 0	DStlS46-1_O
277	W12X53	0.035 = 0.031 + 0.004 + 0	DStlS45-1_O
278	W12X53	0.007 = 0.004 + 0.004 + 0	DStlS44-3_O
341	W12X53	0.061 = 0.057 + 0.004 + 0	DStlS44-1_O
342	W12X53	0.239 = 0.236 + 0.003 + 0	DStlS44-1_O
346	W12X72	0.208 = 0.202 + 0.004 + 0.002	DStlS47-1_O

347	W12X72	0.379 = 0.351 + 0.028 + 2.286E-04	DStlS47-1_O
348	W12X72	0.378 = 0.35 + 0.028 + 2.24E-04	DStlS47-1_O
349	W12X72	0.207 = 0.201 + 0.004 + 0.002	DStlS47-1_O
350	W12X72	0.061 = 0.046 + 0.015 + 0	DStlS46-1_O
352	W12X72	0.395 = 0.366 + 0.029 + 0	DStlS45-1_O
353	W12X72	0.395 = 0.366 + 0.029 + 0	DStlS47-1_O
354	W12X72	0.061 = 0.046 + 0.014 + 1.69E-04	DStlS47-1_O
355	W12X72	0.209 = 0.203 + 0.004 + 0.002	DStlS46-1_O
356	W12X72	0.378 = 0.35 + 0.028 + 2.173E-04	DStlS46-1_O
357	W12X72	0.378 = 0.35 + 0.028 + 2.241E-04	DStlS46-1_O
358	W12X72	0.103 = 0.096 + 0.004 + 0.004	DStlS46-1_O
359	W12X45	0.09 = 0.072 + 0.019 + 0	DStlS47-1_O
546	W8X40	0.036 = 0.033 + 0.003 + 0	DStlS46-1_O
547	W8X31	0.074 = 0.044 + 0.03 + 0	DStlS47-1_O
548	W8X40	0.241 = 0.219 + 0.022 + 0	DStlS47-1_O
549	W8X40	0.242 = 0.22 + 0.022 + 0	DStlS46-1_O
550	W8X31	0.075 = 0.045 + 0.03 + 0	DStlS46-1_O
551	W8X40	0.036 = 0.033 + 0.003 + 0	DStlS47-1_O
558	W8X48	0.015 = 0.011 + 0.003 + 0	DStlS44-1_O
559	W8X48	0.497 = 0.471 + 0.027 + 0	DStlS47-1_O
560	W8X48	0.409 = 0.383 + 0.027 + 0	DStlS47-1_O
561	W8X48	0.41 = 0.384 + 0.027 + 0	DStlS46-1_O
562	W8X48	0.499 = 0.472 + 0.027 + 0	DStlS46-1_O
563	W8X48	0.014 = 0.011 + 0.003 + 0	DStlS44-1_O
564	W8X48	0.033 = 0.03 + 0.003 + 0	DStlS47-1_O
565	W8X48	0.095 = 0.065 + 0.03 + 0	DStlS47-1_O
566	W8X48	0.415 = 0.388 + 0.027 + 0	DStlS45-1_O
567	W8X48	0.415 = 0.388 + 0.027 + 0	DStlS45-1_O
568	W8X48	0.095 = 0.065 + 0.03 + 0	DStlS46-1_O
569	W8X48	0.033 = 0.03 + 0.003 + 0	DStlS46-1_O
594	W8X31	0.042 = 0.039 + 0.003 + 0	DStlS45-1_O
595	W8X31	0.065 = 0.035 + 0.031 + 0	DStlS47-1_O
596	W8X40	0.24 = 0.218 + 0.022 + 0	DStlS44-1_O
597	W8X40	0.262 = 0.24 + 0.022 + 0	DStlS46-1_O
598	W8X31	0.05 = 0.019 + 0.032 + 0	DStlS46-1_O
599	W8X31	0.051 = 0.048 + 0.003 + 0	DStlS46-1_O
630	W8X31	0.094 = 0.091 + 0.003 + 0	DStlS47-1_O
631	W8X31	0.076 = 0.041 + 0.036 + 0	DStlS46-1_O
632	W8X31	0.427 = 0.394 + 0.033 + 0	DStlS47-1_O
633	W8X31	0.428 = 0.395 + 0.033 + 0	DStlS46-1_O
634	W8X31	0.077 = 0.042 + 0.036 + 0	DStlS47-1_O
635	W8X31	0.094 = 0.091 + 0.003 + 0	DStlS46-1_O

642	W8X48	0.025 = 0.022 + 0.003 + 0	DStlS47-1_O
643	W8X48	0.474 = 0.447 + 0.027 + 0	DStlS47-1_O
644	W8X48	0.493 = 0.466 + 0.027 + 0	DStlS47-1_O
645	W8X48	0.493 = 0.466 + 0.027 + 0	DStlS46-1_O
646	W8X48	0.476 = 0.449 + 0.027 + 0	DStlS46-1_O
647	W8X48	0.025 = 0.022 + 0.003 + 0	DStlS46-1_O
648	W8X48	0.034 = 0.03 + 0.003 + 0	DStlS47-1_O
649	W8X48	0.095 = 0.065 + 0.03 + 0	DStlS47-1_O
666	W8X31	0.094 = 0.091 + 0.003 + 0	DStlS47-1_O
667	W8X31	0.075 = 0.039 + 0.036 + 0	DStlS46-1_O
668	W8X31	0.455 = 0.422 + 0.033 + 0	DStlS47-1_O
669	W8X31	0.439 = 0.406 + 0.033 + 0	DStlS44-1_O
670	W8X31	0.066 = 0.03 + 0.036 + 0	DStlS47-1_O
671	W8X31	0.091 = 0.088 + 0.003 + 0	DStlS46-1_O
694	W8X48	0.414 = 0.388 + 0.027 + 0	DStlS44-1_O
695	W8X48	0.414 = 0.388 + 0.027 + 0	DStlS44-1_O
696	W8X48	0.096 = 0.066 + 0.03 + 0	DStlS46-1_O
697	W8X48	0.034 = 0.03 + 0.003 + 0	DStlS46-1_O
698	W8X48	0.015 = 0.011 + 0.003 + 0	DStlS45-1_O
699	W8X48	0.5 = 0.474 + 0.027 + 0	DStlS47-1_O
700	W8X48	0.413 = 0.386 + 0.027 + 0	DStlS47-1_O
701	W8X48	0.407 = 0.38 + 0.027 + 0	DStlS46-1_O
702	W8X48	0.494 = 0.467 + 0.027 + 0	DStlS46-1_O
703	W8X48	0.014 = 0.011 + 0.003 + 0	DStlS45-1_O
704	W8X31	0.036 = 0.004 + 0.029 + 0.004	DStlS45-2_O
705	L6X6X7/8	0.131 = 0.055 + 0.044 + 0.031	DStlS45-3_O
706	L6X6X7/8	0.367 = 0.349 + 0.007 + 0.012	DStlS45-1_O
707	L6X6X7/8	0.128 = 0.05 + 0.058 + 0.02	DStlS44-1_O
708	L6X6X5/16	0.132 = 0.109 + 0.013 + 0.01	DStlS45-1_O
709	L6X6X5/16	0.227 = 0.151 + 0.028 + 0.047	DStlS47-1_O
710	L6X6X5/16	0.031 = 0.005 + 0.004 + 0.022	DStlS44-3_O
711	L6X6X5/16	0.147 = 0.084 + 0.016 + 0.047	DStlS47-1_O
712	L6X6X5/16	0.152 = 0.115 + 0.005 + 0.032	DStlS47-1_O
713	L6X6X5/16	0.144 = 0.096 + 0.012 + 0.036	DStlS47-1_O
714	L6X6X5/16	0.144 = 0.096 + 0.012 + 0.036	DStlS47-1_O
715	L6X6X5/16	0.152 = 0.114 + 0.005 + 0.033	DStlS47-1_O
716	L6X6X5/16	0.147 = 0.084 + 0.016 + 0.047	DStlS47-1_O
717	L6X6X5/16	0.031 = 0.005 + 0.004 + 0.022	DStlS45-2_O
718	L6X6X5/16	0.227 = 0.151 + 0.028 + 0.047	DStlS47-1_O
719	L6X6X5/16	0.134 = 0.111 + 0.012 + 0.011	DStlS44-1_O
720	L6X6X7/8	0.13 = 0.052 + 0.059 + 0.02	DStlS47-1_O
721	L6X6X7/8	0.37 = 0.351 + 0.007 + 0.012	DStlS44-1_O

722	L6X6X7/8	0.134 = 0.056 + 0.047 + 0.03	DStlS46-2_O
723	W8X31	0.038 = 0.008 + 0.027 + 0.003	DStlS44-3_O
724	W8X31	0.087 = 0.051 + 0.027 + 0.009	DStlS44-1_O
725	L6X6X7/8	0.462 = 0.422 + 0.021 + 0.019	DStlS44-1_O
726	L6X6X7/8	0.106 = 0.021 + 0.061 + 0.024	DStlS47-1_O
727	L6X6X7/8	0.261 = 0.25 + 0.004 + 0.006	DStlS44-1_O
728	L6X6X5/16	0.216 = 0.13 + 0.038 + 0.048	DStlS47-1_O
729	L6X6X5/16	0.065 = 0.04 + 0.019 + 0.006	DStlS44-1_O
730	L6X6X5/16	0.236 = 0.167 + 0.021 + 0.048	DStlS47-1_O
731	L6X6X5/16	0.141 = 0.109 + 0.006 + 0.026	DStlS47-1_O
732	L6X6X5/16	0.146 = 0.09 + 0.013 + 0.043	DStlS47-1_O
733	L6X6X5/16	0.165 = 0.121 + 0.008 + 0.036	DStlS47-1_O
734	L6X6X5/16	0.165 = 0.121 + 0.008 + 0.036	DStlS47-1_O
735	L6X6X5/16	0.146 = 0.09 + 0.013 + 0.043	DStlS47-1_O
736	L6X6X5/16	0.141 = 0.109 + 0.006 + 0.026	DStlS47-1_O
737	L6X6X5/16	0.236 = 0.167 + 0.021 + 0.048	DStlS47-1_O
738	L6X6X5/16	0.064 = 0.039 + 0.019 + 0.006	DStlS45-1_O
739	L6X6X5/16	0.215 = 0.129 + 0.038 + 0.048	DStlS47-1_O
740	L6X6X7/8	0.258 = 0.248 + 0.007 + 0.004	DStlS45-1_O
741	L6X6X7/8	0.104 = 0.021 + 0.061 + 0.023	DStlS44-1_O
742	L6X6X7/8	0.46 = 0.42 + 0.022 + 0.019	DStlS45-1_O
743	W8X31	0.089 = 0.055 + 0.026 + 0.009	DStlS45-1_O
744	W8X31	0.036 = 0.033 + 0.004 + 0	DStlS44-1_O
745	W8X31	0.017 = 0.014 + 0.004 + 0	DStlS44-1_O
746	W8X31	0.022 = 0.019 + 0.004 + 0	DStlS47-1_O
747	W8X31	0.028 = 0.025 + 0.004 + 0	DStlS47-1_O
748	W8X31	0.035 = 0.032 + 0.004 + 0	DStlS47-1_O
749	W8X31	0.037 = 0.034 + 0.004 + 0	DStlS47-1_O
750	W8X31	0.037 = 0.034 + 0.004 + 0	DStlS47-1_O
751	W8X31	0.035 = 0.032 + 0.004 + 0	DStlS47-1_O
752	W8X31	0.028 = 0.025 + 0.004 + 0	DStlS47-1_O
753	W8X31	0.022 = 0.019 + 0.004 + 0	DStlS44-1_O
754	W8X31	0.017 = 0.014 + 0.004 + 0	DStlS45-1_O
755	W8X31	0.036 = 0.033 + 0.004 + 0	DStlS45-1_O
756	W8X31	0.037 = 0.004 + 0.029 + 0.004	DStlS45-2_O
757	L6X6X7/8	0.139 = 0.056 + 0.056 + 0.028	DStlS45-3_O
758	L6X6X7/8	0.366 = 0.354 + 0.002 + 0.009	DStlS45-1_O
759	L6X6X7/8	0.13 = 0.049 + 0.063 + 0.018	DStlS46-1_O
760	L6X6X5/16	0.136 = 0.113 + 0.009 + 0.014	DStlS45-1_O
761	L6X6X5/16	0.228 = 0.152 + 0.024 + 0.052	DStlS46-1_O
762	L6X6X5/16	0.03 = 0.004 + 0.003 + 0.023	DStlS45-2_O
763	L6X6X5/16	0.148 = 0.084 + 0.017 + 0.047	DStlS46-1_O

764	L6X6X5/16	0.151 = 0.114 + 0.002 + 0.035	DStlS46-1_O
765	L6X6X5/16	0.146 = 0.097 + 0.006 + 0.043	DStlS46-1_O
766	L6X6X5/16	0.144 = 0.096 + 0.006 + 0.043	DStlS46-1_O
767	L6X6X5/16	0.153 = 0.115 + 0.002 + 0.036	DStlS46-1_O
768	L6X6X5/16	0.147 = 0.084 + 0.017 + 0.046	DStlS46-1_O
769	L6X6X5/16	0.032 = 0.005 + 0.003 + 0.023	DStlS45-2_O
770	L6X6X5/16	0.227 = 0.151 + 0.024 + 0.052	DStlS46-1_O
771	L6X6X5/16	0.131 = 0.107 + 0.01 + 0.014	DStlS44-1_O
772	L6X6X7/8	0.13 = 0.049 + 0.063 + 0.018	DStlS46-1_O
773	L6X6X7/8	0.355 = 0.343 + 0.002 + 0.01	DStlS44-1_O
774	L6X6X7/8	0.146 = 0.062 + 0.057 + 0.028	DStlS44-1_O
775	W8X31	0.038 = 0.008 + 0.027 + 0.002	DStlS44-3_O
776	W8X31	0.086 = 0.05 + 0.028 + 0.009	DStlS44-1_O
777	L6X6X7/8	0.438 = 0.414 + 0.011 + 0.013	DStlS44-1_O
778	L6X6X7/8	0.106 = 0.018 + 0.064 + 0.024	DStlS46-1_O
779	L6X6X7/8	0.25 = 0.242 + 0.005 + 0.004	DStlS44-1_O
780	L6X6X5/16	0.216 = 0.129 + 0.031 + 0.056	DStlS46-1_O
781	L6X6X5/16	0.064 = 0.037 + 0.006 + 0.021	DStlS44-3_O
782	L6X6X5/16	0.236 = 0.167 + 0.02 + 0.049	DStlS46-1_O
783	L6X6X5/16	0.141 = 0.109 + 0.005 + 0.027	DStlS46-1_O
784	L6X6X5/16	0.146 = 0.089 + 0.011 + 0.045	DStlS46-1_O
785	L6X6X5/16	0.166 = 0.122 + 0.006 + 0.039	DStlS46-1_O
786	L6X6X5/16	0.164 = 0.12 + 0.006 + 0.038	DStlS46-1_O
787	L6X6X5/16	0.147 = 0.09 + 0.011 + 0.045	DStlS46-1_O
788	L6X6X5/16	0.14 = 0.108 + 0.005 + 0.027	DStlS46-1_O
789	L6X6X5/16	0.237 = 0.168 + 0.021 + 0.049	DStlS46-1_O
790	L6X6X5/16	0.069 = 0.042 + 0.006 + 0.021	DStlS45-2_O
791	L6X6X5/16	0.217 = 0.13 + 0.031 + 0.056	DStlS46-1_O
792	L6X6X7/8	0.262 = 0.253 + 0.005 + 0.004	DStlS45-1_O
793	L6X6X7/8	0.107 = 0.021 + 0.064 + 0.022	DStlS44-1_O
794	L6X6X7/8	0.447 = 0.422 + 0.011 + 0.014	DStlS45-1_O
795	W8X31	0.09 = 0.055 + 0.026 + 0.009	DStlS45-1_O
796	W8X31	0.036 = 0.033 + 0.004 + 0	DStlS44-1_O
797	W8X31	0.017 = 0.013 + 0.004 + 0	DStlS44-1_O
798	W8X31	0.022 = 0.019 + 0.004 + 0	DStlS46-1_O
799	W8X31	0.028 = 0.025 + 0.004 + 0	DStlS46-1_O
800	W8X31	0.035 = 0.032 + 0.004 + 0	DStlS46-1_O
801	W8X31	0.037 = 0.034 + 0.004 + 0	DStlS46-1_O
802	W8X31	0.037 = 0.033 + 0.004 + 0	DStlS46-1_O
803	W8X31	0.035 = 0.031 + 0.004 + 0	DStlS46-1_O
804	W8X31	0.028 = 0.025 + 0.004 + 0	DStlS46-1_O
805	W8X31	0.022 = 0.019 + 0.004 + 0	DStlS44-1_O

806	W8X31	0.018 = 0.014 + 0.004 + 0	DStlS45-1_O
807	W8X31	0.037 = 0.033 + 0.004 + 0	DStlS45-1_O
808	W8X58	0.307 = 0.202 + 0.099 + 0.006	DStlS47-1_O
809	W8X58	0.459 = 0.373 + 0.081 + 0.004	DStlS47-1_O
810	W8X31	0.338 = 0.239 + 0.099 + 0	DStlS44-1_O
811	W8X31	0.186 = 0.003 + 0.183 + 0	DStlS44-3_O
812	W8X31	0.202 = 0.073 + 0.129 + 0	DStlS47-1_O
813	W8X31	0.201 = 0.072 + 0.129 + 0	DStlS47-1_O
814	W8X31	0.186 = 0.003 + 0.182 + 0	DStlS45-3_O
815	W8X31	0.336 = 0.237 + 0.099 + 0	DStlS45-1_O
816	W8X58	0.429 = 0.343 + 0.081 + 0.004	DStlS45-1_O
817	W8X58	0.31 = 0.202 + 0.101 + 0.007	DStlS47-1_O
818	L8X8X1/2	0.273 = 0.063 + 0.119 + 0.091	DStlS47-1_O
819	L8X8X1/2	0.412 = 0.053 + 0.215 + 0.143	DStlS44-3_O
820	L8X8X1/2	0.565 = 0.207 + 0.215 + 0.143	DStlS47-1_O
821	L8X8X1/2	0.516 = 0.158 + 0.215 + 0.143	DStlS44-1_O
822	L8X8X1/2	0.513 = 0.154 + 0.215 + 0.143	DStlS45-1_O
823	L8X8X1/2	0.568 = 0.209 + 0.215 + 0.143	DStlS47-1_O
824	L8X8X1/2	0.409 = 0.05 + 0.215 + 0.143	DStlS46-2_O
825	L8X8X1/2	0.279 = 0.069 + 0.119 + 0.091	DStlS47-1_O
826	L8X8X1/2	0.265 = 0.055 + 0.119 + 0.091	DStlS46-1_O
827	L8X8X1/2	0.409 = 0.051 + 0.215 + 0.143	DStlS45-2_O
828	L8X8X1/2	0.565 = 0.207 + 0.215 + 0.143	DStlS46-1_O
829	L8X8X1/2	0.516 = 0.158 + 0.215 + 0.143	DStlS45-1_O
830	L8X8X1/2	0.513 = 0.154 + 0.215 + 0.143	DStlS44-1_O
831	L8X8X1/2	0.568 = 0.21 + 0.215 + 0.143	DStlS46-1_O
832	L8X8X1/2	0.409 = 0.051 + 0.215 + 0.143	DStlS44-3_O
833	L8X8X1/2	0.276 = 0.066 + 0.119 + 0.091	DStlS46-1_O
834	W8X58	0.312 = 0.204 + 0.101 + 0.008	DStlS46-1_O
835	W8X58	0.431 = 0.345 + 0.081 + 0.004	DStlS45-1_O
836	W8X31	0.338 = 0.238 + 0.099 + 0	DStlS45-1_O
837	W8X31	0.186 = 0.002 + 0.184 + 0	DStlS46-2_O
838	W8X31	0.202 = 0.073 + 0.129 + 0	DStlS46-1_O
839	W8X31	0.201 = 0.072 + 0.129 + 0	DStlS46-1_O
840	W8X31	0.185 = 0.004 + 0.182 + 0	DStlS44-3_O
841	W8X31	0.336 = 0.237 + 0.099 + 0	DStlS44-1_O
842	W8X58	0.459 = 0.376 + 0.081 + 0.002	DStlS46-1_O
843	W8X58	0.208 = 0.093 + 0.113 + 0.002	DStlS46-1_O

Hangar – All beam sections with DC ratios (Lateral Design)

Label	Section	D/C Ratio
1	W12X53	0.344 = 0.004 + 0.26 + 0.08
6	W12X45	0.61 = 0.329 + 0.015 + 0.266
12	W12X53	0.326 = 0.323 + 0.003 + 0
13	W12X53	0.37 = 0.35 + 0.003 + 0.017
14	W12X53	0.125 = 0.079 + 0.003 + 0.042
15	W12X53	0.062 = 0.007 + 0.003 + 0.051
16	W12X53	0.097 = 0.043 + 0.003 + 0.051
17	W12X53	0.136 = 0.081 + 0.003 + 0.051
18	W12X53	0.287 = 0.262 + 0.003 + 0.023
19	W12X53	0.328 = 0.302 + 0.003 + 0.023
20	W12X53	0.335 = 0.309 + 0.003 + 0.023
21	W12X53	0.331 = 0.305 + 0.003 + 0.023
22	W12X53	0.331 = 0.305 + 0.003 + 0.023
23	W12X53	0.335 = 0.31 + 0.003 + 0.023
24	W12X53	0.329 = 0.303 + 0.003 + 0.023
25	W12X53	0.289 = 0.263 + 0.003 + 0.023
26	W12X53	0.138 = 0.084 + 0.003 + 0.051
27	W12X53	0.1 = 0.046 + 0.003 + 0.051
28	W12X53	0.066 = 0.012 + 0.003 + 0.051
29	W12X53	0.116 = 0.061 + 0.003 + 0.051
66	W12X53	0.141 = 0.027 + 0.097 + 0.017
84	W12X53	0.343 = 0.001 + 0.259 + 0.083
89	W12X45	0.104 = 0.087 + 0.017 + 0
95	W12X53	0.327 = 0.325 + 0.003 + 0
96	W12X53	0.364 = 0.34 + 0.003 + 0.021
97	W12X53	0.082 = 0.079 + 0.003 + 0
98	W12X53	0.033 = 0.03 + 0.003 + 0
99	W12X53	0.215 = 0.212 + 0.003 + 0
100	W12X53	0.26 = 0.257 + 0.003 + 0
101	W12X53	0.279 = 0.277 + 0.003 + 0
102	W12X53	0.304 = 0.301 + 0.003 + 0
103	W12X53	0.335 = 0.332 + 0.003 + 0
104	W12X53	0.352 = 0.349 + 0.003 + 0
105	W12X53	0.353 = 0.35 + 0.003 + 0
106	W12X53	0.336 = 0.333 + 0.003 + 0
107	W12X53	0.304 = 0.301 + 0.003 + 0
108	W12X53	0.28 = 0.277 + 0.003 + 0
109	W12X53	0.261 = 0.258 + 0.003 + 0
110	W12X53	0.217 = 0.214 + 0.003 + 0

111	W12X53	0.03 = 0.027 + 0.003 + 0
112	W12X53	0.081 = 0.077 + 0.003 + 0
148	W12X53	0.345 = 0.004 + 0.261 + 0.081
149	W12X53	0.304 = 0.03 + 0.269 + 0.005
167	W12X53	0.34 = 0.002 + 0.26 + 0.079
171	W12X45	0.096 = 0.079 + 0.017 + 0
172	W12X45	0.092 = 0.075 + 0.017 + 0
178	W12X45	0.666 = 0.384 + 0.015 + 0.266
179	W12X53	0.323 = 0.32 + 0.003 + 0
180	W12X53	0.084 = 0.08 + 0.003 + 0
181	W12X53	0.035 = 0.032 + 0.003 + 0
182	W12X53	0.206 = 0.203 + 0.003 + 0
183	W12X53	0.255 = 0.252 + 0.003 + 0
184	W12X53	0.274 = 0.271 + 0.003 + 0
185	W12X53	0.298 = 0.296 + 0.003 + 0
186	W12X53	0.332 = 0.329 + 0.003 + 0
187	W12X53	0.346 = 0.343 + 0.003 + 0
188	W12X53	0.349 = 0.346 + 0.003 + 0
189	W12X53	0.336 = 0.333 + 0.003 + 0
190	W12X53	0.302 = 0.299 + 0.003 + 0
191	W12X53	0.278 = 0.275 + 0.003 + 0
192	W12X53	0.26 = 0.257 + 0.003 + 0
193	W12X53	0.211 = 0.208 + 0.003 + 0
194	W12X53	0.029 = 0.026 + 0.003 + 0
195	W12X53	0.082 = 0.079 + 0.003 + 0
230	W12X53	0.309 = 0.034 + 0.27 + 0.004
232	W12X53	0.306 = 0.034 + 0.268 + 0.004
233	W12X53	0.213 = 0.014 + 0.096 + 0.103
254	W12X45	0.622 = 0.387 + 0.015 + 0.22
255	W12X45	0.609 = 0.328 + 0.015 + 0.266
262	W12X53	0.331 = 0.329 + 0.003 + 0
263	W12X53	0.375 = 0.355 + 0.003 + 0.017
264	W12X53	0.127 = 0.081 + 0.003 + 0.042
265	W12X53	0.063 = 0.009 + 0.003 + 0.051
266	W12X53	0.086 = 0.031 + 0.003 + 0.051
267	W12X53	0.116 = 0.062 + 0.003 + 0.051
268	W12X53	0.274 = 0.252 + 0.003 + 0.019
269	W12X53	0.314 = 0.292 + 0.003 + 0.019
270	W12X53	0.319 = 0.297 + 0.003 + 0.019
271	W12X53	0.319 = 0.297 + 0.003 + 0.019
272	W12X53	0.32 = 0.298 + 0.003 + 0.019
273	W12X53	0.32 = 0.299 + 0.003 + 0.019

274	W12X53	0.317 = 0.296 + 0.003 + 0.019
275	W12X53	0.278 = 0.257 + 0.003 + 0.019
276	W12X53	0.122 = 0.076 + 0.003 + 0.042
277	W12X53	0.089 = 0.043 + 0.003 + 0.042
278	W12X53	0.065 = 0.01 + 0.003 + 0.051
313	W12X53	0.131 = 0.025 + 0.091 + 0.015
314	W12X53	0.215 = 0.017 + 0.096 + 0.102
315	W12X53	0.133 = 0.023 + 0.094 + 0.015
341	W12X53	0.127 = 0.072 + 0.003 + 0.051
342	W12X53	0.351 = 0.327 + 0.003 + 0.021
343	W12X53	0.139 = 0.029 + 0.093 + 0.017
346	W12X72	0.361 = 0.351 + 0.006 + 0.004
347	W12X72	0.594 = 0.554 + 0.04 + 1.933E-04
348	W12X72	0.591 = 0.551 + 0.04 + 1.989E-04
349	W12X72	0.359 = 0.349 + 0.006 + 0.004
350	W12X72	0.101 = 0.076 + 0.025 + 0
352	W12X72	0.623 = 0.58 + 0.043 + 0
353	W12X72	0.615 = 0.572 + 0.043 + 0
354	W12X72	0.095 = 0.07 + 0.025 + 2.439E-04
355	W12X72	0.327 = 0.315 + 0.007 + 0.005
356	W12X72	0.594 = 0.554 + 0.04 + 1.808E-04
357	W12X72	0.591 = 0.55 + 0.041 + 2.018E-04
358	W12X72	0.323 = 0.309 + 0.007 + 0.006
359	W12X45	0.101 = 0.084 + 0.017 + 0
362	W12X53	0.304 = 0.03 + 0.269 + 0.004
416	W12X87	0.656 = 0.366 + 0.289 + 0.001
418	W12X87	0.656 = 0.367 + 0.288 + 0.001
420	W12X87	0.649 = 0.366 + 0.283 + 0
423	W12X87	0.647 = 0.367 + 0.28 + 2.541E-04
425	W12X87	0.656 = 0.37 + 0.286 + 0.001
427	W12X87	0.654 = 0.369 + 0.284 + 0.001
429	W12X53	0.221 = 0.017 + 0.096 + 0.108
431	W12X53	0.23 = 0.02 + 0.098 + 0.113
546	W8X40	0.079 = 0.039 + 0.003 + 0.038
547	W8X31	0.506 = 0.055 + 0.026 + 0.424
548	W8X40	0.555 = 0.249 + 0.02 + 0.287
549	W8X40	0.556 = 0.25 + 0.02 + 0.287
550	W8X31	0.507 = 0.056 + 0.026 + 0.424
551	W8X40	0.079 = 0.044 + 0.003 + 0.033
552	W8X40	0.023 = 0.017 + 0.003 + 0.003
553	W8X31	0.13 = 0.036 + 0.028 + 0.066
554	W8X40	0.117 = 0.043 + 0.024 + 0.05

555	W8X40	0.118 = 0.043 + 0.024 + 0.05
556	W8X31	0.13 = 0.036 + 0.028 + 0.066
557	W8X40	0.024 = 0.015 + 0.003 + 0.006
558	W8X48	0.022 = 0.019 + 0.003 + 0
559	W8X48	0.79 = 0.765 + 0.024 + 0
560	W8X48	0.655 = 0.631 + 0.024 + 0
561	W8X48	0.656 = 0.631 + 0.024 + 0
562	W8X48	0.791 = 0.767 + 0.024 + 0
563	W8X48	0.022 = 0.018 + 0.003 + 0
564	W8X48	0.049 = 0.046 + 0.003 + 0
565	W8X48	0.261 = 0.237 + 0.024 + 0
566	W8X48	0.67 = 0.646 + 0.024 + 0
567	W8X48	0.67 = 0.646 + 0.024 + 0
568	W8X48	0.261 = 0.237 + 0.024 + 0
569	W8X48	0.049 = 0.046 + 0.003 + 0
588	W8X40	0.008 = 0.004 + 0.003 + 0
589	W8X40	0.007 = 0.003 + 0.003 + 0
590	W8X40	0.008 = 0.005 + 0.003 + 0
591	W8X40	0.007 = 0.003 + 0.003 + 0
594	W8X31	0.107 = 0.055 + 0.003 + 0.05
595	W8X31	0.493 = 0.041 + 0.027 + 0.424
596	W8X40	0.554 = 0.248 + 0.02 + 0.287
597	W8X40	0.552 = 0.245 + 0.02 + 0.287
598	W8X31	0.475 = 0.022 + 0.029 + 0.424
599	W8X31	0.118 = 0.066 + 0.003 + 0.05
600	W8X31	0.025 = 0.015 + 0.003 + 0.008
601	W8X31	0.128 = 0.034 + 0.028 + 0.066
602	W8X40	0.112 = 0.038 + 0.024 + 0.05
603	W8X40	0.122 = 0.048 + 0.024 + 0.05
604	W8X31	0.133 = 0.039 + 0.028 + 0.066
605	W8X31	0.028 = 0.018 + 0.003 + 0.008
630	W8X31	0.32 = 0.318 + 0.002 + 0
631	W8X31	0.288 = 0.259 + 0.029 + 0
632	W8X31	0.694 = 0.664 + 0.03 + 0
633	W8X31	0.696 = 0.666 + 0.03 + 0
634	W8X31	0.29 = 0.261 + 0.029 + 0
635	W8X31	0.277 = 0.274 + 0.002 + 0
636	W8X31	0.197 = 0.027 + 0.156 + 0.014
637	W8X31	0.426 = 0.024 + 0.363 + 0.038
638	W8X31	0.333 = 0.058 + 0.249 + 0.025
639	W8X31	0.332 = 0.058 + 0.249 + 0.025
640	W8X31	0.426 = 0.024 + 0.363 + 0.039

641	W8X31	0.233 = 0.031 + 0.164 + 0.038
642	W8X48	0.036 = 0.033 + 0.003 + 0
643	W8X48	0.714 = 0.69 + 0.024 + 0
644	W8X48	0.772 = 0.747 + 0.024 + 0
645	W8X48	0.769 = 0.745 + 0.024 + 0
646	W8X48	0.716 = 0.691 + 0.024 + 0
647	W8X48	0.036 = 0.033 + 0.003 + 0
648	W8X48	0.05 = 0.046 + 0.003 + 0
649	W8X48	0.26 = 0.236 + 0.024 + 0
666	W8X31	0.318 = 0.316 + 0.002 + 0
667	W8X31	0.238 = 0.21 + 0.029 + 0
668	W8X31	0.706 = 0.675 + 0.03 + 0
669	W8X31	0.681 = 0.65 + 0.03 + 0
670	W8X31	0.127 = 0.095 + 0.032 + 0
671	W8X31	0.265 = 0.262 + 0.002 + 0
672	W8X31	0.202 = 0.023 + 0.159 + 0.02
673	W8X31	0.409 = 0.01 + 0.361 + 0.038
674	W8X31	0.332 = 0.06 + 0.247 + 0.025
675	W8X31	0.333 = 0.06 + 0.249 + 0.025
676	W8X31	0.421 = 0.017 + 0.364 + 0.039
677	W8X31	0.248 = 0.033 + 0.166 + 0.049
694	W8X48	0.659 = 0.635 + 0.024 + 0
695	W8X48	0.659 = 0.635 + 0.024 + 0
696	W8X48	0.26 = 0.236 + 0.024 + 0
697	W8X48	0.05 = 0.046 + 0.003 + 0
698	W8X48	0.023 = 0.02 + 0.003 + 0
699	W8X48	0.777 = 0.752 + 0.024 + 0
700	W8X48	0.644 = 0.62 + 0.024 + 0
701	W8X48	0.639 = 0.614 + 0.024 + 0
702	W8X48	0.77 = 0.746 + 0.024 + 0
703	W8X48	0.022 = 0.019 + 0.003 + 0
704	W8X31	0.049 = 0.014 + 0.026 + 0.01
705	L6X6X7/8	0.178 = 0.132 + 0.044 + 0.002
706	L6X6X7/8	0.535 = 0.487 + 0.024 + 0.024
707	L6X6X7/8	0.186 = 0.081 + 0.076 + 0.028
708	L6X6X5/16	0.278 = 0.235 + 0.001 + 0.041
709	L6X6X5/16	0.345 = 0.25 + 0.034 + 0.061
710	L6X6X5/16	0.12 = 0.087 + 0.017 + 0.017
711	L6X6X5/16	0.214 = 0.14 + 0.014 + 0.06
712	L6X6X5/16	0.221 = 0.175 + 0.034 + 0.012
713	L6X6X5/16	0.203 = 0.153 + 0.045 + 0.004
714	L6X6X5/16	0.202 = 0.153 + 0.044 + 0.005

715	L6X6X5/16	0.222 = 0.176 + 0.034 + 0.012
716	L6X6X5/16	0.214 = 0.14 + 0.014 + 0.06
717	L6X6X5/16	0.099 = 0.065 + 0.017 + 0.017
718	L6X6X5/16	0.345 = 0.25 + 0.034 + 0.061
719	L6X6X5/16	0.273 = 0.23 + 0.002 + 0.041
720	L6X6X7/8	0.186 = 0.082 + 0.077 + 0.028
721	L6X6X7/8	0.517 = 0.469 + 0.025 + 0.023
722	L6X6X7/8	0.187 = 0.14 + 0.044 + 0.002
723	W8X31	0.052 = 0.022 + 0.023 + 0.006
724	W8X31	0.118 = 0.083 + 0.026 + 0.009
725	L6X6X7/8	0.601 = 0.557 + 0.008 + 0.036
726	L6X6X7/8	0.152 = 0.038 + 0.079 + 0.035
727	L6X6X7/8	0.378 = 0.337 + 0.026 + 0.015
728	L6X6X5/16	0.342 = 0.228 + 0.05 + 0.064
729	L6X6X5/16	0.14 = 0.098 + 0.01 + 0.032
730	L6X6X5/16	0.354 = 0.271 + 0.022 + 0.061
731	L6X6X5/16	0.21 = 0.169 + 0.025 + 0.016
732	L6X6X5/16	0.198 = 0.139 + 0.008 + 0.052
733	L6X6X5/16	0.239 = 0.196 + 0.04 + 0.003
734	L6X6X5/16	0.239 = 0.196 + 0.039 + 0.004
735	L6X6X5/16	0.199 = 0.139 + 0.008 + 0.052
736	L6X6X5/16	0.208 = 0.167 + 0.013 + 0.027
737	L6X6X5/16	0.354 = 0.271 + 0.022 + 0.061
738	L6X6X5/16	0.146 = 0.104 + 0.01 + 0.032
739	L6X6X5/16	0.341 = 0.228 + 0.05 + 0.063
740	L6X6X7/8	0.399 = 0.358 + 0.025 + 0.016
741	L6X6X7/8	0.15 = 0.037 + 0.078 + 0.035
742	L6X6X7/8	0.624 = 0.578 + 0.009 + 0.037
743	W8X31	0.122 = 0.088 + 0.023 + 0.01
744	W8X31	0.049 = 0.046 + 0.003 + 0
745	W8X31	0.023 = 0.02 + 0.003 + 0
746	W8X31	0.033 = 0.03 + 0.003 + 0
747	W8X31	0.042 = 0.039 + 0.003 + 0
748	W8X31	0.052 = 0.048 + 0.003 + 0
749	W8X31	0.055 = 0.051 + 0.003 + 0
750	W8X31	0.055 = 0.051 + 0.003 + 0
751	W8X31	0.052 = 0.048 + 0.003 + 0
752	W8X31	0.041 = 0.038 + 0.003 + 0
753	W8X31	0.033 = 0.029 + 0.003 + 0
754	W8X31	0.024 = 0.02 + 0.003 + 0
755	W8X31	0.051 = 0.048 + 0.003 + 0
756	W8X31	0.052 = 0.016 + 0.027 + 0.009

757	L6X6X7/8	0.197 = 0.096 + 0.064 + 0.037
758	L6X6X7/8	0.546 = 0.495 + 0.03 + 0.021
759	L6X6X7/8	0.189 = 0.081 + 0.083 + 0.024
760	L6X6X5/16	0.232 = 0.191 + 0.018 + 0.024
761	L6X6X5/16	0.338 = 0.241 + 0.027 + 0.07
762	L6X6X5/16	0.124 = 0.091 + 0.015 + 0.018
763	L6X6X5/16	0.193 = 0.119 + 0.015 + 0.059
764	L6X6X5/16	0.216 = 0.167 + 0.011 + 0.038
765	L6X6X5/16	0.208 = 0.154 + 0.004 + 0.05
766	L6X6X5/16	0.206 = 0.152 + 0.005 + 0.05
767	L6X6X5/16	0.221 = 0.171 + 0.011 + 0.039
768	L6X6X5/16	0.192 = 0.118 + 0.014 + 0.059
769	L6X6X5/16	0.122 = 0.091 + 0.013 + 0.017
770	L6X6X5/16	0.337 = 0.24 + 0.026 + 0.07
771	L6X6X5/16	0.213 = 0.17 + 0.019 + 0.024
772	L6X6X7/8	0.188 = 0.08 + 0.084 + 0.024
773	L6X6X7/8	0.506 = 0.454 + 0.031 + 0.021
774	L6X6X7/8	0.204 = 0.1 + 0.065 + 0.038
775	W8X31	0.051 = 0.022 + 0.024 + 0.006
776	W8X31	0.119 = 0.085 + 0.026 + 0.009
777	L6X6X7/8	0.604 = 0.546 + 0.045 + 0.013
778	L6X6X7/8	0.153 = 0.036 + 0.084 + 0.033
779	L6X6X7/8	0.37 = 0.325 + 0.03 + 0.015
780	L6X6X5/16	0.315 = 0.2 + 0.038 + 0.078
781	L6X6X5/16	0.136 = 0.097 + 0.029 + 0.01
782	L6X6X5/16	0.354 = 0.27 + 0.021 + 0.063
783	L6X6X5/16	0.212 = 0.17 + 0.014 + 0.028
784	L6X6X5/16	0.195 = 0.136 + 0.004 + 0.055
785	L6X6X5/16	0.221 = 0.172 + 0.003 + 0.045
786	L6X6X5/16	0.217 = 0.169 + 0.004 + 0.044
787	L6X6X5/16	0.198 = 0.137 + 0.005 + 0.055
788	L6X6X5/16	0.207 = 0.165 + 0.014 + 0.028
789	L6X6X5/16	0.355 = 0.272 + 0.021 + 0.063
790	L6X6X5/16	0.149 = 0.11 + 0.029 + 0.011
791	L6X6X5/16	0.315 = 0.2 + 0.038 + 0.077
792	L6X6X7/8	0.413 = 0.367 + 0.03 + 0.016
793	L6X6X7/8	0.153 = 0.037 + 0.082 + 0.034
794	L6X6X7/8	0.643 = 0.583 + 0.046 + 0.015
795	W8X31	0.122 = 0.089 + 0.023 + 0.01
796	W8X31	0.051 = 0.048 + 0.003 + 0
797	W8X31	0.023 = 0.019 + 0.003 + 0
798	W8X31	0.033 = 0.03 + 0.003 + 0

799	W8X31	0.042 = 0.039 + 0.003 + 0
800	W8X31	0.052 = 0.049 + 0.003 + 0
801	W8X31	0.055 = 0.051 + 0.003 + 0
802	W8X31	0.054 = 0.051 + 0.003 + 0
803	W8X31	0.051 = 0.048 + 0.003 + 0
804	W8X31	0.041 = 0.038 + 0.003 + 0
805	W8X31	0.032 = 0.029 + 0.003 + 0
806	W8X31	0.024 = 0.021 + 0.003 + 0
807	W8X31	0.052 = 0.049 + 0.003 + 0
808	W8X58	0.431 = 0.299 + 0.124 + 0.008
809	W8X58	0.709 = 0.559 + 0.145 + 0.005
810	W8X31	0.467 = 0.391 + 0.076 + 0
811	W8X31	0.268 = 0.098 + 0.17 + 0
812	W8X31	0.321 = 0.229 + 0.091 + 0
813	W8X31	0.322 = 0.23 + 0.091 + 0
814	W8X31	0.323 = 0.294 + 0.029 + 0
815	W8X31	0.467 = 0.391 + 0.076 + 0
816	W8X58	0.669 = 0.517 + 0.145 + 0.007
817	W8X58	0.437 = 0.3 + 0.126 + 0.01
818	L8X8X1/2	0.773 = 0.626 + 0.081 + 0.066
819	L8X8X1/2	0.542 = 0.291 + 0.147 + 0.104
820	L8X8X1/2	0.67 = 0.344 + 0.196 + 0.13
821	L8X8X1/2	0.567 = 0.241 + 0.196 + 0.13
822	L8X8X1/2	0.564 = 0.238 + 0.196 + 0.13
823	L8X8X1/2	0.674 = 0.347 + 0.196 + 0.13
824	L8X8X1/2	0.685 = 0.435 + 0.147 + 0.104
825	L8X8X1/2	0.705 = 0.558 + 0.081 + 0.066
826	L8X8X1/2	0.43 = 0.283 + 0.081 + 0.066
827	L8X8X1/2	0.784 = 0.534 + 0.147 + 0.104
828	L8X8X1/2	0.647 = 0.321 + 0.196 + 0.13
829	L8X8X1/2	0.572 = 0.245 + 0.196 + 0.13
830	L8X8X1/2	0.568 = 0.242 + 0.196 + 0.13
831	L8X8X1/2	0.656 = 0.33 + 0.196 + 0.13
832	L8X8X1/2	0.892 = 0.641 + 0.147 + 0.104
833	L8X8X1/2	0.472 = 0.325 + 0.081 + 0.066
834	W8X58	0.419 = 0.282 + 0.128 + 0.009
835	W8X58	0.668 = 0.513 + 0.149 + 0.006
836	W8X31	0.463 = 0.387 + 0.076 + 0
837	W8X31	0.333 = 0.305 + 0.029 + 0
838	W8X31	0.32 = 0.228 + 0.091 + 0
839	W8X31	0.315 = 0.223 + 0.092 + 0
840	W8X31	0.249 = 0.079 + 0.17 + 0

841	W8X31	0.451 = 0.374 + 0.077 + 0
842	W8X58	0.679 = 0.532 + 0.145 + 0.003
843	W8X58	0.38 = 0.252 + 0.126 + 0.002
844	W8X31	0.371 = 0.006 + 0.316 + 0.05
845	W8X31	0.372 = 0.043 + 0.324 + 0.005
846	W8X31	0.436 = 0.08 + 0.327 + 0.029
847	W8X31	0.436 = 0.081 + 0.327 + 0.028
848	W8X31	0.375 = 0.045 + 0.324 + 0.006
849	W8X31	0.371 = 0.006 + 0.315 + 0.05
850	W8X31	0.16 = 0.004 + 0.155 + 3.824E-04
851	W8X31	0.492 = 0.006 + 0.485 + 0
852	W8X31	0.201 = 0.007 + 0.193 + 1.12E-04
853	W8X31	0.203 = 0.005 + 0.198 + 0
854	W8X31	0.491 = 0.006 + 0.485 + 1.239E-04
855	W8X31	0.16 = 0.003 + 0.156 + 0.001
856	W8X31	0.367 = 0.006 + 0.314 + 0.047
857	W8X31	0.373 = 0.044 + 0.323 + 0.006
858	W8X31	0.436 = 0.08 + 0.326 + 0.029
859	W8X31	0.435 = 0.081 + 0.326 + 0.028
860	W8X31	0.374 = 0.045 + 0.323 + 0.006
861	W8X31	0.366 = 0.006 + 0.312 + 0.048
862	W8X31	0.022 = 0.001 + 0.002 + 0.019
863	W8X31	0.03 = 0.002 + 0.005 + 0.023
864	W8X31	0.017 = 0.012 + 0.003 + 0.002
865	W8X31	0.022 = 0.003 + 0.001 + 0.018
866	W8X31	0.032 = 0.001 + 0.001 + 0.029
867	W8X31	0.039 = 0.001 + 0.002 + 0.036
868	W8X31	0.045 = 0.003 + 0.004 + 0.039
869	W8X31	0.028 = 0.002 + 0.003 + 0.023
870	W8X31	0.023 = 0.002 + 3.997E-04 + 0.021
871	W8X31	0.018 = 0.012 + 0.003 + 0.002
872	W8X31	0.022 = 0.003 + 0.001 + 0.018
873	W8X31	0.033 = 0.002 + 0.001 + 0.03
874	W8X31	0.04 = 0.001 + 0.002 + 0.037
875	W8X31	0.047 = 0.003 + 0.003 + 0.041
876	W8X31	0.021 = 0 + 0.002 + 0.019
877	W8X31	0.029 = 0.002 + 0.005 + 0.023
878	W8X31	0.04 = 0.001 + 0.002 + 0.037
879	W8X31	0.033 = 0.002 + 0.001 + 0.03
880	W8X31	0.022 = 0.003 + 0.001 + 0.018
881	W8X31	0.018 = 0.011 + 0.001 + 0.006
882	W8X31	0.017 = 0.012 + 0.003 + 0.002

883	W8X31	0.022 = 0.003 + 0.001 + 0.019
884	W8X31	0.033 = 0.001 + 0.001 + 0.03
885	W8X31	0.04 = 0.001 + 0.002 + 0.036
886	W8X31	0.046 = 0.003 + 0.004 + 0.04
887	W8X31	0.047 = 0.003 + 0.003 + 0.041
888	W8X31	0.027 = 0.002 + 0.005 + 0.021
889	W8X31	0.018 = 1.181E-04 + 0.002 + 0.016

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ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	B1	178	12.12424	DStlS7-1	Special Concentrically Braced Frame	W12X45	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
22.30000	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.101	0.305	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.091	0.000061	0.016782	0.002411	0.0643	0.0281

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.033287	0.007189	0.037153	0.010995	0.42951	0.16281	0.000556

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	21.43333	0.42951	49.902	200	49.902	300
Minor (2-2)	1	21.43333	0.16281	131.65	200	131.65	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
12.12424	-65.995	3.0463	-21.3558	0.059	-0.341	-0.0521

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.961	1	1	1	1	1

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	0.961	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.961	1	1.136

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.668 =	0.386 + 0.015 + 0.266

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
65.995	170.753	589.5

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	3.0463	178.4817	240.75	157.0639
Minor Bending	21.3558	71.25		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.059	121.605	4.835E-04
Minor Shear	0.341	249.953	0.001

End Reaction Major Shear Forces

Left End Reaction (kip)	Load Combo	Right End Reaction (kip)	Load Combo
0.659	DStlS1	0.679	DStlS1

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ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	B181	352	98.35	DStlS47-3	Special Concentrically Braced Frame	W12X72	Seismic MD

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
98.35000	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
-0.279	-0.815	1	0.8	0.8

Seismic Parameters

Ignore Seismic Code?	Ignore Special EQ Load?	Plug Welded?	SDC	I	Rho	S_{DS}	R	Ω_0	C_d
No	No	Yes	D	1.25	1	0.59	6	2	5

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.1465	0.000141	0.028791	0.009404	0.1117	0.0367

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.056177	0.018808	0.0625	0.028472	0.44327	0.25333	0.002185

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	98.35	0.44327	221.876	200	221.876	300
Minor (2-2)	1	32.78333	0.25333	129.407	200	129.407	300

Stress Check Message - Section is not seismically compact for highly ductile members (AISC 341-16 F2.5a, D1.1, Table D1.1)

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
98.35	293.994	0	0	1.473	0.006	-0.0001

Axial Force & Biaxial Moment Design Factors (H1.2,H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	1	1	1	1	1
Minor Bending	0.333	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.333	1	1.249

Demand/Capacity (D/C) Ratio Eqn.(H1.2,H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.31 =	0.31 + 0 + 0

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
293.994	96.828	949.5

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	0	351.2678	405	281.179
Minor Bending	0	184.5		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	1.473	158.67	0.009
Minor Shear	0.006	434.16	1.301E-05

Hangar - Beam 420

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story33	B193	420	120.73491	DStlS47-3	Special Concentrically Braced Frame	W12X87	Seismic MD

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
121.06371	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.212	0.083	1	0.8	0.8

Seismic Parameters

Ignore Seismic Code?	Ignore Special EQ Load?	Plug Welded?	SDC	I	Rho	S_{DS}	R	Ω_0	C_d
No	No	Yes	D	1.25	1	0.59	6	2	5

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.1778	0.000246	0.035687	0.011622	0.1361	0.0447

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.068519	0.023052	0.076389	0.034954	0.44804	0.25569	0.002736

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	11.18823	0.44804	24.972	200	24.972	300
Minor (2-2)	1	12.10637	0.25569	47.349	200	47.349	300

Stress Check Message - Section is not seismically compact for highly ductile members (AISC 341-16 F2.5a, D1.1, Table D1.1)

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
120.73491	-271.754	-35.4919	-0.0387	8.5	-0.023	0.0026

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.092	1	1	1	1	1
Minor Bending	0.1	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.1	1	1.295

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.342 =	0.278 + 0.064 + 1.521E-04

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
271.754	977.828	1152

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	35.4919	495	495	487.6874
Minor Bending	0.0387	226.5		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	8.5	193.125	0.044
Minor Shear	0.023	529.254	4.387E-05

End Reaction Major Shear Forces

Left End Reaction (kip)	Load Combo	Right End Reaction (kip)	Load Combo
		16.54	DStlS21-3

Hangar - Beam 668

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	B126	668	24.6	DStlS47-3	Special Concentrically Braced Frame	W8X31	Non-Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
24.60000	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.096	0.36	1	0.8	0.8

Seismic Parameters

Ignore Seismic Code?	Ignore Special EQ Load?	Plug Welded?	SDC	I	Rho	S_{DS}	R	Ω_0	C_d
No	No	Yes	D	1.25	1	0.59	6	2	5

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.0634	0.000026	0.005305	0.001789	0.0483	0.0158

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.015914	0.005367	0.017593	0.00816	0.28925	0.16798	0.000178

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	24.6	0.28925	85.046	200	85.046	300
Minor (2-2)	1	24.6	0.16798	146.442	200	146.442	300

Stress Check Message - Section is not seismically compact for highly ductile members (AISC 341-16 F2.5a, D1.1, Table D1.1)

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
24.6	-43.812	0	0	0.489	-0.017	-0.002

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	1	1	1	1	1
Minor Bending	1	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
1	1	1.136

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.456 =	0.456 + 0 + 0

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
43.812	96.179	410.85

Moments and Capacities

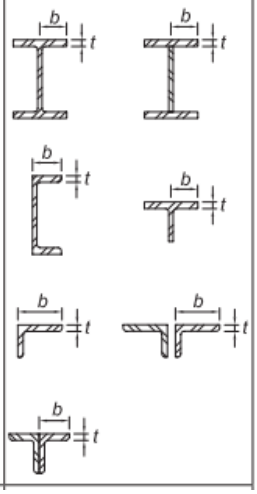
	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	0	82.5191	113.8774	72.6168
Minor Bending	0	52.7913		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.489	68.4	0.007
Minor Shear	0.017	187.92	8.82E-05

Highly Ductile Requirement and Unbraced Lengths from AISC 341 for SCBF Beams:

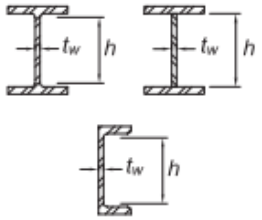
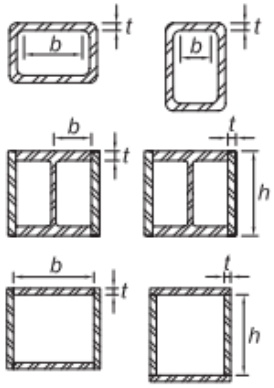
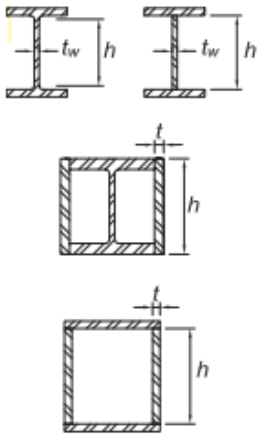
TABLE D1.1
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees; legs of single angles or double-angle members with separators; outstanding legs of pairs of angles in continuous contact	b/t	$0.32\sqrt{\frac{E}{R_y F_y}}$	$0.40\sqrt{\frac{E}{R_y F_y}}$	

W8x40: b/t (=7.2) ≤ $0.32\sqrt{\frac{29000}{1.1\times50}}$ = 7.35

W12x45: b/t (=7) ≤ $0.32\sqrt{\frac{29000}{1.1\times50}}$ = 7.35

TABLE D1.1 (continued)
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Stiffened Elements	Webs of rolled or built-up I shaped sections and channels used as diagonal braces	h/t_w	$1.57\sqrt{\frac{E}{R_yF_y}}$	$1.57\sqrt{\frac{E}{R_yF_y}}$	
	Where used in beams or columns as flanges in uniform compression due to axial, flexure, or combined axial and flexure: 1) Walls of rectangular HSS 2) Flanges and side plates of boxed I-shaped sections, webs and flanges of built-up box shapes	b/t h/t	$0.65\sqrt{\frac{E}{R_yF_y}}$	$1.18\sqrt{\frac{E}{R_yF_y}}$	
	Where used in beams, columns, or links, as webs in flexure, or combined axial and flexure: 1) Webs of rolled or built-up I-shaped sections or channels [b] 2) Side plates of boxed I-shaped sections 3) Webs of built-up box sections	h/t_w h/t h/t	For $C_a \le 0.114$: $2.57\sqrt{\frac{E}{R_yF_y}}(1-1.04C_a)$ For $C_a > 0.114$: $0.88\sqrt{\frac{E}{R_yF_y}}(2.68-C_a) \ge 1.57\sqrt{\frac{E}{R_yF_y}}$ where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD), $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD), $P_y = R_yF_yA_g$	For $C_a \le 0.114$: $3.96\sqrt{\frac{E}{R_yF_y}}(1-3.04C_a)$ For $C_a > 0.114$: $1.29\sqrt{\frac{E}{R_yF_y}}(2.12-C_a) \ge 1.57\sqrt{\frac{E}{R_yF_y}}$ where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD), $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD), $P_y = R_yF_yA_g$	

W8x40: $C_a = \frac{40}{0.9\times1.1\times50\times11.7} = 0.07 \leq 0.114$

h/t_w (=23) $\leq 2.57\sqrt{\frac{29000}{1.1\times50}}(1-1.04\times0.07) = 55$

W12x45: $C_a = \frac{40}{0.9\times1.1\times50\times13.1} = 0.06 \leq 0.114$

h/t_w (=36) $\leq 2.57\sqrt{\frac{29000}{1.1\times50}}(1-1.04\times0.06) = 55$

Highly Ductile Members

In addition to the requirements of Sections D1.2a.1(a) and (b), and D1.2a.2(a) and (b), the bracing of highly ductile beam members shall have a maximum spacing of $L_b = 0.095 r_y E/(R_y F_y)$. For concrete-encased composite beams, the material properties of the steel section shall be used and the calculation for r_y in the plane of buckling shall be based on the elastic transformed section.

W8x40: $L_b = 0.095 \times 2.04 \times \frac{29000}{1.1 \times 50} = 8.5'$

W12x45: $L_b = 0.095 \times 1.95 \times \frac{29000}{1.1 \times 50} = 8.1'$

Hangar

SCBF Beam Section	Frame	Label	Actual Length (ft)	Calculated L_b Length (ft)	Bracing Required (brace size /spacing of brace)	P_{br} (kips)	$\phi_c P_n$ (kips)	β_{br} (K/in)	K_{brace} (K/in)
W12x45	A.1	64	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.1	160	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.1	343	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W12x45	A.1	21	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.1	172	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.1	66	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W12x45	A.3	88	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.3	158	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.3	362	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W12x45	A.3	231	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.3	68	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	A.3	149	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W12x45	D.7	250	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.7	71	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.7	230	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W12x45	D.7	337	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.7	89	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.7	232	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W12x45	D.9	409	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.9	112	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.9	313	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W12x45	D.9	338	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.9	150	22	8.1	L4x4x1/4 / 7.3'	6.5	28	45	483
W12x45	D.9	315	22	8.1	W8x31 / 7.3'	6.5	283	45	1471
W8x40	1.1	549	24	8.5	L4x4x1/4 / 8'	6	28	38	405
W8x40	1.1	555	24	8.5	L4x4x1/4 / 8'	6	28	38	405
W8x40	1.1	341	8.2	8.5	-	-	-	-	-
W8x40	1.1	332	8.2	8.5	-	-	-	-	-

W8x40	1.1	546	8.2	8.5	-	-	-	-	-
W8x40	1.1	552	8.2	8.5	-	-	-	-	-
W8x40	1.1	344	8.2	8.5	-	-	-	-	-
W8x40	1.1	342	8.2	8.5	-	-	-	-	-
W8x40	1.1	551	8.2	8.5	-	-	-	-	-
W8x40	1.1	557	8.2	8.5	-	-	-	-	-
W8x40	3.9	589	8.2	8.5	-	-	-	-	-
W8x40	3.9	588	8.2	8.5	-	-	-	-	-
W8x40	3.9	594	8.2	8.5	-	-	-	-	-
W8x40	3.9	600	8.2	8.5	-	-	-	-	-
W8x40	3.9	591	8.2	8.5	-	-	-	-	-
W8x40	3.9	590	8.2	8.5	-	-	-	-	-
W8x40	3.9	599	8.2	8.5	-	-	-	-	-
W8x40	3.9	605	8.2	8.5	-	-	-	-	-
W8x40	3.9	596	24	8.5	L4x4x1/4 / 8’	6	28	38	405
W8x40	3.9	602	24	8.5	L4x4x1/4 / 8’	6	28	38	405

Hangar – All column sections with DC ratios (Lateral Design)

Label	Section	D/C Ratio
2	W10X68	0.09
3	W10X68	0.1
4	W10X68	0.074
5	W10X68	0.07
7	W8X31	0.011
8	W10X68	0.842
9	W10X68	0.767
10	W10X68	0.912
11	W10X68	0.772
30	W8X31	0.095
31	W8X31	0.087
32	W8X31	0.074
33	W8X31	0.07
34	W8X31	0.058
35	W8X31	0.045
36	W8X31	0.031
37	W8X31	0.017
38	W8X31	0.016
39	W8X31	0.03
40	W8X31	0.044
41	W8X31	0.057
42	W8X31	0.069
43	W8X31	0.073
44	W8X31	0.087
45	W8X31	0.094
85	W10X68	0.123
86	W10X68	0.133
87	W10X68	0.134
90	W8X31	0.015
91	W10X68	0.462
92	W10X68	0.432
93	W10X68	0.457
94	W10X68	0.427
113	W8X31	0.554
114	W8X31	0.08
115	W8X31	0.772
116	W8X31	0.352
117	W8X31	0.286
118	W8X31	0.215

119	W8X31	0.076
120	W8X31	0.046
121	W8X31	0.045
122	W8X31	0.075
123	W8X31	0.212
124	W8X31	0.283
125	W8X31	0.349
126	W8X31	0.437
127	W8X31	0.515
128	W8X31	0.55
147	W10X68	0.274
168	W10X68	0.117
169	W10X68	0.127
170	W10X68	0.137
173	W8X31	0.354
174	W10X68	0.457
175	W10X68	0.412
176	W10X68	0.481
177	W10X68	0.406
196	W8X31	0.555
197	W8X31	0.52
198	W8X31	0.42
199	W8X31	0.353
200	W8X31	0.287
201	W8X31	0.216
202	W8X31	0.077
203	W8X31	0.047
204	W8X31	0.044
205	W8X31	0.073
206	W8X31	0.21
207	W8X31	0.28
208	W8X31	0.346
209	W8X31	0.435
210	W8X31	0.513
211	W8X31	0.548
251	W10X68	0.098
252	W10X68	0.1
253	W10X68	0.084
256	W8X31	0.011
257	W10X68	0.87
258	W10X68	0.752
259	W10X68	0.919

260	W10X68	0.756
279	W8X31	0.095
280	W8X31	0.087
281	W8X31	0.074
282	W8X31	0.069
283	W8X31	0.057
284	W8X31	0.044
285	W8X31	0.031
286	W8X31	0.017
287	W8X31	0.016
288	W8X31	0.03
289	W8X31	0.044
290	W8X31	0.057
291	W8X31	0.069
292	W8X31	0.073
293	W8X31	0.087
294	W8X31	0.094
333	W12X136	0.743
334	W12X136	0.343
335	W12X136	0.745
336	W12X136	0.348
339	W8X31	0.624
344	W10X68	0.282
351	W8X31	0.627
361	W10X68	0.071
363	W8X31	0.762
364	W8X31	0.549
365	W8X31	0.446
366	W8X31	0.278
367	W8X31	0.205
368	W8X31	0.057
369	W8X31	0.019
370	W8X31	0.046
371	W8X31	0.08
372	W8X31	0.353
373	W8X31	0.046
374	W8X31	0.018
375	W8X31	0.056
376	W8X31	0.203
377	W8X31	0.202
378	W8X31	0.277
379	W8X31	0.448

380	W8X31	0.55
381	W8X31	0.627
404	W10X88	0.721
405	W10X88	0.457
406	W10X88	0.868
407	W10X88	0.351
410	W8X31	0.463
422	W8X31	0.461
434	W8X31	0.527
435	W8X31	0.482
436	W8X31	0.391
437	W8X31	0.252
438	W8X31	0.213
439	W8X31	0.06
440	W8X31	0.015
441	W8X31	0.04
442	W8X31	0.072
443	W8X31	0.26
444	W8X31	0.072
445	W8X31	0.039
446	W8X31	0.015
447	W8X31	0.061
448	W8X31	0.215
449	W8X31	0.253
450	W8X31	0.395
451	W8X31	0.485
452	W8X31	0.527
475	W12X136	0.678
476	W12X136	0.347
477	W12X136	0.721
478	W12X136	0.35
481	W8X31	0.629
493	W8X31	0.626
505	W8X31	0.766
506	W8X31	0.553
507	W8X31	0.451
508	W8X31	0.278
509	W8X31	0.203
510	W8X31	0.056
511	W8X31	0.019
512	W8X31	0.046
513	W8X31	0.08

514	W8X31	0.354
515	W8X31	0.079
516	W8X31	0.045
517	W8X31	0.018
518	W8X31	0.057
519	W8X31	0.206
520	W8X31	0.28
521	W8X31	0.452
522	W8X31	0.554
523	W8X31	0.764

Hangar - Column 8

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	C5	8	32.99167	DStlS6-1	Special Concentrically Braced Frame	W10X68	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
34.00000	0.837	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.061	0.248	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.1382	0.000172	0.019001	0.006462	0.108	0.0339

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.043848	0.015356	0.049363	0.023206	0.3708	0.21624	0.001027

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	32.99167	0.3708	88.974	200	88.974	300
Minor (2-2)	1	32.99167	0.21624	152.567	200	152.567	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
32.99167	-60.654	3.747	87.354	-0.102	-16.725	0.161

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.97	1	1	1	1	0.6

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	0.97	1	1	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.97	1	1.584

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.841 =	0.314 + 0.01 + 0.516

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
60.654	193.141	895.5

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	3.747	319.875	319.875	227.7926
Minor Bending	87.354	150.375		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.102	146.64	0.001
Minor Shear	16.725	419.958	0.04

Hangar - Column 147

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	C3	147	0	DStlS45-3	Special Concentrically Braced Frame	W10X68	Seismic HD

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
34.00000	0.531	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.038	0.154	1	0.8	0.8

Seismic Parameters

Ignore Seismic Code?	Ignore Special EQ Load?	Plug Welded?	SDC	I	Rho	S_{DS}	R	Ω_0	C_d
No	No	Yes	D	1.25	1	0.59	6	2	5

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.1382	0.000172	0.019001	0.006462	0.108	0.0339

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.043848	0.015356	0.049363	0.023206	0.3708	0.21624	0.001027

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	32.99167	0.3708	88.974	200	88.974	300
Minor (2-2)	1	32.99167	0.21624	152.567	200	152.567	300

(ASCE 2.3.6-6, 12.4.2.2, 12.4.3.1: (1.2+0.2*Sds)*D + 1.0*L - Omega0*Qe)

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
0	-52.978	0	0	-0.214	-0.041	0

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.97	1	1	1	1	0.6
Minor Bending	0.97	1	1	1	1	0.6

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.97	1	1.629

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.274 =	0.274 + 0 + 0

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
52.978	193.141	895.5

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	0	319.875	319.875	227.7926
Minor Bending	0	150.375		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.214	146.64	0.001
Minor Shear	0.041	419.958	9.785E-05

Hangar - Column 176

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	C52	176	32.99167	DStlS21-1	Special Concentrically Braced Frame	W10X68	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
34.00000	0.607	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.083	0.338	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.1382	0.000172	0.019001	0.006462	0.108	0.0339

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.043848	0.015356	0.049363	0.023206	0.3708	0.21624	0.001027

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	32.99167	0.3708	88.974	200	88.974	300
Minor (2-2)	1	32.99167	0.21624	152.567	200	152.567	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
32.99167	-82.78	-10.419	3.6029	0.273	-0.098	0.0051

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.97	1	1	1	1	0.6

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	0.97	1	1	1	1	0.6

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.97	1	1.572

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.479 =	0.429 + 0.029 + 0.021

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
82.78	193.141	895.5

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	10.419	319.875	319.875	227.7926
Minor Bending	3.6029	150.375		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.273	146.64	0.002
Minor Shear	0.098	419.958	2.323E-04

Hangar - Column 406

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	C171	406	32.975	DStlS21-1	Special Concentrically Braced Frame	W10X88	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
34.00000	0.418	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.13	0.515	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.1806	0.000363	0.025752	0.008632	0.1416	0.0454

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.057227	0.020114	0.065394	0.030729	0.37766	0.21865	0.001453

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.1	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	1	32.975	0.37766	87.314	200	87.314	300
Minor (2-2)	1	32.975	0.21865	150.809	200	150.809	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
32.975	-168.381	-101.4835	-0.6494	4.75	2.602E-04	0.0523

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	0.97	1	1	1	1	1

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	0.97	1	1	1	1	0.6

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
0.97	1	1.809

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.868 =	0.652 + 0.213 + 0.003

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
168.381	258.262	1170

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	101.4835	423.75	423.75	330.6082
Minor Bending	0.6494	199.125		

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	4.75	196.02	0.024
Minor Shear	2.602E-04	550.638	0

Hangar – All brace sections with DC ratios (Lateral Design)

Label	Section	D/C Ratio
46	W8X31	0.207
47	W8X31	0.092
48	W8X31	0.074
49	W8X31	0.064
50	W8X31	0.051
51	W8X31	0.04
52	W8X31	0.029
53	W8X31	0.021
54	W8X31	0.026
55	W8X31	0.025
56	W8X31	0.02
57	W8X31	0.029
58	W8X31	0.039
59	W8X31	0.051
60	W8X31	0.064
61	W8X31	0.073
62	W8X31	0.092
63	W8X31	0.2
68	HSS9X9X5/8-1	0.714
69	HSS9X9X5/8-1	0.703
70	HSS5X5X1/2-1	0.637
71	HSS5X5X1/2-1	0.615
72	HSS9X9X5/8-1	0.074
73	HSS9X9X5/8-1	0.072
74	HSS5X5X1/2-1	0.076
75	HSS5X5X1/2-1	0.082
76	HSS9X9X5/8-1	0.058
77	HSS9X9X5/8-1	0.061
78	HSS5X5X1/2-1	0.092
79	HSS5X5X1/2-1	0.096
80	HSS5X5X1/2-1	0.713
81	HSS5X5X1/2-1	0.694
82	HSS9X9X5/8-1	0.698
83	HSS9X9X5/8-1	0.7
129	W8X31	0.584
130	W8X31	0.5
131	W8X31	0.395
132	W8X31	0.287
133	W8X31	0.215

134	W8X31	0.081
135	W8X31	0.056
136	W8X31	0.036
137	W8X31	0.066
138	W8X31	0.069
139	W8X31	0.035
140	W8X31	0.055
141	W8X31	0.08
142	W8X31	0.213
143	W8X31	0.284
144	W8X31	0.391
145	W8X31	0.497
146	W8X31	0.58
151	HSS9X9X5/8-1	0.884
152	HSS9X9X5/8-1	0.878
153	HSS7X7X1/2	0.826
154	HSS7X7X1/2	0.81
155	HSS9X9X5/8-1	0.094
156	HSS9X9X5/8-1	0.09
157	HSS7X7X1/2	0.293
158	HSS7X7X1/2	0.297
159	HSS9X9X5/8-1	0.098
160	HSS9X9X5/8-1	0.096
161	HSS7X7X1/2	0.279
162	HSS7X7X1/2	0.284
163	HSS7X7X1/2	0.801
164	HSS7X7X1/2	0.784
165	HSS9X9X5/8-1	0.853
166	HSS9X9X5/8-1	0.852
212	W8X31	0.585
213	W8X31	0.501
214	W8X31	0.396
215	W8X31	0.288
216	W8X31	0.216
217	W8X31	0.082
218	W8X31	0.056
219	W8X31	0.036
220	W8X31	0.069
221	W8X31	0.07
222	W8X31	0.034
223	W8X31	0.054
224	W8X31	0.079

225	W8X31	0.212
226	W8X31	0.282
227	W8X31	0.39
228	W8X31	0.495
229	W8X31	0.578
234	HSS9X9X5/8-1	0.896
235	HSS9X9X5/8-1	0.891
236	HSS7X7X1/2	0.825
237	HSS7X7X1/2	0.81
238	HSS9X9X5/8-1	0.095
239	HSS9X9X5/8-1	0.092
240	HSS7X7X1/2	0.293
241	HSS7X7X1/2	0.297
242	HSS9X9X5/8-1	0.088
243	HSS9X9X5/8-1	0.087
244	HSS7X7X1/2	0.274
245	HSS7X7X1/2	0.28
246	HSS7X7X1/2	0.798
247	HSS7X7X1/2	0.783
248	HSS9X9X5/8-1	0.828
249	HSS9X9X5/8-1	0.828
261	HSS4X4X3/8-1	0.256
295	W8X31	0.207
296	W8X31	0.092
297	W8X31	0.074
298	W8X31	0.064
299	W8X31	0.051
300	W8X31	0.04
301	W8X31	0.029
302	W8X31	0.021
303	W8X31	0.026
304	W8X31	0.024
305	W8X31	0.02
306	W8X31	0.029
307	W8X31	0.039
308	W8X31	0.051
309	W8X31	0.064
310	W8X31	0.073
311	W8X31	0.091
312	W8X31	0.2
317	HSS9X9X5/8-1	0.73
318	HSS9X9X5/8-1	0.719

319	HSS5X5X1/2-1	0.608
320	HSS5X5X1/2-1	0.597
321	HSS9X9X5/8-1	0.076
322	HSS9X9X5/8-1	0.072
323	HSS5X5X1/2-1	0.075
324	HSS5X5X1/2-1	0.079
325	HSS9X9X5/8-1	0.061
326	HSS9X9X5/8-1	0.058
327	HSS5X5X1/2-1	0.09
328	HSS5X5X1/2-1	0.092
329	HSS5X5X1/2-1	0.68
330	HSS5X5X1/2-1	0.662
331	HSS9X9X5/8-1	0.655
332	HSS9X9X5/8-1	0.647
340	HSS4X4X3/8-1	0.24
345	HSS4X4X3/8-1	0.288
360	HSS4X4X3/8-1	0.295
382	W8X35	0.843
383	W8X31	0.817
384	W8X31	0.548
385	W8X31	0.413
386	W8X31	0.29
387	W8X31	0.086
388	W8X31	0.048
389	W8X31	0.048
390	W8X31	0.298
391	W8X31	0.563
392	W8X31	0.721
393	W8X31	0.718
394	W8X31	0.561
395	W8X31	0.296
396	W8X31	0.046
397	W8X31	0.047
398	W8X31	0.085
399	W8X31	0.291
400	W8X31	0.415
401	W8X31	0.55
402	W8X31	0.827
403	W8X35	0.854
411	HSS9X9X5/8-1	0.298
412	HSS9X9X5/8-1	0.296
413	HSS9X9X5/8-1	0.485

414	HSS9X9X5/8-1	0.482
453	W8X31	0.654
454	W8X31	0.567
455	W8X31	0.482
456	W8X31	0.36
457	W8X31	0.247
458	W8X31	0.088
459	W8X31	0.05
460	W8X31	0.035
461	W8X31	0.255
462	W8X31	0.508
463	W8X31	0.68
464	W8X31	0.677
465	W8X31	0.505
466	W8X31	0.252
467	W8X31	0.034
468	W8X31	0.051
469	W8X31	0.089
470	W8X31	0.25
471	W8X31	0.363
472	W8X31	0.485
473	W8X31	0.567
474	W8X31	0.654
490	L8X8X1/2	0.563
491	L8X8X1/2	0.553
492	L8X8X1/2	0.564
494	L8X8X1/2	0.553
503	L8X8X1/2	0.157
504	L8X8X1/2	0.145
524	W8X35	0.847
525	W8X31	0.82
526	W8X31	0.553
527	W8X31	0.418
528	W8X31	0.293
529	W8X31	0.085
530	W8X31	0.047
531	W8X31	0.048
532	W8X31	0.298
533	W8X31	0.563
534	W8X31	0.721
535	W8X31	0.715
536	W8X31	0.556

537	W8X31	0.291
538	W8X31	0.044
539	W8X31	0.048
540	W8X31	0.086
541	W8X31	0.295
542	W8X31	0.419
543	W8X31	0.554
544	W8X31	0.818
545	W8X35	0.845
570	HSS4X4X3/8-1	0.266
571	HSS4X4X3/8-1	0.259
572	HSS4X4X3/8-1	0.429
573	HSS4X4X3/8-1	0.424
574	HSS4X4X3/8-1	0.273
575	HSS4X4X3/8-1	0.278
580	HSS4X4X3/8-1	0.304
581	HSS4X4X3/8-1	0.285
582	HSS4X4X3/8-1	0.429
583	HSS4X4X3/8-1	0.428
584	HSS4X4X3/8-1	0.261
585	HSS4X4X3/8-1	0.255
586	HSS9X9X5/8-1	0.563
587	HSS9X9X5/8-1	0.56
592	HSS9X9X5/8-1	0.304
593	HSS9X9X5/8-1	0.301
606	HSS4X4X3/8-1	0.259
607	HSS4X4X3/8-1	0.253
608	HSS4X4X3/8-1	0.449
609	HSS4X4X3/8-1	0.439
610	HSS4X4X3/8-1	0.222
611	HSS4X4X3/8-1	0.224
612	HSS4X4X3/8-1	0.107
613	HSS4X4X3/8-1	0.12
614	HSS4X4X3/8-1	0.08
615	HSS4X4X3/8-1	0.083
616	HSS4X4X3/8-1	0.078
617	HSS4X4X3/8-1	0.076
618	HSS4X4X3/8-1	0.518
619	HSS4X4X3/8-1	0.516
620	HSS4X4X3/8-1	0.29
621	HSS4X4X3/8-1	0.283
622	HSS9X9X5/8-1	0.564

623	HSS9X9X5/8-1	0.571
624	HSS4X4X3/8-1	0.112
625	HSS4X4X3/8-1	0.112
626	HSS4X4X3/8-1	0.089
627	HSS4X4X3/8-1	0.091
628	HSS9X9X5/8-1	0.459
629	HSS9X9X5/8-1	0.456
650	HSS4X4X3/8-1	0.883
651	HSS4X4X3/8-1	0.883
652	HSS4X4X3/8-1	0.282
653	HSS4X4X3/8-1	0.262
654	HSS5X5X1/2-1	0.817
655	HSS5X5X1/2-1	0.802
656	HSS5X5X1/2-1	0.821
657	HSS5X5X1/2-1	0.806
658	HSS4X4X3/8-1	0.283
659	HSS4X4X3/8-1	0.264
660	HSS4X4X3/8-1	0.886
661	HSS4X4X3/8-1	0.885
662	HSS5X5X1/2-1	0.07
663	HSS5X5X1/2-1	0.074
664	HSS5X5X1/2-1	0.071
665	HSS5X5X1/2-1	0.075
678	HSS4X4X3/8-1	0.888
679	HSS4X4X3/8-1	0.887
680	HSS4X4X3/8-1	0.283
681	HSS4X4X3/8-1	0.263
682	HSS5X5X1/2-1	0.86
683	HSS5X5X1/2-1	0.846
684	HSS5X5X1/2-1	0.795
685	HSS5X5X1/2-1	0.781
686	HSS4X4X3/8-1	0.266
687	HSS4X4X3/8-1	0.246
688	HSS4X4X3/8-1	0.854
689	HSS4X4X3/8-1	0.854
690	HSS5X5X1/2-1	0.093
691	HSS5X5X1/2-1	0.098
692	HSS5X5X1/2-1	0.053
693	HSS5X5X1/2-1	0.056
890	L8X8X1/2	0.157
891	L8X8X1/2	0.144
892	HSS4X4X3/8-1	0.102

893	HSS4X4X3/8-1	0.103
894	HSS4X4X3/8-1	0.065
895	HSS4X4X3/8-1	0.073
896	HSS4X4X3/8-1	0.224
897	HSS4X4X3/8-1	0.213
898	HSS4X4X3/8-1	0.082
899	HSS4X4X3/8-1	0.081
900	L8X8X1/2	0.113
901	L8X8X1/2	0.081
902	L8X8X1/2	0.104
903	L8X8X1/2	0.071
904	HSS4X4X3/8-1	0.127
905	HSS4X4X3/8-1	0.124
906	HSS4X4X3/8-1	0.1
907	HSS4X4X3/8-1	0.103
908	HSS4X4X3/8-1	0.101
909	HSS4X4X3/8-1	0.1
910	HSS4X4X3/8-1	0.079
911	HSS4X4X3/8-1	0.077
912	L8X8X1/2	0.157
913	L8X8X1/2	0.124
914	L8X8X1/2	0.153
915	L8X8X1/2	0.12
924	L8X8X1/2	0.551
925	L8X8X1/2	0.541
926	L8X8X1/2	0.554
927	L8X8X1/2	0.545
936	L8X8X1/2	0.151
937	L8X8X1/2	0.139
938	L8X8X1/2	0.147
939	L8X8X1/2	0.135

Hangar - Brace 80

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story22	D37	80	13.14354	DStlS23-1	Special Concentrically Braced Frame	HSS5X5X1/2-1	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
13.14354	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.152	0.542	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.0547	0.002151	0.001254	0.001254	0.0233	0.0233

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.006019	0.006019	0.007581	0.007581	0.15137	0.15137	Not required

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.3	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	0.9	13.14354	0.15137	78.147	200	86.83	300
Minor (2-2)	0.9	24.04766	0.15137	142.979	200	158.866	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
13.14354	-59.851	-0.7644	-0.4399	0.233	0.001	1.6827

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	0.9	0.9	1	1	1

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	1.83	0.9	0.9	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
1.83	1	2.36

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.709 =	0.687 + 0.014 + 0.008

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
59.851	87.08	354.6

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	0.7644	49.125	49.125	49.125
Minor Bending	0.4399	49.125		

Torsion Moment and Capacities

T_u Moment (kip-ft)	T_n Capacity (kip-ft)	ϕT_n Capacity (kip-ft)
1.6827	46.8455	42.161

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.233	90.522	0.003
Minor Shear	0.001	90.522	1.584E-05

End Reaction Axial Forces

Left End Reaction (kip)	Load Combo	Right End Reaction (kip)	Load Combo
-60.009	DStlS23-1	-60.009	DStlS23-1

Hangar - Brace 151

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story11	D53	151	20.33033	DStlS21-1	Special Concentrically Braced Frame	HSS9X9X5/8-1	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
20.33033	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.233	0.68	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.1299	0.017168	0.010417	0.010417	0.0586	0.0586

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.027778	0.027778	0.033623	0.033623	0.28322	0.28322	Not required

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.3	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	0.9	20.33033	0.28322	64.604	200	71.783	300
Minor (2-2)	0.9	40.66067	0.28322	129.209	200	143.565	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
20.33033	-218.068	-2.0884	-3.3891	0.448	0.113	-0.0069

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	0.9	0.9	1	1	1

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	2	0.9	0.9	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
2	1	2.029

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.884 =	0.862 + 0.009 + 0.014

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
218.068	253.045	841.5

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	2.0884	217.875	217.875	217.875
Minor Bending	3.3891	217.875		

Torsion Moment and Capacities

T_u Moment (kip-ft)	T_n Capacity (kip-ft)	ϕT_n Capacity (kip-ft)
-0.0069	204.0111	183.61

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.448	227.681	0.002
Minor Shear	0.113	227.681	4.978E-04

End Reaction Axial Forces

Left End Reaction (kip)	Load Combo	Right End Reaction (kip)	Load Combo
-219.366	DStlS21-1	-219.366	DStlS21-1

Hangar - Brace 153

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story22	D55	153	13.14352	DStlS21-1	Special Concentrically Braced Frame	HSS7X7X1/2	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
13.14352	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.354	0.601	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.0806	0.006414	0.003882	0.003882	0.0362	0.0362

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.01331	0.01331	0.016146	0.016146	0.21953	0.21953	Not required

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.3	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	0.9	13.14352	0.21953	53.885	200	59.872	300
Minor (2-2)	0.9	24.04766	0.21953	98.589	200	109.543	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
13.14352	-205.181	0.0902	3.1246	0.254	-0.017	1.9775

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	0.9	0.9	1	1	1

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	1.83	0.9	0.9	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
1.83	1	1.299

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.827 =	0.8 + 0.001 + 0.027

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
205.181	256.463	522

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	0.0902	104.625	104.625	104.625
Minor Bending	3.1246	104.625		

Torsion Moment and Capacities

T_u Moment (kip-ft)	T_n Capacity (kip-ft)	ϕT_n Capacity (kip-ft)
1.9775	98.321	88.4889

Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.254	140.742	0.002
Minor Shear	0.017	140.742	1.187E-04

End Reaction Axial Forces

Left End Reaction (kip)	Load Combo	Right End Reaction (kip)	Load Combo
-205.414	DStlS21-1	-205.414	DStlS21-1

Hangar - Brace 650

ETABS Steel Frame Design

AISC 360-16 Steel Section Check (Strength Summary)

Element Details

Level	Element	Unique Name	Location (ft)	Combo	Element Type	Section	Classification
Story22	D267	650	7.07983	DStlS21-1	Special Concentrically Braced Frame	HSS4X4X3/8-1	Compact

LLRF and Demand/Capacity Ratio

L (ft)	LLRF	Stress Ratio Limit
7.07983	1	0.95

Analysis and Design Parameters

Provision	Analysis	2nd Order	Reduction
LRFD	Direct Analysis	General 2nd Order	Tau-b Fixed

Stiffness Reduction Factors

$\alpha P_r / P_y$	$\alpha P_r / P_e$	τ_b	EA factor	EI factor
0.352	0.668	1	0.8	0.8

Design Code Parameters

ϕ_b	ϕ_c	ϕ_{TY}	ϕ_{TF}	ϕ_V	$\phi_{V\text{-}RI}$	ϕ_{VT}
0.9	0.9	0.9	0.75	0.9	1	1

Section Properties

A (ft^2)	J (ft^4)	I_{33} (ft^4)	I_{22} (ft^4)	A_{v3} (ft^2)	A_{v2} (ft^2)
0.0332	0.000844	0.000497	0.000497	0.0143	0.0143

Design Properties

S_{33} (ft^3)	S_{22} (ft^3)	Z_{33} (ft^3)	Z_{22} (ft^3)	r_{33} (ft)	r_{22} (ft)	C_w (ft^6)
0.00298	0.00298	0.003698	0.003698	0.12233	0.12233	Not required

Material Properties

E (kip/ft^2)	f_y (kip/ft^2)	R_y	C_{pr}	α
4176000	7200	1.3	1.4	NA

Slenderness Check

Buckling Mode	K Factor	L Length (ft)	r (ft)	KL/r	KL/r Limit	L/r	L/r Limit
Major (3-3)	0.9	7.07983	0.12233	52.089	200	57.876	300
Minor (2-2)	0.9	14.15966	0.12233	104.177	200	115.752	300

Stress Check Forces and Moments

Location (ft)	P_u (kip)	M_{u33} (kip-ft)	M_{u22} (kip-ft)	V_{u2} (kip)	V_{u3} (kip)	T_u (kip-ft)
7.07983	-84.206	0.1915	0.2651	0.009	-0.066	0.5821

Axial Force & Biaxial Moment Design Factors (H1-1a)

	L Factor	K_1	K_2	B_1	B_2	C_m
Major Bending	1	0.9	0.9	1	1	1

	L Factor	K_1	K_2	B_1	B_2	C_m
Minor Bending	2	0.9	0.9	1	1	1

Parameters for Lateral Torsion Buckling

L_{ltb}	K_{ltb}	C_b
2	1	1.146

Demand/Capacity (D/C) Ratio Eqn.(H1-1a)

D/C Ratio =	$(P_r/P_c) + (8/9)(M_{r33}/M_{c33}) + (8/9)(M_{r22}/M_{c22})$
0.883 =	0.866 + 0.007 + 0.01

Axial Force and Capacities

P_u Force (kip)	ϕP_{nc} Capacity (kip)	ϕP_{nt} Capacity (kip)
84.206	97.278	215.1

Moments and Capacities

	M_u Moment (kip-ft)	ϕM_n (kip-ft)	ϕM_n No LTB (kip-ft)	ϕM_n Cb=1 (kip-ft)
Major Bending	0.1915	23.9625	23.9625	23.9625
Minor Bending	0.2651	23.9625		

Torsion Moment and Capacities

T_u Moment (kip-ft)	T_n Capacity (kip-ft)	ϕT_n Capacity (kip-ft)
0.5821	22.85	20.565

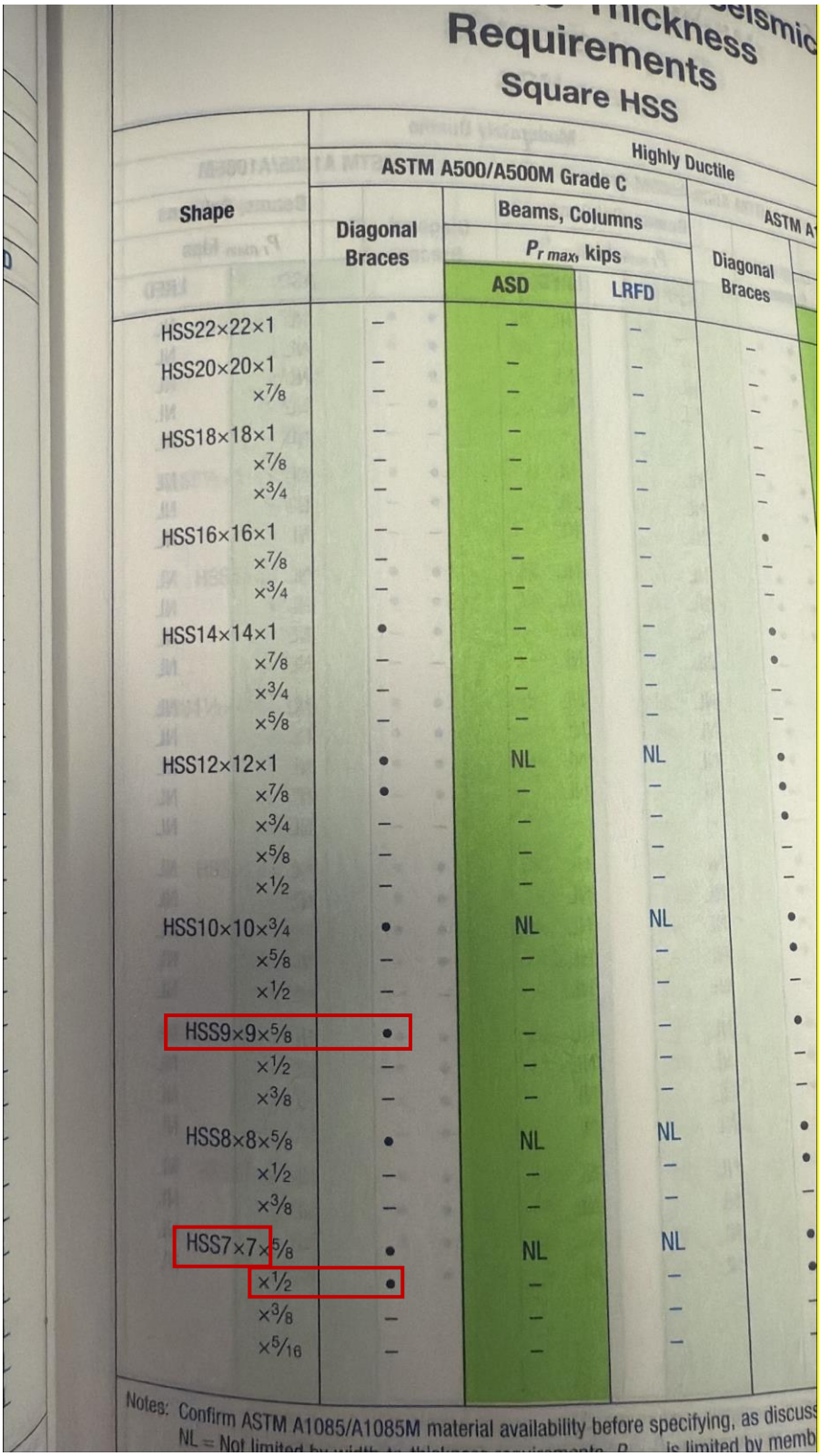
Shear Design

	V_u Force (kip)	ϕV_n Capacity (kip)	Stress Ratio
Major Shear	0.009	55.652	1.618E-04
Minor Shear	0.066	55.652	0.001

End Reaction Axial Forces

Left End Reaction (kip)	Load Combo	Right End Reaction (kip)	Load Combo
-84.312	DStlS21-1	-84.312	DStlS21-1

Highly Ductile Requirement and Unbraced Lengths from AISC 341 for SCBF Braces:



Thickness Requirements

Square HSS

Shape	Highly Ductile			
	ASTM A500/A500M Grade C			ASTM A
	Diagonal Braces	Beams, Columns $P_{r\,max}$, kips		Diagonal Braces
		ASD	LRFD	
HSS22×22×1	–	–	–	–
HSS20×20×1	–	–	–	–
×7/8	–	–	–	–
HSS18×18×1	–	–	–	–
×7/8	–	–	–	–
×3/4	–	–	–	–
HSS16×16×1	–	–	–	•
×7/8	–	–	–	–
×3/4	–	–	–	–
HSS14×14×1	•	–	–	•
×7/8	–	–	–	•
×3/4	–	–	–	–
×5/8	–	–	–	–
HSS12×12×1	•	NL	NL	•
×7/8	•	–	–	•
×3/4	–	–	–	•
×5/8	–	–	–	–
×1/2	–	–	–	–
HSS10×10×3/4	•	NL	NL	•
×5/8	–	–	–	•
×1/2	–	–	–	–
HSS9×9×5/8	•	–	–	•
×1/2	–	–	–	–
×3/8	–	–	–	–
HSS8×8×5/8	•	NL	NL	•
×1/2	–	–	–	•
×3/8	–	–	–	–
HSS7×7×5/8	•	NL	NL	•
×1/2	•	–	–	•
×3/8	–	–	–	–
×5/16	–	–	–	–

Notes: Confirm ASTM A1085/A1085M material availability before specifying, as discuss

NL = Not limited by width-to-thickness requirements. P_r is limited by memb

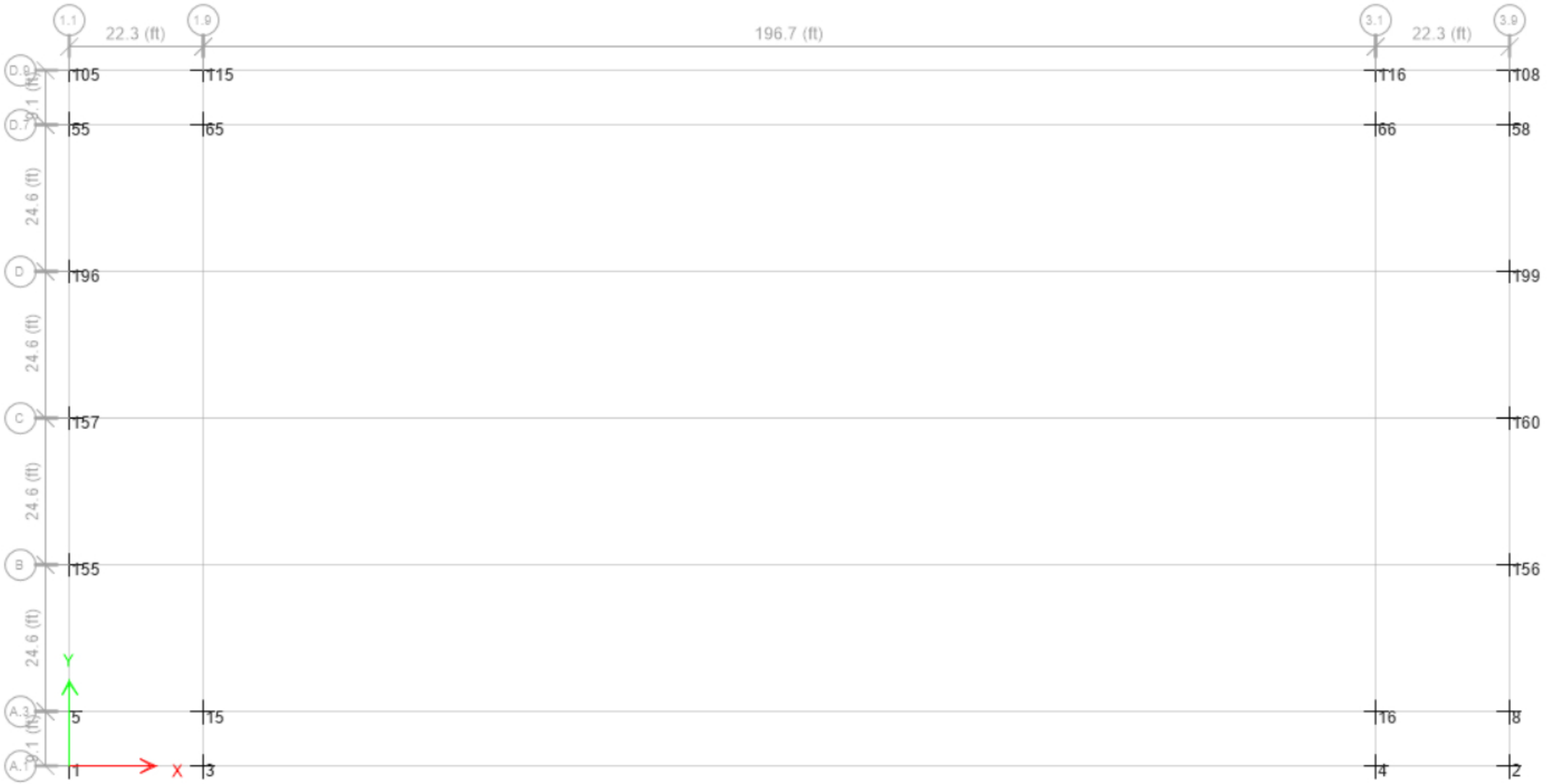
...that Satisfy Seismic Width-to-Thickness Requirements

Square HSS

Shape	Highly Ductile				
	ASTM A500/A500M Grade C			ASTM A1085	
	Diagonal Braces	Beams, Columns $P_{r\,max}$, kips		Diagonal Braces	Beams
		ASD	LRFD		
HSS6×6×5/8	•	NL	NL	•	
×1/2	•	NL	NL	•	
×3/8	–	–	–	•	
×5/16	–	–	–	•	
×1/4	–	–	–	–	
HSS5 1/2×5 1/2×3/8	•	–	–	•	
×5/16	–	–	–	–	
×1/4	–	–	–	–	
HSS5×5×1/2	•	NL	NL	•	
×3/8	•	NL	NL	•	
×5/16	–	–	–	•	
×1/4	–	–	–	–	
HSS4 1/2×4 1/2×1/2	•	NL	NL	•	
×3/8	•	NL	NL	•	
×5/16	•	–	–	•	
×1/4	–	–	–	–	
×3/16	–	–	–	–	
HSS4×4×1/2	•	NL	NL	•	
×3/8	•	NL	NL	•	
×5/16	•	NL	NL	•	
×1/4	–	–	–	•	
×3/16	–	–	–	–	
HSS3 1/2×3 1/2×3/8	•	NL	NL	•	
×5/16	•	NL	NL	•	
×1/4	•	–	–	•	
×3/16	–	–	–	–	
HSS3×3×3/8	•	NL	NL	•	
×5/16	•	NL	NL	•	
×1/4	•	NL	NL	•	
×3/16	–	–	–	•	

Notes: Confirm ASTM A1085/A1085M material availability before specifying, as discussed
NL = Not limited by width-to-thickness requirements. $P_{r\,max}$ is limited by member
• The member width-to-thickness requirements are satisfied for the given steel gr

Hangar - Base Labels



Point Results

Joint Reactions

Story	Label	Unique Name	Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip
Base	1	1	Dead	LinStatic		48.022	8.509	95.948
Base	1	1	Live	LinStatic		19.021	4.15	40.91
Base	1	1	Exx	LinStatic	Max	-9.65	-1.779	-30.177
Base	1	1	Exx	LinStatic	Min	-9.65	-1.779	-30.177
Base	1	1	Eyy	LinStatic	Max	-5.551	-6.177	-50.792
Base	1	1	Eyy	LinStatic	Min	-5.551	-6.177	-50.792
Base	1	1	Mass	LinStatic		41.598	9.023	89.214
Base	1	1	Sb	LinStatic		20.029	4.363	43.031
Base	1	1	Su1	LinStatic		12.327	2.714	27.989
Base	1	1	Su2	LinStatic		13.352	2.76	26.604
Base	1	1	Wx	LinStatic		-51.315	36.05	-45.525
Base	1	1	Wy	LinStatic		-11.979	-42.916	-96.354
Base	1	1	Wxx	LinStatic		-11.314	37.263	-13.436
Base	1	1	Wyy	LinStatic		-7.164	34.242	29.211
Base	1	1	Exx-O	LinStatic	Max	-19.3	-3.559	-60.355
Base	1	1	Exx-O	LinStatic	Min	-19.3	-3.559	-60.355
Base	1	1	Eyy-O	LinStatic	Max	-11.102	-12.354	-101.584
Base	1	1	Eyy-O	LinStatic	Min	-11.102	-12.354	-101.584
Base	2	2	Dead	LinStatic		-49.542	6.931	93.09
Base	2	2	Live	LinStatic		-19.775	3.361	39.489
Base	2	2	Exx	LinStatic	Max	-9.673	1.663	29.721
Base	2	2	Exx	LinStatic	Min	-9.673	1.663	29.721
Base	2	2	Eyy	LinStatic	Max	5.802	-5.627	-49.287
Base	2	2	Eyy	LinStatic	Min	5.802	-5.627	-49.287
Base	2	2	Mass	LinStatic		-43.239	7.306	86.124
Base	2	2	Sb	LinStatic		-20.822	3.533	41.537
Base	2	2	Su1	LinStatic		-13.857	2.193	25.452
Base	2	2	Su2	LinStatic		-12.825	2.228	27.234
Base	2	2	Wx	LinStatic		10.812	36.754	-14.43
Base	2	2	Wy	LinStatic		12.502	-41.869	-93.533
Base	2	2	Wxx	LinStatic		50.828	35.532	-46.647
Base	2	2	Wyy	LinStatic		6.85	33.317	26.884
Base	2	2	Exx-O	LinStatic	Max	-19.346	3.327	59.441
Base	2	2	Exx-O	LinStatic	Min	-19.346	3.327	59.441
Base	2	2	Eyy-O	LinStatic	Max	11.604	-11.254	-98.573
Base	2	2	Eyy-O	LinStatic	Min	11.604	-11.254	-98.573
Base	3	3	Dead	LinStatic		18.341	-0.029	40.666
Base	3	3	Live	LinStatic		8.279	-0.016	11.245
Base	3	3	Exx	LinStatic	Max	-8.665	0.001	25.621
Base	3	3	Exx	LinStatic	Min	-8.665	0.001	25.621
Base	3	3	Eyy	LinStatic	Max	3.919	0.011	-12.36
Base	3	3	Eyy	LinStatic	Min	3.919	0.011	-12.36
Base	3	3	Mass	LinStatic		18.08	-0.034	24.812
Base	3	3	Sb	LinStatic		8.717	-0.016	11.869
Base	3	3	Su1	LinStatic		6.567	-0.01	0.173
Base	3	3	Su2	LinStatic		4.852	-0.011	14.08
Base	3	3	Wx	LinStatic		-16.282	0.001	-16.491
Base	3	3	Wy	LinStatic		2.561	0.006	-60.7
Base	3	3	Wxx	LinStatic		-3.61	0.002	-50.629
Base	3	3	Wyy	LinStatic		-12.77	-0.002	-26.745
Base	3	3	Exx-O	LinStatic	Max	-17.33	0.002	51.242
Base	3	3	Exx-O	LinStatic	Min	-17.33	0.002	51.242
Base	3	3	Eyy-O	LinStatic	Max	7.838	0.022	-24.72
Base	3	3	Eyy-O	LinStatic	Min	7.838	0.022	-24.72

Story	Label	Unique Name	Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip
Base	4	4	Dead	LinStatic		-19.887	-0.032	38.369
Base	4	4	Live	LinStatic		-9.037	-0.017	10.14
Base	4	4	Exx	LinStatic	Max	-8.719	-0.001	-25.65
Base	4	4	Exx	LinStatic	Min	-8.719	-0.001	-25.65
Base	4	4	Eyy	LinStatic	Max	-3.721	0.011	-12.569
Base	4	4	Eyy	LinStatic	Min	-3.721	0.011	-12.569
Base	4	4	Mass	LinStatic		-19.73	-0.037	22.405
Base	4	4	Sb	LinStatic		-9.514	-0.018	10.706
Base	4	4	Su1	LinStatic		-5.38	-0.012	13.331
Base	4	4	Su2	LinStatic		-7.053	-0.011	-0.563
Base	4	4	Wx	LinStatic		3.036	0.002	-51.641
Base	4	4	Wy	LinStatic		-2.082	0.006	-60.72
Base	4	4	Wxx	LinStatic		15.687	0.001	-17.577
Base	4	4	Wyy	LinStatic		12.746	-0.003	-25.488
Base	4	4	Exx-O	LinStatic	Max	-17.438	-0.003	-51.301
Base	4	4	Exx-O	LinStatic	Min	-17.438	-0.003	-51.301
Base	4	4	Eyy-O	LinStatic	Max	-7.442	0.021	-25.137
Base	4	4	Eyy-O	LinStatic	Min	-7.442	0.021	-25.137
Base	6	5	Dead	LinStatic		47.834	5.888	48.369
Base	6	5	Live	LinStatic		18.926	3.039	17.279
Base	6	5	Exx	LinStatic	Max	-9.917	0.906	-27.704
Base	6	5	Exx	LinStatic	Min	-9.917	0.906	-27.704
Base	6	5	Eyy	LinStatic	Max	1.144	-5.632	38.344
Base	6	5	Eyy	LinStatic	Min	1.144	-5.632	38.344
Base	6	5	Mass	LinStatic		41.379	6.61	37.79
Base	6	5	Sb	LinStatic		19.955	3.197	18.222
Base	6	5	Su1	LinStatic		11.909	1.726	12.883
Base	6	5	Su2	LinStatic		13.454	2.325	9.86
Base	6	5	Wx	LinStatic		-34.262	6.006	-65.203
Base	6	5	Wy	LinStatic		-25.931	-9.945	31.338
Base	6	5	Wxx	LinStatic		-23.137	4.48	-33.948
Base	6	5	Wyy	LinStatic		-28.492	11.344	-90.536
Base	6	5	Exx-O	LinStatic	Max	-19.835	1.812	-55.407
Base	6	5	Exx-O	LinStatic	Min	-19.835	1.812	-55.407
Base	6	5	Eyy-O	LinStatic	Max	2.288	-11.264	76.689
Base	6	5	Eyy-O	LinStatic	Min	2.288	-11.264	76.689
Base	25	8	Dead	LinStatic		-48.481	4.066	56.823
Base	25	8	Live	LinStatic		-19.241	2.13	21.449
Base	25	8	Exx	LinStatic	Max	-9.913	-1.023	28.189
Base	25	8	Exx	LinStatic	Min	-9.913	-1.023	28.189
Base	25	8	Eyy	LinStatic	Max	-1.381	-5.072	36.734
Base	25	8	Eyy	LinStatic	Min	-1.381	-5.072	36.734
Base	25	8	Mass	LinStatic		-42.065	4.634	46.862
Base	25	8	Sb	LinStatic		-20.286	2.241	22.606
Base	25	8	Su1	LinStatic		-13.664	1.679	12.834
Base	25	8	Su2	LinStatic		-12.119	1.158	15.48
Base	25	8	Wx	LinStatic		22.881	3.878	-30.909
Base	25	8	Wy	LinStatic		25.659	-8.864	27.868
Base	25	8	Wxx	LinStatic		33.996	5.395	-62.039
Base	25	8	Wyy	LinStatic		29.035	10.455	-89.032
Base	25	8	Exx-O	LinStatic	Max	-19.827	-2.046	56.378
Base	25	8	Exx-O	LinStatic	Min	-19.827	-2.046	56.378
Base	25	8	Eyy-O	LinStatic	Max	-2.763	-10.143	73.467
Base	25	8	Eyy-O	LinStatic	Min	-2.763	-10.143	73.467
Base	26	15	Dead	LinStatic		29.903	-0.093	-3.866
Base	26	15	Live	LinStatic		13.899	-0.046	-10.779
Base	26	15	Exx	LinStatic	Max	-9.435	0.002	27.738

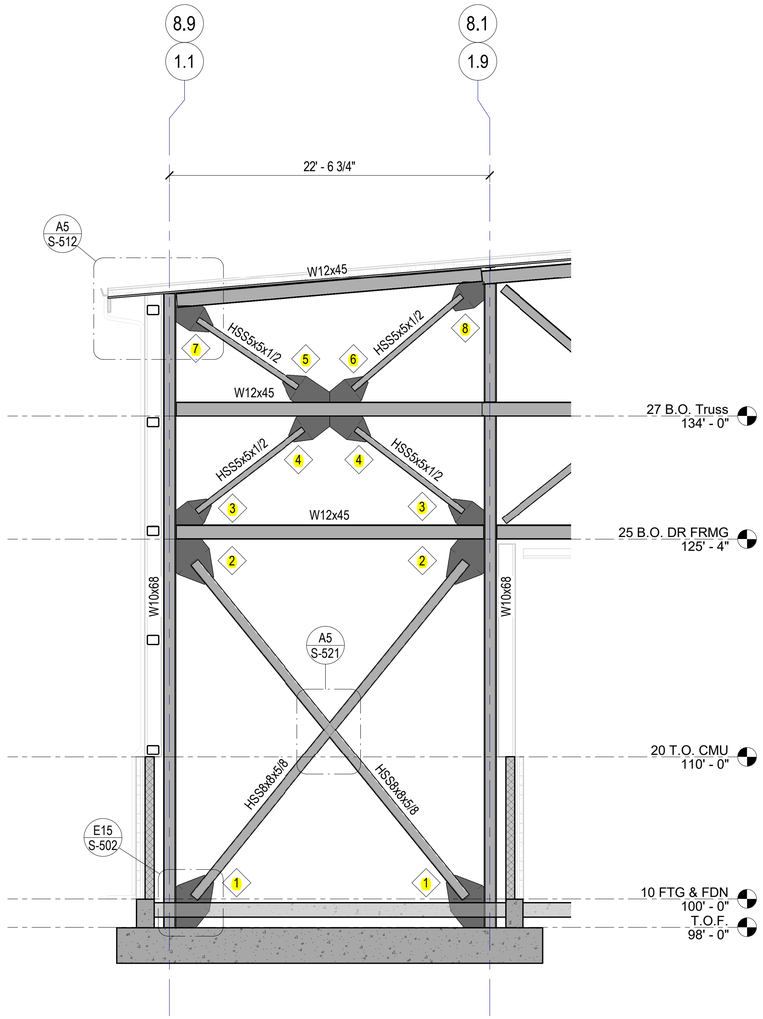
Story	Label	Unique Name	Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip
Base	26	15	Exx	LinStatic	Min	-9.435	0.002	27.738
Base	26	15	Eyy	LinStatic	Max	-7.586	-0.012	21.694
Base	26	15	Eyy	LinStatic	Min	-7.586	-0.012	21.694
Base	26	15	Mass	LinStatic		30.312	-0.1	-23.152
Base	26	15	Sb	LinStatic		14.626	-0.048	-11.217
Base	26	15	Su1	LinStatic		9.982	-0.028	-14.179
Base	26	15	Su2	LinStatic		8.982	-0.034	-1.493
Base	26	15	Wx	LinStatic		-17.045	0.01	-13.363
Base	26	15	Wy	LinStatic		-19.59	-0.026	-14.616
Base	26	15	Wxx	LinStatic		-6.636	0.011	-42.758
Base	26	15	Wyy	LinStatic		-2.449	0.039	-51.1
Base	26	15	Exx-O	LinStatic	Max	-18.87	0.004	55.476
Base	26	15	Exx-O	LinStatic	Min	-18.87	0.004	55.476
Base	26	15	Eyy-O	LinStatic	Max	-15.172	-0.025	43.388
Base	26	15	Eyy-O	LinStatic	Min	-15.172	-0.025	43.388
Base	27	16	Dead	LinStatic		-30.457	-0.102	-7.297
Base	27	16	Live	LinStatic		-14.178	-0.051	-12.499
Base	27	16	Exx	LinStatic	Max	-9.401	-0.003	-27.744
Base	27	16	Exx	LinStatic	Min	-9.401	-0.003	-27.744
Base	27	16	Eyy	LinStatic	Max	7.38	-0.012	21.933
Base	27	16	Eyy	LinStatic	Min	7.38	-0.012	21.933
Base	27	16	Mass	LinStatic		-30.921	-0.111	-26.892
Base	27	16	Sb	LinStatic		-14.92	-0.054	-13.025
Base	27	16	Su1	LinStatic		-9.149	-0.037	-2.624
Base	27	16	Su2	LinStatic		-10.182	-0.031	-15.322
Base	27	16	Wx	LinStatic		6.5	0.009	-43.722
Base	27	16	Wy	LinStatic		19.328	-0.024	-14.017
Base	27	16	Wxx	LinStatic		16.935	0.008	-14.251
Base	27	16	Wyy	LinStatic		2.692	0.037	-51.634
Base	27	16	Exx-O	LinStatic	Max	-18.803	-0.005	-55.489
Base	27	16	Exx-O	LinStatic	Min	-18.803	-0.005	-55.489
Base	27	16	Eyy-O	LinStatic	Max	14.759	-0.023	43.867
Base	27	16	Eyy-O	LinStatic	Min	14.759	-0.023	43.867
Base	87	55	Dead	LinStatic		48.472	-4.066	56.806
Base	87	55	Live	LinStatic		19.24	-2.13	21.447
Base	87	55	Exx	LinStatic	Max	-9.913	-1.023	-28.189
Base	87	55	Exx	LinStatic	Min	-9.913	-1.023	-28.189
Base	87	55	Eyy	LinStatic	Max	-1.381	-5.072	-36.734
Base	87	55	Eyy	LinStatic	Min	-1.381	-5.072	-36.734
Base	87	55	Mass	LinStatic		42.061	-4.634	46.858
Base	87	55	Sb	LinStatic		20.284	-2.241	22.604
Base	87	55	Su1	LinStatic		12.118	-1.158	15.479
Base	87	55	Su2	LinStatic		13.663	-1.679	12.833
Base	87	55	Wx	LinStatic		-20.558	-7.99	-63.975
Base	87	55	Wy	LinStatic		-15.594	-13.054	-90.992
Base	87	55	Wxx	LinStatic		-9.433	-6.584	-33.368
Base	87	55	Wyy	LinStatic		-12.218	6.264	25.909
Base	87	55	Exx-O	LinStatic	Max	-19.826	-2.046	-56.378
Base	87	55	Exx-O	LinStatic	Min	-19.826	-2.046	-56.378
Base	87	55	Eyy-O	LinStatic	Max	-2.763	-10.143	-73.467
Base	87	55	Eyy-O	LinStatic	Min	-2.763	-10.143	-73.467
Base	88	58	Dead	LinStatic		-47.825	-5.888	48.352
Base	88	58	Live	LinStatic		-18.925	-3.039	17.278
Base	88	58	Exx	LinStatic	Max	-9.917	0.906	27.704
Base	88	58	Exx	LinStatic	Min	-9.917	0.906	27.704
Base	88	58	Eyy	LinStatic	Max	1.144	-5.632	-38.344
Base	88	58	Eyy	LinStatic	Min	1.144	-5.632	-38.344

Story	Label	Unique Name	Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip
Base	88	58	Mass	LinStatic		-41.376	-6.611	37.787
Base	88	58	Sb	LinStatic		-19.953	-3.197	18.22
Base	88	58	Su1	LinStatic		-13.452	-2.325	9.859
Base	88	58	Su2	LinStatic		-11.908	-1.726	12.882
Base	88	58	Wx	LinStatic		9.721	-7.119	-36.369
Base	88	58	Wy	LinStatic		15.083	-13.877	-92.461
Base	88	58	Wxx	LinStatic		20.857	-8.533	-67.103
Base	88	58	Wyy	LinStatic		12.523	7.413	29.412
Base	88	58	Exx-O	LinStatic	Max	-19.835	1.812	55.407
Base	88	58	Exx-O	LinStatic	Min	-19.835	1.812	55.407
Base	88	58	Eyy-O	LinStatic	Max	2.288	-11.264	-76.689
Base	88	58	Eyy-O	LinStatic	Min	2.288	-11.264	-76.689
Base	89	65	Dead	LinStatic		30.454	0.102	-7.318
Base	89	65	Live	LinStatic		14.176	0.051	-12.498
Base	89	65	Exx	LinStatic	Max	-9.401	-0.003	27.745
Base	89	65	Exx	LinStatic	Min	-9.401	-0.003	27.745
Base	89	65	Eyy	LinStatic	Max	7.379	-0.012	-21.933
Base	89	65	Eyy	LinStatic	Min	7.379	-0.012	-21.933
Base	89	65	Mass	LinStatic		30.917	0.111	-26.89
Base	89	65	Sb	LinStatic		14.918	0.054	-13.024
Base	89	65	Su1	LinStatic		10.181	0.031	-15.322
Base	89	65	Su2	LinStatic		9.148	0.037	-2.623
Base	89	65	Wx	LinStatic		-7.512	0.001	-7.55
Base	89	65	Wy	LinStatic		6.744	-0.029	-44.979
Base	89	65	Wxx	LinStatic		3.024	-0.0003863	-37.192
Base	89	65	Wyy	LinStatic		-9.891	0.032	-7.363
Base	89	65	Exx-O	LinStatic	Max	-18.802	-0.005	55.489
Base	89	65	Exx-O	LinStatic	Min	-18.802	-0.005	55.489
Base	89	65	Eyy-O	LinStatic	Max	14.759	-0.023	-43.867
Base	89	65	Eyy-O	LinStatic	Min	14.759	-0.023	-43.867
Base	90	66	Dead	LinStatic		-29.901	0.092	-3.888
Base	90	66	Live	LinStatic		-13.897	0.046	-10.778
Base	90	66	Exx	LinStatic	Max	-9.435	0.002	-27.738
Base	90	66	Exx	LinStatic	Min	-9.435	0.002	-27.738
Base	90	66	Eyy	LinStatic	Max	-7.586	-0.012	-21.694
Base	90	66	Eyy	LinStatic	Min	-7.586	-0.012	-21.694
Base	90	66	Mass	LinStatic		-30.308	0.1	-23.15
Base	90	66	Sb	LinStatic		-14.624	0.048	-11.217
Base	90	66	Su1	LinStatic		-8.98	0.034	-1.493
Base	90	66	Su2	LinStatic		-9.98	0.028	-14.179
Base	90	66	Wx	LinStatic		-2.91	-0.002	-36.367
Base	90	66	Wy	LinStatic		-7.012	-0.03	-44.586
Base	90	66	Wxx	LinStatic		7.599	-0.001	-6.804
Base	90	66	Wyy	LinStatic		10.13	0.035	-8.102
Base	90	66	Exx-O	LinStatic	Max	-18.87	0.004	-55.476
Base	90	66	Exx-O	LinStatic	Min	-18.87	0.004	-55.476
Base	90	66	Eyy-O	LinStatic	Max	-15.172	-0.025	-43.388
Base	90	66	Eyy-O	LinStatic	Min	-15.172	-0.025	-43.388
Base	173	105	Dead	LinStatic		49.54	-6.931	93.087
Base	173	105	Live	LinStatic		19.776	-3.361	39.491
Base	173	105	Exx	LinStatic	Max	-9.673	1.663	-29.721
Base	173	105	Exx	LinStatic	Min	-9.673	1.663	-29.721
Base	173	105	Eyy	LinStatic	Max	5.802	-5.627	49.287
Base	173	105	Eyy	LinStatic	Min	5.802	-5.627	49.287
Base	173	105	Mass	LinStatic		43.242	-7.306	86.127
Base	173	105	Sb	LinStatic		20.823	-3.533	41.539
Base	173	105	Su1	LinStatic		12.826	-2.228	27.235

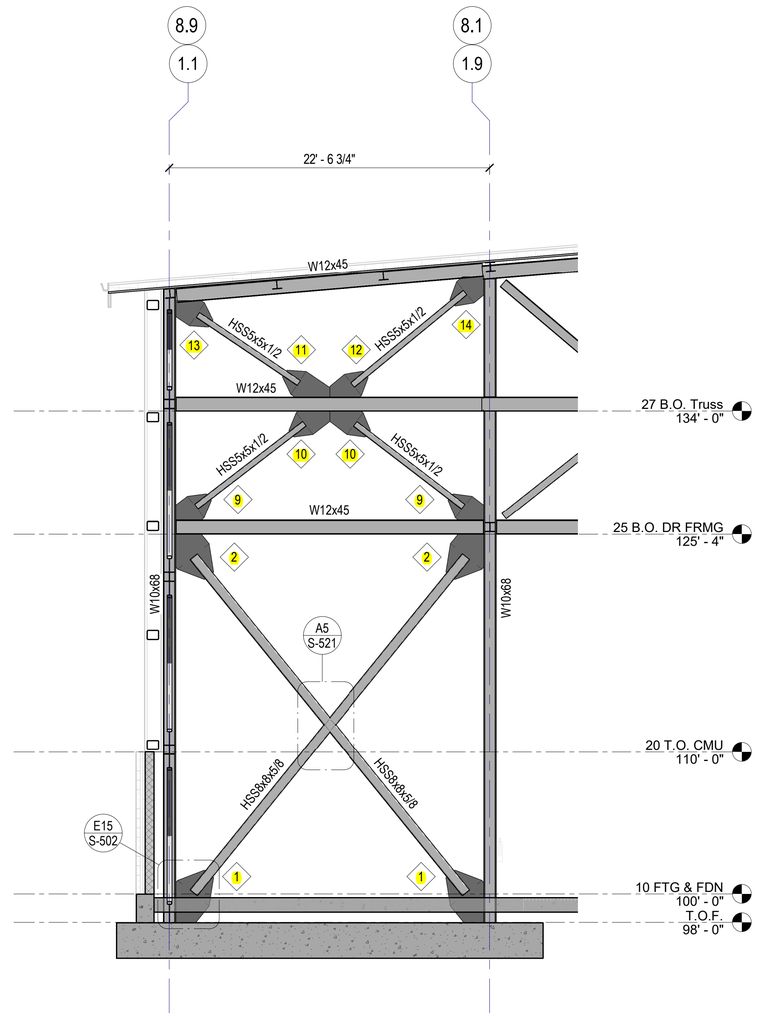
Story	Label	Unique Name	Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip
Base	173	105	Su2	LinStatic		13.858	-2.193	25.453
Base	173	105	Wx	LinStatic		-36.78	-35.208	-52.101
Base	173	105	Wy	LinStatic		7.216	-33.001	21.512
Base	173	105	Wxx	LinStatic		3.226	-36.443	-20.647
Base	173	105	Wyy	LinStatic		1.563	42.185	-98.904
Base	173	105	Exx-O	LinStatic	Max	-19.346	3.327	-59.442
Base	173	105	Exx-O	LinStatic	Min	-19.346	3.327	-59.442
Base	173	105	Eyy-O	LinStatic	Max	11.605	-11.254	98.574
Base	173	105	Eyy-O	LinStatic	Min	11.605	-11.254	98.574
Base	174	108	Dead	LinStatic		-48.021	-8.509	95.945
Base	174	108	Live	LinStatic		-19.022	-4.15	40.911
Base	174	108	Exx	LinStatic	Max	-9.65	-1.779	30.178
Base	174	108	Exx	LinStatic	Min	-9.65	-1.779	30.178
Base	174	108	Eyy	LinStatic	Max	-5.551	-6.177	50.792
Base	174	108	Eyy	LinStatic	Min	-5.551	-6.177	50.792
Base	174	108	Mass	LinStatic		-41.601	-9.023	89.217
Base	174	108	Sb	LinStatic		-20.03	-4.363	43.032
Base	174	108	Su1	LinStatic		-13.353	-2.76	26.605
Base	174	108	Su2	LinStatic		-12.328	-2.715	27.99
Base	174	108	Wx	LinStatic		-2.755	-36.882	-19.744
Base	174	108	Wy	LinStatic		-6.933	-33.854	23.749
Base	174	108	Wxx	LinStatic		37.235	-35.655	-51.071
Base	174	108	Wyy	LinStatic		-2.118	43.304	-101.816
Base	174	108	Exx-O	LinStatic	Max	-19.301	-3.559	60.355
Base	174	108	Exx-O	LinStatic	Min	-19.301	-3.559	60.355
Base	174	108	Eyy-O	LinStatic	Max	-11.102	-12.354	101.585
Base	174	108	Eyy-O	LinStatic	Min	-11.102	-12.354	101.585
Base	175	115	Dead	LinStatic		19.885	0.032	38.373
Base	175	115	Live	LinStatic		9.038	0.017	10.139
Base	175	115	Exx	LinStatic	Max	-8.719	-0.001	25.651
Base	175	115	Exx	LinStatic	Min	-8.719	-0.001	25.651
Base	175	115	Eyy	LinStatic	Max	-3.721	0.011	12.568
Base	175	115	Eyy	LinStatic	Min	-3.721	0.011	12.568
Base	175	115	Mass	LinStatic		19.732	0.037	22.404
Base	175	115	Sb	LinStatic		9.515	0.018	10.705
Base	175	115	Su1	LinStatic		7.053	0.011	-0.563
Base	175	115	Su2	LinStatic		5.38	0.012	13.33
Base	175	115	Wx	LinStatic		-3.732	0.003	-13.237
Base	175	115	Wy	LinStatic		-0.786	0.006	-21.158
Base	175	115	Wxx	LinStatic		9.188	0.002	-47.888
Base	175	115	Wyy	LinStatic		14.042	-0.002	-56.39
Base	175	115	Exx-O	LinStatic	Max	-17.438	-0.003	51.301
Base	175	115	Exx-O	LinStatic	Min	-17.438	-0.003	51.301
Base	175	115	Eyy-O	LinStatic	Max	-7.442	0.021	25.137
Base	175	115	Eyy-O	LinStatic	Min	-7.442	0.021	25.137
Base	176	116	Dead	LinStatic		-18.339	0.029	40.669
Base	176	116	Live	LinStatic		-8.28	0.016	11.244
Base	176	116	Exx	LinStatic	Max	-8.665	0.001	-25.621
Base	176	116	Exx	LinStatic	Min	-8.665	0.001	-25.621
Base	176	116	Eyy	LinStatic	Max	3.919	0.011	12.36
Base	176	116	Eyy	LinStatic	Min	3.919	0.011	12.36
Base	176	116	Mass	LinStatic		-18.082	0.034	24.811
Base	176	116	Sb	LinStatic		-8.718	0.016	11.868
Base	176	116	Su1	LinStatic		-4.852	0.011	14.079
Base	176	116	Su2	LinStatic		-6.568	0.01	0.173
Base	176	116	Wx	LinStatic		-8.594	0.002	-46.719
Base	176	116	Wy	LinStatic		0.831	0.006	-22.256

Story	Label	Unique Name	Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip
Base	176	116	Wxx	LinStatic		4.347	0.003	-11.991
Base	176	116	Wyy	LinStatic		-14.5	-0.002	-56.211
Base	176	116	Exx-O	LinStatic	Max	-17.33	0.002	-51.243
Base	176	116	Exx-O	LinStatic	Min	-17.33	0.002	-51.243
Base	176	116	Eyy-O	LinStatic	Max	7.837	0.022	24.719
Base	176	116	Eyy-O	LinStatic	Min	7.837	0.022	24.719
Base	216	155	Dead	LinStatic		1.458	0.004	123.557
Base	216	155	Live	LinStatic		0.722	0.003	59.663
Base	216	155	Exx	LinStatic	Max	-0.004	2.774E-05	-0.781
Base	216	155	Exx	LinStatic	Min	-0.004	2.774E-05	-0.781
Base	216	155	Eyy	LinStatic	Max	-0.028	0.074	-5.621
Base	216	155	Eyy	LinStatic	Min	-0.028	0.074	-5.621
Base	216	155	Mass	LinStatic		1.57	0.006	129.759
Base	216	155	Sb	LinStatic		0.759	0.003	62.696
Base	216	155	Su1	LinStatic		0.43	0.002	28.293
Base	216	155	Su2	LinStatic		0.53	0.002	47.719
Base	216	155	Wx	LinStatic		-0.074	-9.22E-05	-2.151
Base	216	155	Wy	LinStatic		-0.105	0.091	-5.08
Base	216	155	Wxx	LinStatic		-0.069	0.0004678	-1.836
Base	216	155	Wyy	LinStatic		-0.067	-0.086	2.618
Base	216	155	Exx-O	LinStatic	Max	-0.008	5.549E-05	-1.561
Base	216	155	Exx-O	LinStatic	Min	-0.008	5.549E-05	-1.561
Base	216	155	Eyy-O	LinStatic	Max	-0.055	0.147	-11.241
Base	216	155	Eyy-O	LinStatic	Min	-0.055	0.147	-11.241
Base	217	156	Dead	LinStatic		-1.452	15.588	131.865
Base	217	156	Live	LinStatic		-0.719	7.134	62.301
Base	217	156	Exx	LinStatic	Max	-0.001	-0.117	0.973
Base	217	156	Exx	LinStatic	Min	-0.001	-0.117	0.973
Base	217	156	Eyy	LinStatic	Max	0.017	-26.857	-84.46
Base	217	156	Eyy	LinStatic	Min	0.017	-26.857	-84.46
Base	217	156	Mass	LinStatic		-1.564	15.516	135.5
Base	217	156	Sb	LinStatic		-0.756	7.497	65.47
Base	217	156	Su1	LinStatic		-0.529	5.895	49.573
Base	217	156	Su2	LinStatic		-0.427	3.068	29.964
Base	217	156	Wx	LinStatic		0.074	0.768	2.562
Base	217	156	Wy	LinStatic		0.089	-35.756	-109.647
Base	217	156	Wxx	LinStatic		0.078	0.756	2.241
Base	217	156	Wyy	LinStatic		0.087	38.026	110.467
Base	217	156	Exx-O	LinStatic	Max	-0.002	-0.233	1.946
Base	217	156	Exx-O	LinStatic	Min	-0.002	-0.233	1.946
Base	217	156	Eyy-O	LinStatic	Max	0.034	-53.715	-168.919
Base	217	156	Eyy-O	LinStatic	Min	0.034	-53.715	-168.919
Base	310	157	Dead	LinStatic		1.053	10.205	93.528
Base	310	157	Live	LinStatic		0.522	4.455	43.123
Base	310	157	Exx	LinStatic	Max	-0.004	0.35	0.261
Base	310	157	Exx	LinStatic	Min	-0.004	0.35	0.261
Base	310	157	Eyy	LinStatic	Max	0.006	-26.09	-78.919
Base	310	157	Eyy	LinStatic	Min	0.006	-26.09	-78.919
Base	310	157	Mass	LinStatic		1.136	9.69	93.792
Base	310	157	Sb	LinStatic		0.549	4.681	45.318
Base	310	157	Su1	LinStatic		0.312	1.363	18.501
Base	310	157	Su2	LinStatic		0.383	4.031	35.694
Base	310	157	Wx	LinStatic		-0.042	-1.416	-0.021
Base	310	157	Wy	LinStatic		-0.033	-35.614	-104.81
Base	310	157	Wxx	LinStatic		-0.038	-1.39	-0.032
Base	310	157	Wyy	LinStatic		-0.049	37.891	107.548
Base	310	157	Exx-O	LinStatic	Max	-0.009	0.701	0.523

Story	Label	Unique Name	Output Case	Case Type	Step Type	FX kip	FY kip	FZ kip
Base	310	157	Exx-O	LinStatic	Min	-0.009	0.701	0.523
Base	310	157	Eyy-O	LinStatic	Max	0.011	-52.179	-157.839
Base	310	157	Eyy-O	LinStatic	Min	0.011	-52.179	-157.839
Base	311	160	Dead	LinStatic		-1.053	-10.205	93.528
Base	311	160	Live	LinStatic		-0.522	-4.455	43.122
Base	311	160	Exx	LinStatic	Max	-0.004	0.35	-0.261
Base	311	160	Exx	LinStatic	Min	-0.004	0.35	-0.261
Base	311	160	Eyy	LinStatic	Max	0.006	-26.09	78.919
Base	311	160	Eyy	LinStatic	Min	0.006	-26.09	78.919
Base	311	160	Mass	LinStatic		-1.136	-9.69	93.792
Base	311	160	Sb	LinStatic		-0.549	-4.681	45.318
Base	311	160	Su1	LinStatic		-0.383	-4.03	35.694
Base	311	160	Su2	LinStatic		-0.312	-1.363	18.501
Base	311	160	Wx	LinStatic		0.034	3.222	-4.904
Base	311	160	Wy	LinStatic		0.045	-36.123	102.92
Base	311	160	Wxx	LinStatic		0.038	3.214	-4.739
Base	311	160	Wyy	LinStatic		0.029	37.382	-109.438
Base	311	160	Exx-O	LinStatic	Max	-0.009	0.701	-0.523
Base	311	160	Exx-O	LinStatic	Min	-0.009	0.701	-0.523
Base	311	160	Eyy-O	LinStatic	Max	0.011	-52.179	157.839
Base	311	160	Eyy-O	LinStatic	Min	0.011	-52.179	157.839
Base	353	196	Dead	LinStatic		1.453	-15.587	131.865
Base	353	196	Live	LinStatic		0.719	-7.134	62.301
Base	353	196	Exx	LinStatic	Max	-0.001	-0.117	-0.973
Base	353	196	Exx	LinStatic	Min	-0.001	-0.117	-0.973
Base	353	196	Eyy	LinStatic	Max	0.017	-26.857	84.46
Base	353	196	Eyy	LinStatic	Min	0.017	-26.857	84.46
Base	353	196	Mass	LinStatic		1.564	-15.516	135.499
Base	353	196	Sb	LinStatic		0.756	-7.497	65.469
Base	353	196	Su1	LinStatic		0.427	-3.068	29.964
Base	353	196	Su2	LinStatic		0.529	-5.895	49.573
Base	353	196	Wx	LinStatic		-0.025	1.91	-2.194
Base	353	196	Wy	LinStatic		-0.034	-35.39	106.129
Base	353	196	Wxx	LinStatic		-0.021	1.939	-2.059
Base	353	196	Wyy	LinStatic		-0.036	38.391	-113.985
Base	353	196	Exx-O	LinStatic	Max	-0.002	-0.233	-1.945
Base	353	196	Exx-O	LinStatic	Min	-0.002	-0.233	-1.945
Base	353	196	Eyy-O	LinStatic	Max	0.034	-53.715	168.919
Base	353	196	Eyy-O	LinStatic	Min	0.034	-53.715	168.919
Base	354	199	Dead	LinStatic		-1.458	-0.004	123.557
Base	354	199	Live	LinStatic		-0.722	-0.003	59.663
Base	354	199	Exx	LinStatic	Max	-0.004	2.763E-05	0.781
Base	354	199	Exx	LinStatic	Min	-0.004	2.763E-05	0.781
Base	354	199	Eyy	LinStatic	Max	-0.028	0.074	5.621
Base	354	199	Eyy	LinStatic	Min	-0.028	0.074	5.621
Base	354	199	Mass	LinStatic		-1.57	-0.006	129.759
Base	354	199	Sb	LinStatic		-0.759	-0.003	62.695
Base	354	199	Su1	LinStatic		-0.53	-0.002	47.719
Base	354	199	Su2	LinStatic		-0.43	-0.002	28.292
Base	354	199	Wx	LinStatic		0.021	-0.001	-1.546
Base	354	199	Wy	LinStatic		0.019	0.086	2.949
Base	354	199	Wxx	LinStatic		0.026	0.000183	-1.827
Base	354	199	Wyy	LinStatic		0.057	-0.09	-4.749
Base	354	199	Exx-O	LinStatic	Max	-0.008	5.527E-05	1.561
Base	354	199	Exx-O	LinStatic	Min	-0.008	5.527E-05	1.561
Base	354	199	Eyy-O	LinStatic	Max	-0.055	0.147	11.241
Base	354	199	Eyy-O	LinStatic	Min	-0.055	0.147	11.241



D6 BRACED FRAME - GRIDS A.1, D.9
3/16" = 1'-0" REF: SB101



D14 BRACED FRAME - GRIDS A.3, D.7
3/16" = 1'-0" REF: SB101

3/16" = 1' - 0" 0 5' 10' 20'

PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

US Army Corps of Engineers ®

MARK	DESCRIPTION	DATE

ISSUE DATE: NOVEMBER 5, 2025
SOLICITATION NO.: W912PL26RA0012
CONTRACT NO.: W912PL25C0037

DESIGNED BY: A. VALENCIA
DRAWN BY: R. CARLSON
CHECKED BY: D. CAMPION
SUBMITTED BY: P. PASZCZUK
SIZE: ANSI D

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

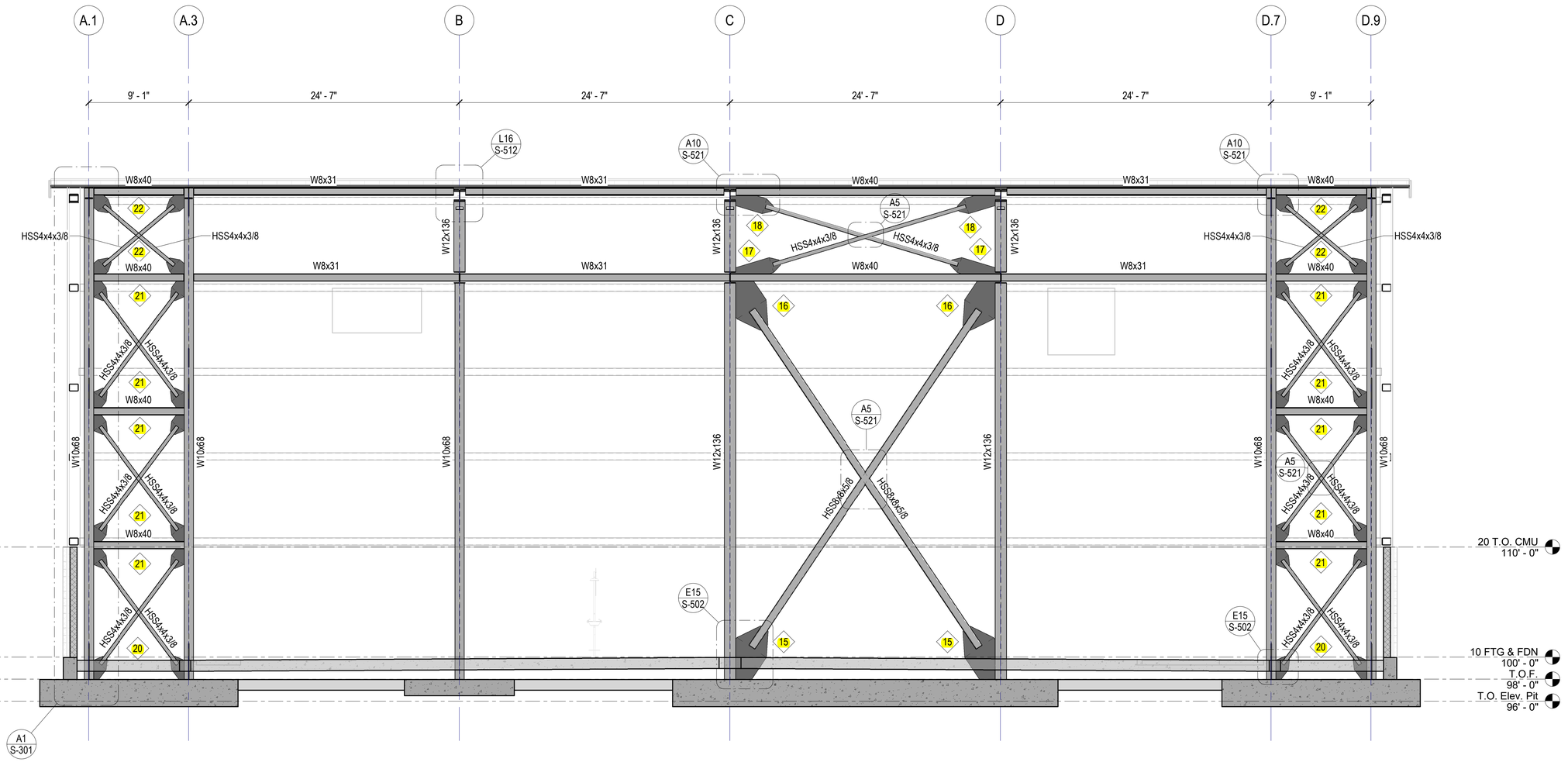
KORTE CONSTRUCTION
5700 OAKLAND AVE, SUITE 275
ST. LOUIS, MO 63110

CREECH AIR FORCE BASE, CLARK COUNTY, NV
DISASTER RESILIENCY PROGRAM (DRP) - PHASE 2
494137

BRACED FRAME ELEVATIONS

SHEET ID
S-201

DP-1 95% SUBMISSION



B2 BRACED FRAME ELEVATION - GRIDS 1.1, 8.9

3/16" = 1'-0" REF: SB101

3/16" = 1' - 0" 0 5' 10' 20'

PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

US Army Corps of Engineers ®

MARK	DESCRIPTION	DATE

ISSUE DATE: NOVEMBER 5, 2025
SOLICITATION NO.: W912PL26RA0012
CONTRACT NO.: W912PL25C0007

DESIGNED BY: A. VALENCIA
DRAWN BY: R. CARLSON
CHECKED BY: D. CARLSON
SUBMITTED BY: P. PASZCZUK
SIZE: ANSI D

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

KORTE CONSTRUCTION
5700 OAKLAND AVE. SUITE 275
ST. LOUIS, MO 63110

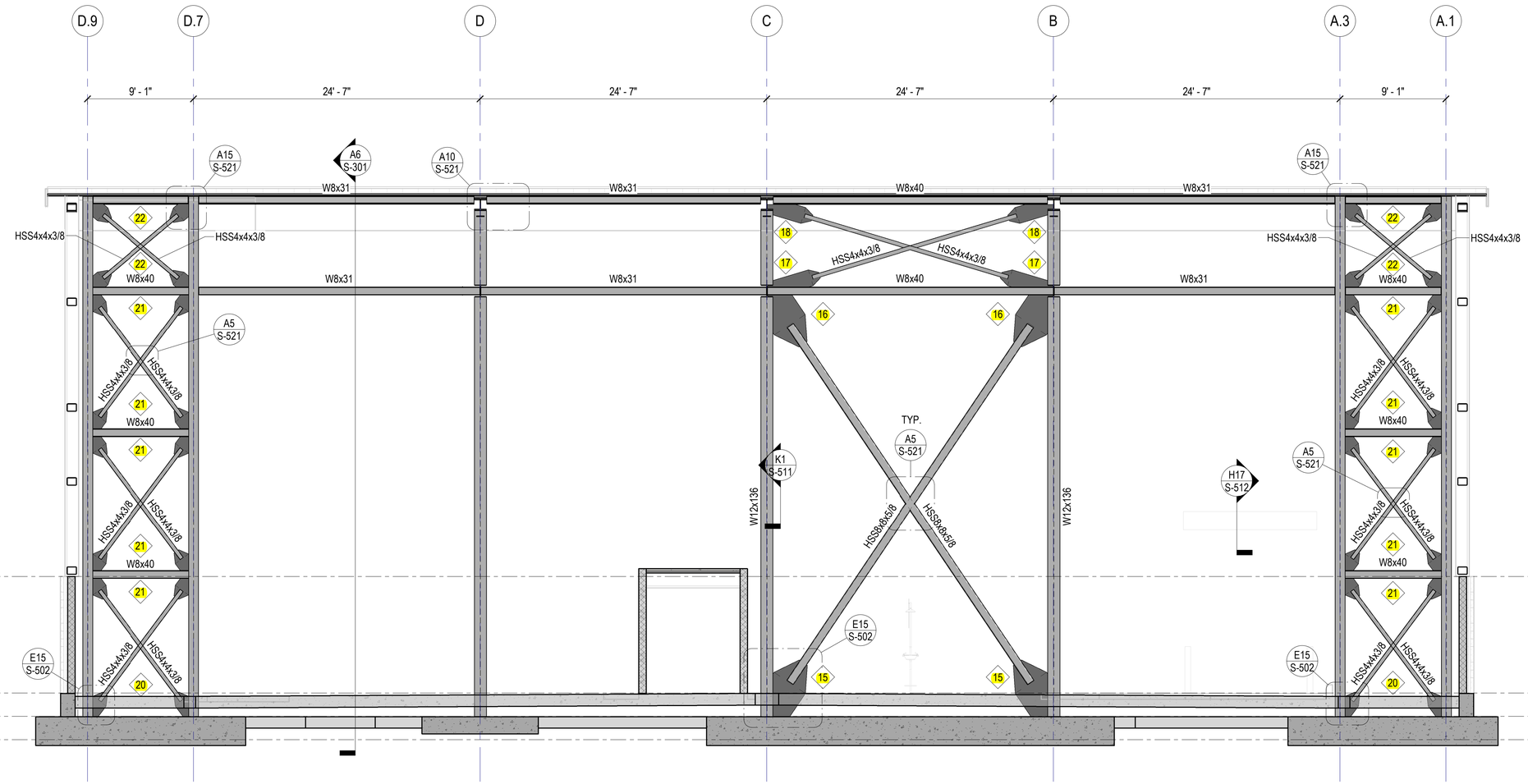
CREECH AIR FORCE BASE, CLARK COUNTY, NV
DISASTER RESILIENCY PROGRAM (DRP) - PHASE 2
494137

BRACED FRAME ELEVATIONS

SHEET ID

S-202

DP-1 95% SUBMISSION



B3 BRACED FRAME ELEVATION - GRIDS 3.9, 6.1

3/16" = 1'-0" REF: SB103

3/16" = 1' - 0" 0 5' 10' 20'

PRELIMINARY DESIGN NOT FOR CONSTRUCTION

US Army Corps of Engineers ®

MARK	DESCRIPTION	DATE

ISSUE DATE: NOVEMBER 5, 2025
SOLICITATION NO.: W912PL26RA0012
CONTRACT NO.: W912PL25C0007

DESIGNED BY: A. VALENCIA
DRAWN BY: R. CARLSON
CHECKED BY: D. CARLSON
SUBMITTED BY: P. PASZCZUK
SIZE: ANSI D

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

KORTE CONSTRUCTION
5700 OAKLAND AVE, SUITE 275
ST. LOUIS, MO 63110

CREECH AIR FORCE BASE, CLARK COUNTY, NV
DISASTER RESILIENCY PROGRAM (DRP) - PHASE 2
494137

BRACED FRAME ELEVATIONS

SHEET ID

S-203

DP-1 95% SUBMISSION

DESIGN MODULE - SCBF Gusset Design

Michael Baker INTERNATIONAL

Creech DRP Phase 2

TYPICAL BRACED FRAME DESIGN INFORMATION

Frame Height:	20.00 ft	Gusset Plate Fy:	50 ksi
Floor Thickness:	0.00 in	Weld Size Brace to Gusset:	5 /16
Gusset Thickness:	0.75 in	Corner Snip:	1.00 in
Measured from Top of Slab:	No		
Beam Size Above:	W12X45		
Beam Size Below:	BasePlate		
Column Size:	W10X68		
Brace Size:	HSS8X8X5/8		
Brace Grade:	A500 Gr C		

BRACED FRAME GUSSET SUMMARY SCHEDULE

FRAME ID	GUSSET MARK	LENGTH	HEIGHT	BEAM		COLUMN	BRACE	STATUS
BF1	1	22.56 ft	27.84 ft	TOP	W12X45	W10X68	HSS8X8X5/8	PASS
	2			BTM	BasePlate			PASS
BF2	3	11.28 ft	8.66 ft	TOP	W12X45	W10X68	HSS5X5X1/2	PASS
	4			BTM	W12X45			PASS
BF3	5	11.28 ft	7.59 ft	TOP	W12X45	W10X68	HSS5X5X1/2	PASS
	6			BTM	W12X45			PASS
BF4	7	11.28 ft	9.47 ft	TOP	W12X45	W10X68	HSS5X5X1/2	PASS
	8			BTM	W12X45			PASS
BF5	9	24.58 ft	36.51 ft	TOP	W8X40	BasePlate	HSS8X8X5/8	PASS
	10			BTM	BasePlate			PASS
BF6	11	24.58 ft	7.79 ft	TOP	W8X40	BasePlate	HSS4X4X3/8	PASS
	12			BTM	W8X40			PASS
BF7	13	9.08 ft	12.17 ft	TOP	W8X40	BasePlate	HSS4X4X3/8	PASS
	14			BTM	BasePlate			PASS
BF8	15	9.08 ft	7.79 ft	TOP	W8X40	BasePlate	HSS4X4X3/8	PASS
	16			BTM	W8X40			PASS

DESIGN MODULE - SCBF Gusset Plate

Michael Baker INTERNATIONAL

Creech DRP Phase 2

Steel Gusset Plate Geometry and Modeling Information

Mark	Frame	Gusset Thickness	Angle from Horizontal	Brace Information		Length Along Beam	Length Along Column	Comment
				Brace Weld Length	Brace Width			
1	BF1	1.00 in	50.98 deg	39 in	8.00 in	37 in	46 in	
2		1.00 in	50.98 deg	39 in	8.00 in	32 in	52 in	
3	BF2	0.88 in	37.52 deg	19 in	5.00 in	29 in	18 in	
4		0.88 in	37.52 deg	19 in	5.00 in	29 in	18 in	
5	BF3	0.88 in	33.92 deg	19 in	5.00 in	32 in	17 in	
6		0.88 in	33.92 deg	19 in	5.00 in	32 in	17 in	
7	BF4	0.88 in	40.00 deg	19 in	5.00 in	27 in	19 in	
8		0.88 in	40.00 deg	19 in	5.00 in	27 in	19 in	
9	BF5	0.88 in	56.05 deg	39 in	8.00 in	38 in	47 in	
10		0.88 in	56.05 deg	39 in	8.00 in	35 in	52 in	
11	BF6	0.75 in	17.59 deg	12 in	4.00 in	49 in	8 in	
12		0.75 in	17.59 deg	12 in	4.00 in	49 in	8 in	
13	BF7	0.75 in	53.27 deg	12 in	4.00 in	17 in	14 in	
14		0.75 in	53.27 deg	12 in	4.00 in	14 in	19 in	
15	BF8	0.75 in	40.63 deg	12 in	4.00 in	22 in	11 in	
16		0.75 in	40.63 deg	12 in	4.00 in	22 in	11 in	

Steel Gusset Plate Connection Schedule

Mark	Gusset Thickness	Geometry + Welding Information					Cover Plate Info	
		W1 Size	L1 Length	W2 Size	L2 Length	W3 Size	Plate Size	Weld
1	1.00 in	5 /16	39 in	7 /16	37 in	6 /16		
2	1.00 in	5 /16	39 in	6 /16	32 in	7 /16		
3	0.88 in	5 /16	19 in	5 /16	29 in	6 /16		
4	0.88 in	5 /16	19 in	5 /16	29 in	6 /16		
5	0.88 in	5 /16	19 in	5 /16	32 in	6 /16		
6	0.88 in	5 /16	19 in	5 /16	32 in	6 /16		
7	0.88 in	5 /16	19 in	5 /16	27 in	6 /16		
8	0.88 in	5 /16	19 in	5 /16	27 in	6 /16		
9	0.88 in	5 /16	39 in	7 /16	38 in	6 /16		
10	0.88 in	5 /16	39 in	7 /16	35 in	7 /16		
11	0.75 in	5 /16	12 in	3 /16	49 in	3 /16		
12	0.75 in	5 /16	12 in	3 /16	49 in	3 /16		
13	0.75 in	5 /16	12 in	6 /16	17 in	4 /16		
14	0.75 in	5 /16	12 in	5 /16	14 in	5 /16		
15	0.75 in	5 /16	12 in	5 /16	22 in	4 /16		
16	0.75 in	5 /16	12 in	5 /16	22 in	4 /16		

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF1 Top, Gusset 1

GENERAL CRITERIA

Frame Height:	27.84 ft	Beam Size:	W12X45
Frame Length:	22.56 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS8X8X5/8
Gusset Thickness:	1.00 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

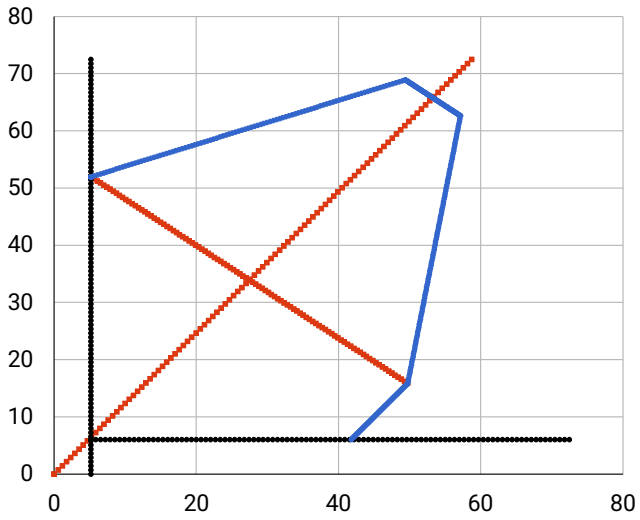
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	27.84 ft
Frame Length:	22.56 ft
Brace Length:	35.84 ft
Angle from Horz:	50.98 deg
Angle from Vert:	39.02 deg

Gusset Plate Geometry

Brace on Gusset:	39.00 in
Length along Beam:	36.61 in
Length along Column:	45.91 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	6.05 in
a:	18.76 in
b:	23.51 in
r:	38.05

Max Brace Force, Pu **= 1066 kips**

Vub:	0.159 x Pu	**= 170 kips**	V= Vertical Force
Hub:	0.493 x Pu	**= 526 kips**	H= Horizontal Force
Vuc:	0.618 x Pu	**= 659 kips**	b= Beam Side
Huc:	0.137 x Pu	**= 146 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	7/16	Weld to Column Size:	6/16
Interaction for Vub:	0.155	Interaction for Vuc:	0.839
Interaction for Hub:	0.720	Interaction for Huc:	0.124
Total Interaction:	**0.875**	Total Interaction:	**0.963**

Gusset Design

Buckling Capacity:	238k OK
Yielding Capacity:	2476k OK
Block Shear Capacity:	2145k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF1 Bottom, Gusset 2

GENERAL CRITERIA

Frame Height:	27.84 ft	Beam Size:	BasePlate
Frame Length:	22.56 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS8X8X5/8
Gusset Thickness:	1.00 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

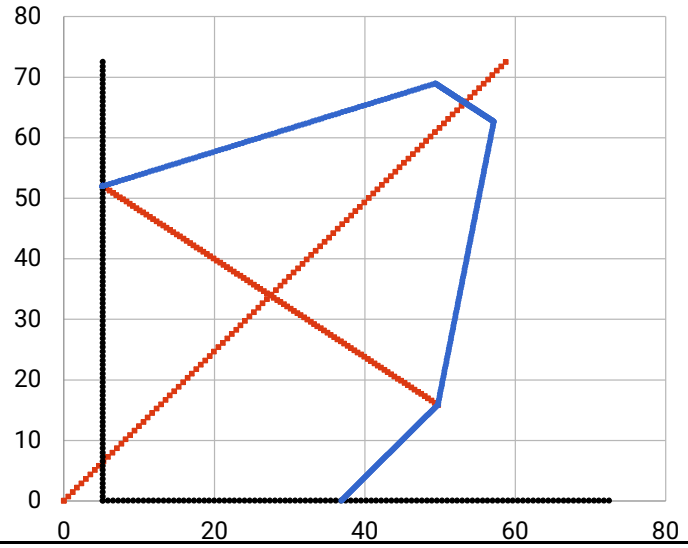
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	27.84 ft
Frame Length:	22.56 ft
Brace Length:	35.84 ft
Angle from Horz:	50.98 deg
Angle from Vert:	39.02 deg

Gusset Plate Geometry

Brace on Gusset:	39.00 in
Length along Beam:	31.70 in
Length along Column:	51.96 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	0.00 in
a:	16.32 in
b:	26.55 in
r:	34.18

Max Brace Force, Pu **= 1066 kips**

Vub:	0.000 x Pu	**= 0 kips**	V= Vertical Force
Hub:	0.477 x Pu	**= 509 kips**	H= Horizontal Force
Vuc:	0.777 x Pu	**= 829 kips**	b= Beam Side
Huc:	0.152 x Pu	**= 163 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	6/16	Weld to Column Size:	7/16
Interaction for Vub:	0.000	Interaction for Vuc:	0.801
Interaction for Hub:	0.934	Interaction for Huc:	0.105
Total Interaction:	**0.934**	Total Interaction:	**0.906**

Gusset Design

Buckling Capacity:	238k OK
Yielding Capacity:	2476k OK
Block Shear Capacity:	2145k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF2 Top, Gusset 3

GENERAL CRITERIA

Frame Height:	8.66 ft	Beam Size:	W12X45
Frame Length:	11.28 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS5X5X1/2
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

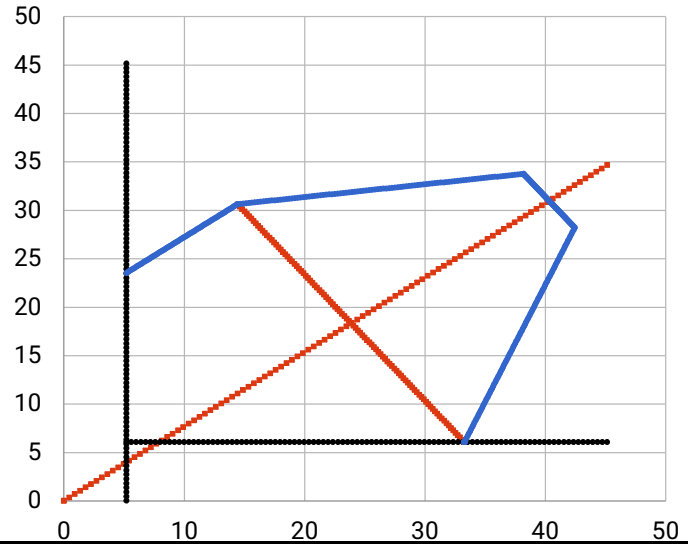
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	8.66 ft
Frame Length:	11.28 ft
Brace Length:	14.22 ft
Angle from Horz:	37.52 deg
Angle from Vert:	52.48 deg

Gusset Plate Geometry

Brace on Gusset:	19.00 in
Length along Beam:	28.10 in
Length along Column:	17.46 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	6.05 in
a:	14.64 in
b:	9.18 in
r:	25.01

Max Brace Force, Pu **= 512 kips**

Vub:	0.242 x Pu	**= 124 kips**	V= Vertical Force
Hub:	0.585 x Pu	**= 300 kips**	H= Horizontal Force
Vuc:	0.367 x Pu	**= 189 kips**	b= Beam Side
Huc:	0.208 x Pu	**= 107 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	6/16
Interaction for Vub:	0.203	Interaction for Vuc:	0.616
Interaction for Hub:	0.736	Interaction for Huc:	0.233
Total Interaction:	**0.939**	Total Interaction:	**0.849**

Gusset Design

Buckling Capacity:	303k OK
Yielding Capacity:	1139k OK
Block Shear Capacity:	961k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF2 Bottom, Gusset 4

GENERAL CRITERIA

Frame Height:	8.66 ft	Beam Size:	W12X45
Frame Length:	11.28 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS5X5X1/2
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

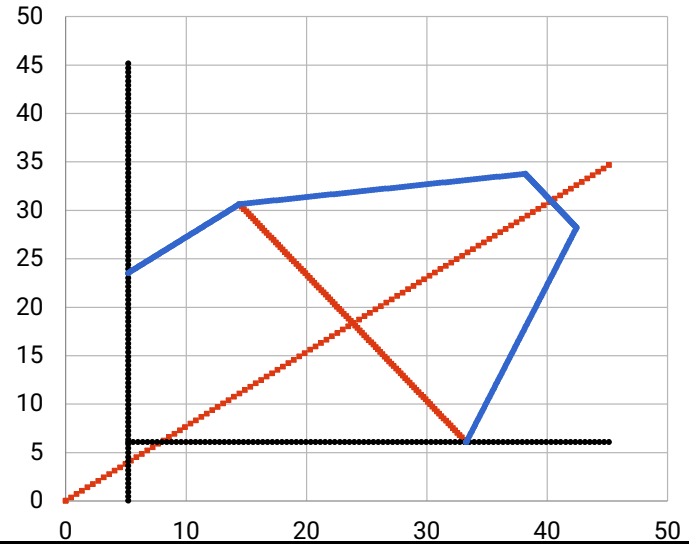
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	8.66 ft
Frame Length:	11.28 ft
Brace Length:	14.22 ft
Angle from Horz:	37.52 deg
Angle from Vert:	52.48 deg

Gusset Plate Geometry

Brace on Gusset:	19.00 in
Length along Beam:	28.10 in
Length along Column:	17.46 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	6.05 in
a:	14.64 in
b:	9.18 in
r:	25.01

Max Brace Force, Pu **= 512 kips**

Vub:	0.242 x Pu	**= 124 kips**	V= Vertical Force
Hub:	0.585 x Pu	**= 300 kips**	H= Horizontal Force
Vuc:	0.367 x Pu	**= 189 kips**	b= Beam Side
Huc:	0.208 x Pu	**= 107 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	6/16
Interaction for Vub:	0.203	Interaction for Vuc:	0.616
Interaction for Hub:	0.736	Interaction for Huc:	0.233
Total Interaction:	**0.939**	Total Interaction:	**0.849**

Gusset Design

Buckling Capacity:	303k OK
Yielding Capacity:	1139k OK
Block Shear Capacity:	961k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF3 Top, Gusset 5

GENERAL CRITERIA

Frame Height:	7.59 ft	Beam Size:	W12X45
Frame Length:	11.28 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS5X5X1/2
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

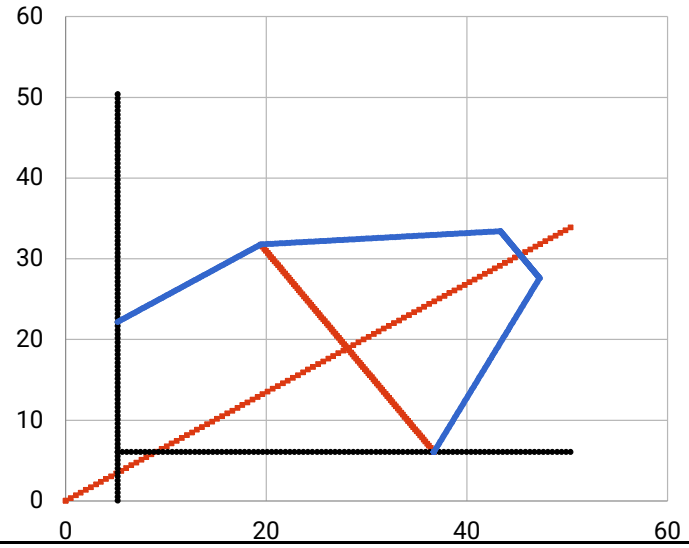
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	7.59 ft
Frame Length:	11.28 ft
Brace Length:	13.60 ft
Angle from Horz:	33.92 deg
Angle from Vert:	56.08 deg

Gusset Plate Geometry

Brace on Gusset:	19.00 in
Length along Beam:	31.54 in
Length along Column:	16.10 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	6.05 in
a:	16.42 in
b:	8.49 in
r:	26.05

Max Brace Force, Pu **= 512 kips**

Vub:	0.232 x Pu	**= 119 kips**	V= Vertical Force
Hub:	0.630 x Pu	**= 323 kips**	H= Horizontal Force
Vuc:	0.326 x Pu	**= 167 kips**	b= Beam Side
Huc:	0.200 x Pu	**= 103 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	6/16
Interaction for Vub:	0.174	Interaction for Vuc:	0.589
Interaction for Hub:	0.707	Interaction for Huc:	0.242
Total Interaction:	**0.880**	Total Interaction:	**0.831**

Gusset Design

Buckling Capacity:	303k OK
Yielding Capacity:	1139k OK
Block Shear Capacity:	961k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF3 Bottom, Gusset 6

GENERAL CRITERIA

Frame Height:	7.59 ft	Beam Size:	W12X45
Frame Length:	11.28 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS5X5X1/2
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

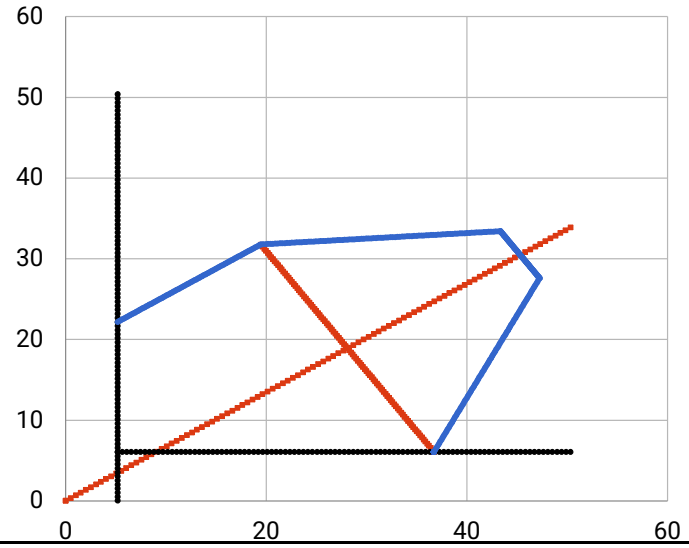
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	7.59 ft
Frame Length:	11.28 ft
Brace Length:	13.60 ft
Angle from Horz:	33.92 deg
Angle from Vert:	56.08 deg

Gusset Plate Geometry

Brace on Gusset:	19.00 in
Length along Beam:	31.54 in
Length along Column:	16.10 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	6.05 in
a:	16.42 in
b:	8.49 in
r:	26.05

Max Brace Force, Pu **= 512 kips**

Vub:	0.232 x Pu	**= 119 kips**	V= Vertical Force
Hub:	0.630 x Pu	**= 323 kips**	H= Horizontal Force
Vuc:	0.326 x Pu	**= 167 kips**	b= Beam Side
Huc:	0.200 x Pu	**= 103 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	6/16
Interaction for Vub:	0.174	Interaction for Vuc:	0.589
Interaction for Hub:	0.707	Interaction for Huc:	0.242
Total Interaction:	**0.880**	Total Interaction:	**0.831**

Gusset Design

Buckling Capacity:	303k OK
Yielding Capacity:	1139k OK
Block Shear Capacity:	961k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF4 Top, Gusset 7

GENERAL CRITERIA

Frame Height:	9.47 ft	Beam Size:	W12X45
Frame Length:	11.28 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS5X5X1/2
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

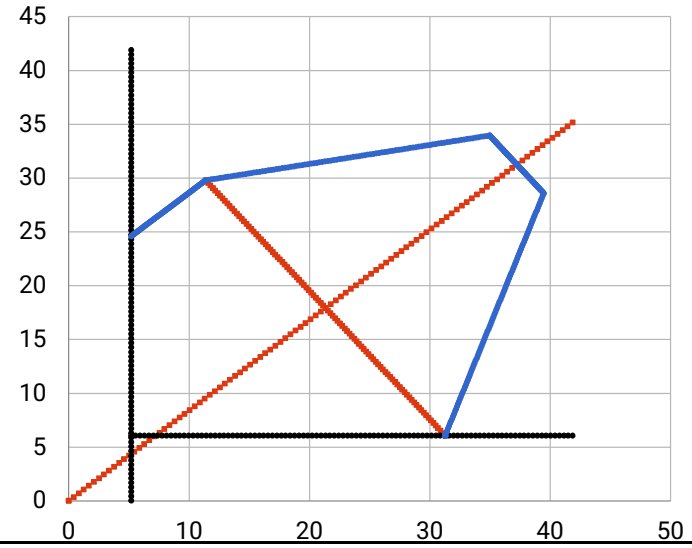
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	9.47 ft
Frame Length:	11.28 ft
Brace Length:	14.73 ft
Angle from Horz:	40.00 deg
Angle from Vert:	50.00 deg

Gusset Plate Geometry

Brace on Gusset:	19.00 in
Length along Beam:	26.09 in
Length along Column:	18.52 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	6.05 in
a:	13.60 in
b:	9.73 in
r:	24.54

Max Brace Force, Pu **= 512 kips**

Vub:	0.247 x Pu	**= 127 kips**	V= Vertical Force
Hub:	0.554 x Pu	**= 284 kips**	H= Horizontal Force
Vuc:	0.396 x Pu	**= 203 kips**	b= Beam Side
Huc:	0.212 x Pu	**= 109 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	6/16
Interaction for Vub:	0.224	Interaction for Vuc:	0.625
Interaction for Hub:	0.750	Interaction for Huc:	0.224
Total Interaction:	**0.974**	Total Interaction:	**0.848**

Gusset Design

Buckling Capacity:	303k OK
Yielding Capacity:	1139k OK
Block Shear Capacity:	961k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF4 Bottom, Gusset 8

GENERAL CRITERIA

Frame Height:	9.47 ft	Beam Size:	W12X45
Frame Length:	11.28 ft	Column Size:	W10X68
Floor Thickness:	0.00 in	Brace Size:	HSS5X5X1/2
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

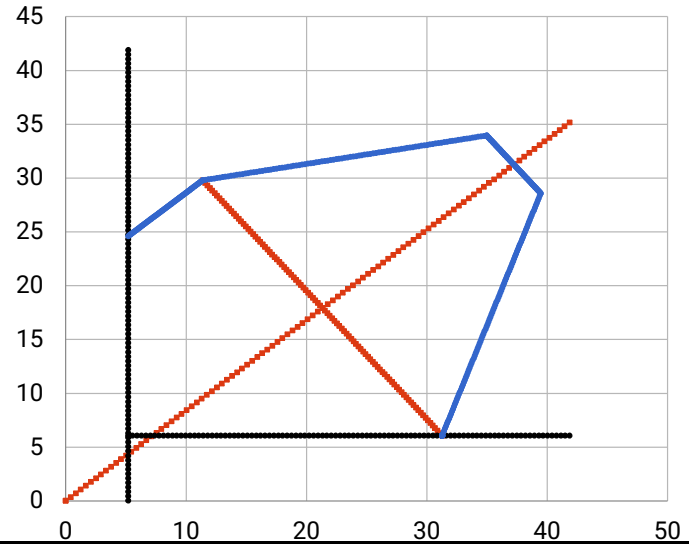
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	9.47 ft
Frame Length:	11.28 ft
Brace Length:	14.73 ft
Angle from Horz:	40.00 deg
Angle from Vert:	50.00 deg

Gusset Plate Geometry

Brace on Gusset:	19.00 in
Length along Beam:	26.09 in
Length along Column:	18.52 in



DESIGN OUTPUT

Uniform Force Method

ec:	5.20 in
eb:	6.05 in
a:	13.60 in
b:	9.73 in
r:	24.54

Max Brace Force, Pu **= 512 kips**

Vub:	0.247 x Pu	**= 127 kips**	V= Vertical Force
Hub:	0.554 x Pu	**= 284 kips**	H= Horizontal Force
Vuc:	0.396 x Pu	**= 203 kips**	b= Beam Side
Huc:	0.212 x Pu	**= 109 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	6/16
Interaction for Vub:	0.224	Interaction for Vuc:	0.625
Interaction for Hub:	0.750	Interaction for Huc:	0.224
Total Interaction:	**0.974**	Total Interaction:	**0.848**

Gusset Design

Buckling Capacity:	303k OK
Yielding Capacity:	1139k OK
Block Shear Capacity:	961k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF5 Top, Gusset 9

GENERAL CRITERIA

Frame Height:	36.51 ft	Beam Size:	W8X40
Frame Length:	24.58 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS8X8X5/8
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

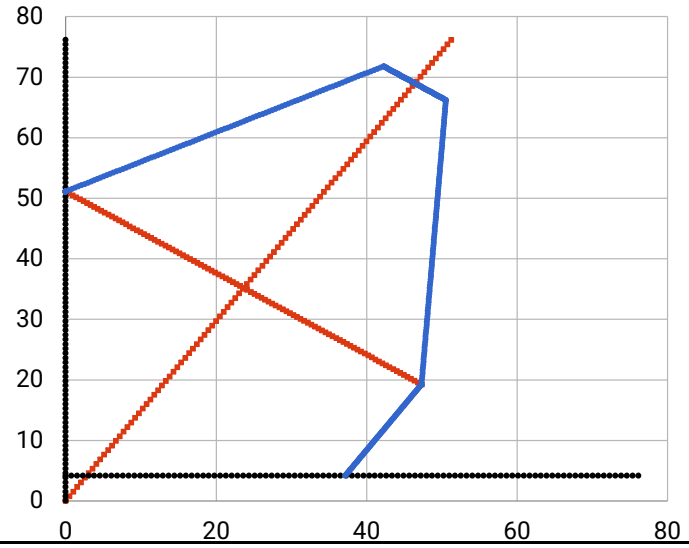
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	36.51 ft
Frame Length:	24.58 ft
Brace Length:	44.02 ft
Angle from Horz:	56.05 deg
Angle from Vert:	33.95 deg

Gusset Plate Geometry

Brace on Gusset:	39.00 in
Length along Beam:	37.17 in
Length along Column:	46.95 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	4.13 in
a:	18.99 in
b:	24.08 in
r:	34.00

Max Brace Force, Pu = **1066 kips**

Vub:	0.121 x Pu	**= 130 kips**	V= Vertical Force
Hub:	0.558 x Pu	**= 596 kips**	H= Horizontal Force
Vuc:	0.708 x Pu	**= 755 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	7/16	Weld to Column Size:	6/16
Interaction for Vub:	0.117	Interaction for Vuc:	0.939
Interaction for Hub:	0.805	Interaction for Huc:	0.000
Total Interaction:	**0.922**	Total Interaction:	**0.939**

Gusset Design

Buckling Capacity:	159k OK
Yielding Capacity:	2167k OK
Block Shear Capacity:	1877k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF5 Bottom, Gusset 10

GENERAL CRITERIA

Frame Height:	36.51 ft	Beam Size:	BasePlate
Frame Length:	24.58 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS8X8X5/8
Gusset Thickness:	0.88 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

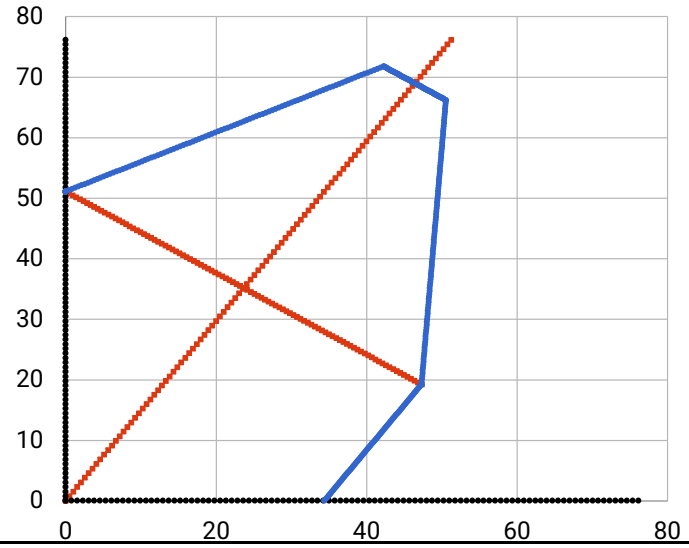
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	36.51 ft
Frame Length:	24.58 ft
Brace Length:	44.02 ft
Angle from Horz:	56.05 deg
Angle from Vert:	33.95 deg

Gusset Plate Geometry

Brace on Gusset:	39.00 in
Length along Beam:	34.39 in
Length along Column:	51.08 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	0.00 in
a:	17.61 in
b:	26.16 in
r:	31.54

Max Brace Force, Pu **= 1066 kips**

Vub:	0.000 x Pu	**= 0 kips**	V= Vertical Force
Hub:	0.558 x Pu	**= 596 kips**	H= Horizontal Force
Vuc:	0.830 x Pu	**= 885 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	7/16	Weld to Column Size:	7/16
Interaction for Vub:	0.000	Interaction for Vuc:	0.868
Interaction for Hub:	0.868	Interaction for Huc:	0.000
Total Interaction:	**0.868**	Total Interaction:	**0.868**

Gusset Design

Buckling Capacity:	159k OK
Yielding Capacity:	2167k OK
Block Shear Capacity:	1877k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF6 Top, Gusset 11

GENERAL CRITERIA

Frame Height:	7.79 ft	Beam Size:	W8X40
Frame Length:	24.58 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS4X4X3/8
Gusset Thickness:	0.75 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

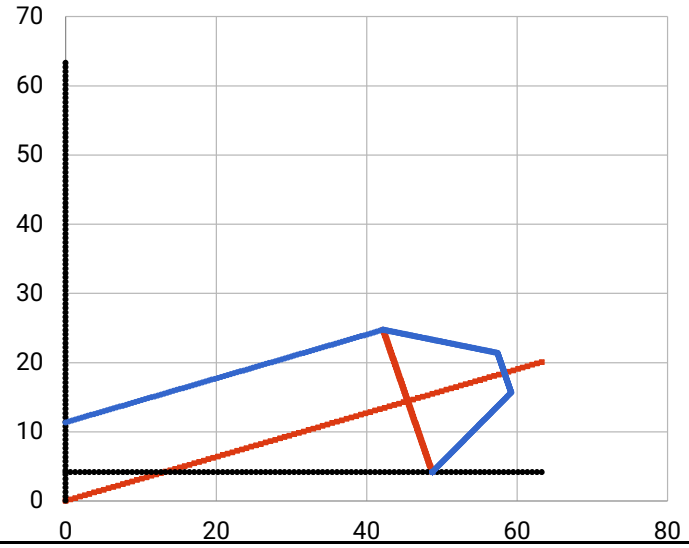
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	7.79 ft
Frame Length:	24.58 ft
Brace Length:	25.79 ft
Angle from Horz:	17.59 deg
Angle from Vert:	72.41 deg

Gusset Plate Geometry

Brace on Gusset:	12.00 in
Length along Beam:	48.74 in
Length along Column:	7.20 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	4.13 in
a:	25.72 in
b:	4.03 in
r:	26.98

Max Brace Force, Pu **= 312 kips**

Vub:	0.153 x Pu	**= 48 kips**	V= Vertical Force
Hub:	0.953 x Pu	**= 298 kips**	H= Horizontal Force
Vuc:	0.149 x Pu	**= 47 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	3/16	Weld to Column Size:	3/16
Interaction for Vub:	0.075	Interaction for Vuc:	0.699
Interaction for Hub:	0.694	Interaction for Huc:	0.000
Total Interaction:	**0.768**	Total Interaction:	**0.699**

Gusset Design

Buckling Capacity:	276k OK
Yielding Capacity:	670k OK
Block Shear Capacity:	551k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF6 Bottom, Gusset 12

GENERAL CRITERIA

Frame Height:	7.79 ft	Beam Size:	W8X40
Frame Length:	24.58 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS4X4X3/8
Gusset Thickness:	0.75 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

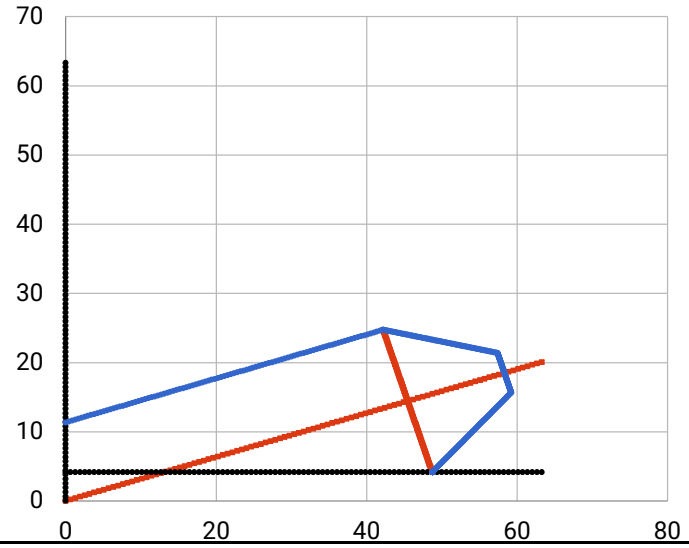
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	7.79 ft
Frame Length:	24.58 ft
Brace Length:	25.79 ft
Angle from Horz:	17.59 deg
Angle from Vert:	72.41 deg

Gusset Plate Geometry

Brace on Gusset:	12.00 in
Length along Beam:	48.74 in
Length along Column:	7.20 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	4.13 in
a:	25.72 in
b:	4.03 in
r:	26.98

Max Brace Force, Pu **= 312 kips**

Vub:	0.153 x Pu	**= 48 kips**	V= Vertical Force
Hub:	0.953 x Pu	**= 298 kips**	H= Horizontal Force
Vuc:	0.149 x Pu	**= 47 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	3/16	Weld to Column Size:	3/16
Interaction for Vub:	0.075	Interaction for Vuc:	0.699
Interaction for Hub:	0.694	Interaction for Huc:	0.000
Total Interaction:	**0.768**	Total Interaction:	**0.699**

Gusset Design

Buckling Capacity:	276k OK
Yielding Capacity:	670k OK
Block Shear Capacity:	551k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF7 Top, Gusset 13

GENERAL CRITERIA

Frame Height:	12.17 ft	Beam Size:	W8X40
Frame Length:	9.08 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS4X4X3/8
Gusset Thickness:	0.75 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

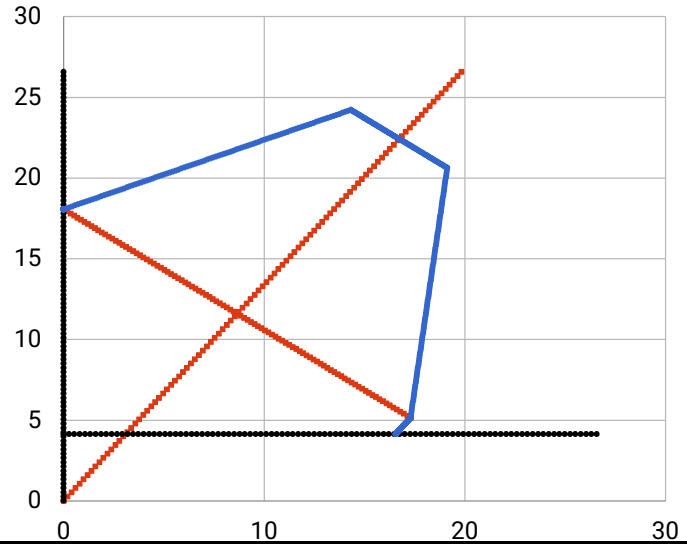
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	12.17 ft
Frame Length:	9.08 ft
Brace Length:	15.19 ft
Angle from Horz:	53.27 deg
Angle from Vert:	36.73 deg

Gusset Plate Geometry

Brace on Gusset:	12.00 in
Length along Beam:	16.55 in
Length along Column:	13.92 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	4.13 in
a:	8.68 in
b:	7.51 in
r:	14.52

Max Brace Force, Pu **= 312 kips**

Vub:	0.284 x Pu	**= 89 kips**	V= Vertical Force
Hub:	0.598 x Pu	**= 187 kips**	H= Horizontal Force
Vuc:	0.517 x Pu	**= 162 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	6/16	Weld to Column Size:	4/16
Interaction for Vub:	0.205	Interaction for Vuc:	0.969
Interaction for Hub:	0.645	Interaction for Huc:	0.000
Total Interaction:	**0.849**	Total Interaction:	**0.969**

Gusset Design

Buckling Capacity:	276k OK
Yielding Capacity:	670k OK
Block Shear Capacity:	551k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF7 Bottom, Gusset 14

GENERAL CRITERIA

Frame Height:	12.17 ft	Beam Size:	BasePlate
Frame Length:	9.08 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS4X4X3/8
Gusset Thickness:	0.75 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

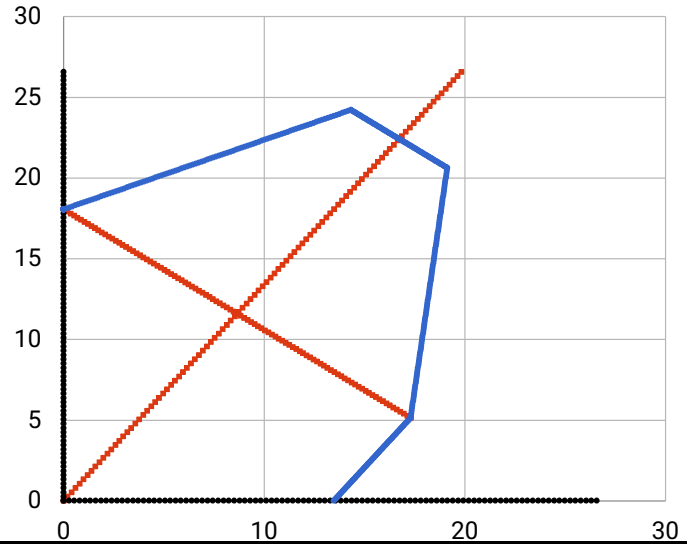
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	12.17 ft
Frame Length:	9.08 ft
Brace Length:	15.19 ft
Angle from Horz:	53.27 deg
Angle from Vert:	36.73 deg

Gusset Plate Geometry

Brace on Gusset:	12.00 in
Length along Beam:	13.47 in
Length along Column:	18.05 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	0.00 in
a:	7.17 in
b:	9.61 in
r:	11.99

Max Brace Force, Pu = **312 kips**

Vub:	0.000 x Pu	**= 0 kips**	V= Vertical Force
Hub:	0.598 x Pu	**= 187 kips**	H= Horizontal Force
Vuc:	0.801 x Pu	**= 251 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	5/16
Interaction for Vub:	0.000	Interaction for Vuc:	0.939
Interaction for Hub:	0.937	Interaction for Huc:	0.000
Total Interaction:	**0.937**	Total Interaction:	**0.939**

Gusset Design

Buckling Capacity:	276k OK
Yielding Capacity:	670k OK
Block Shear Capacity:	551k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF8 Top, Gusset 15

GENERAL CRITERIA

Frame Height:	7.79 ft	Beam Size:	W8X40
Frame Length:	9.08 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS4X4X3/8
Gusset Thickness:	0.75 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

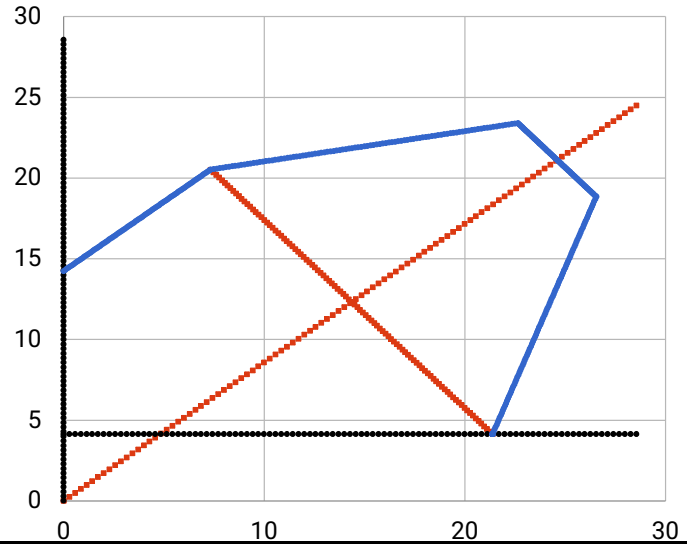
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	7.79 ft
Frame Length:	9.08 ft
Brace Length:	11.97 ft
Angle from Horz:	40.63 deg
Angle from Vert:	49.37 deg

Gusset Plate Geometry

Brace on Gusset:	12.00 in
Length along Beam:	21.39 in
Length along Column:	10.10 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	4.13 in
a:	11.26 in
b:	5.53 in
r:	14.83

Max Brace Force, Pu **= 312 kips**

Vub:	0.278 x Pu	**= 87 kips**	V= Vertical Force
Hub:	0.759 x Pu	**= 237 kips**	H= Horizontal Force
Vuc:	0.373 x Pu	**= 117 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	4/16
Interaction for Vub:	0.185	Interaction for Vuc:	0.950
Interaction for Hub:	0.756	Interaction for Huc:	0.000
Total Interaction:	**0.942**	Total Interaction:	**0.950**

Gusset Design

Buckling Capacity:	276k OK
Yielding Capacity:	670k OK
Block Shear Capacity:	551k OK

SCBF GUSSET DESIGN

Michael Baker INTERNATIONAL

BF8 Bottom, Gusset 16

GENERAL CRITERIA

Frame Height:	7.79 ft	Beam Size:	W8X40
Frame Length:	9.08 ft	Column Size:	BasePlate
Floor Thickness:	0.00 in	Brace Size:	HSS4X4X3/8
Gusset Thickness:	0.75 in	Brace Grade:	A500 Gr C
Measured from Top of Slab:	No	Gusset Plate Fy:	50 ksi
		Weld Size Brace to Gusset:	5 /16
		Corner Snip:	1.00 in

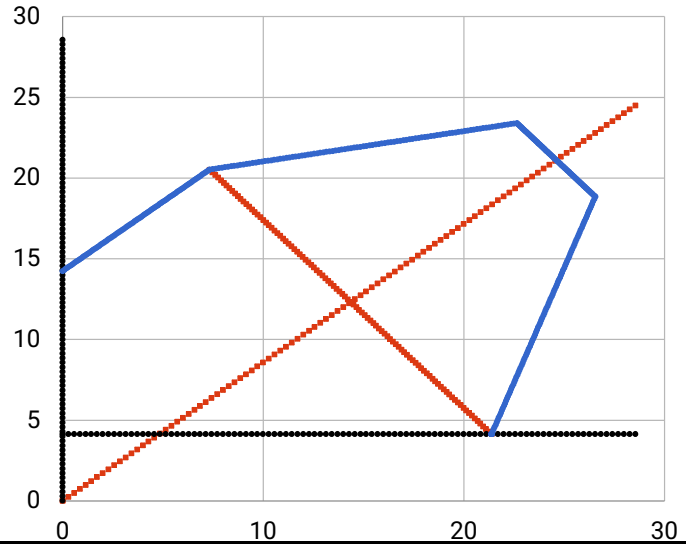
ANALYSIS AND CALCULATIONS

Frame Geometry (Work-Point to Work-Point)

Frame Height:	7.79 ft
Frame Length:	9.08 ft
Brace Length:	11.97 ft
Angle from Horz:	40.63 deg
Angle from Vert:	49.37 deg

Gusset Plate Geometry

Brace on Gusset:	12.00 in
Length along Beam:	21.39 in
Length along Column:	10.10 in



DESIGN OUTPUT

Uniform Force Method

ec:	0.00 in
eb:	4.13 in
a:	11.26 in
b:	5.53 in
r:	14.83

Max Brace Force, Pu **= 312 kips**

Vub:	0.278 x Pu	**= 87 kips**	V= Vertical Force
Hub:	0.759 x Pu	**= 237 kips**	H= Horizontal Force
Vuc:	0.373 x Pu	**= 117 kips**	b= Beam Side
Huc:	0.000 x Pu	**= 0 kips**	c= Column Side

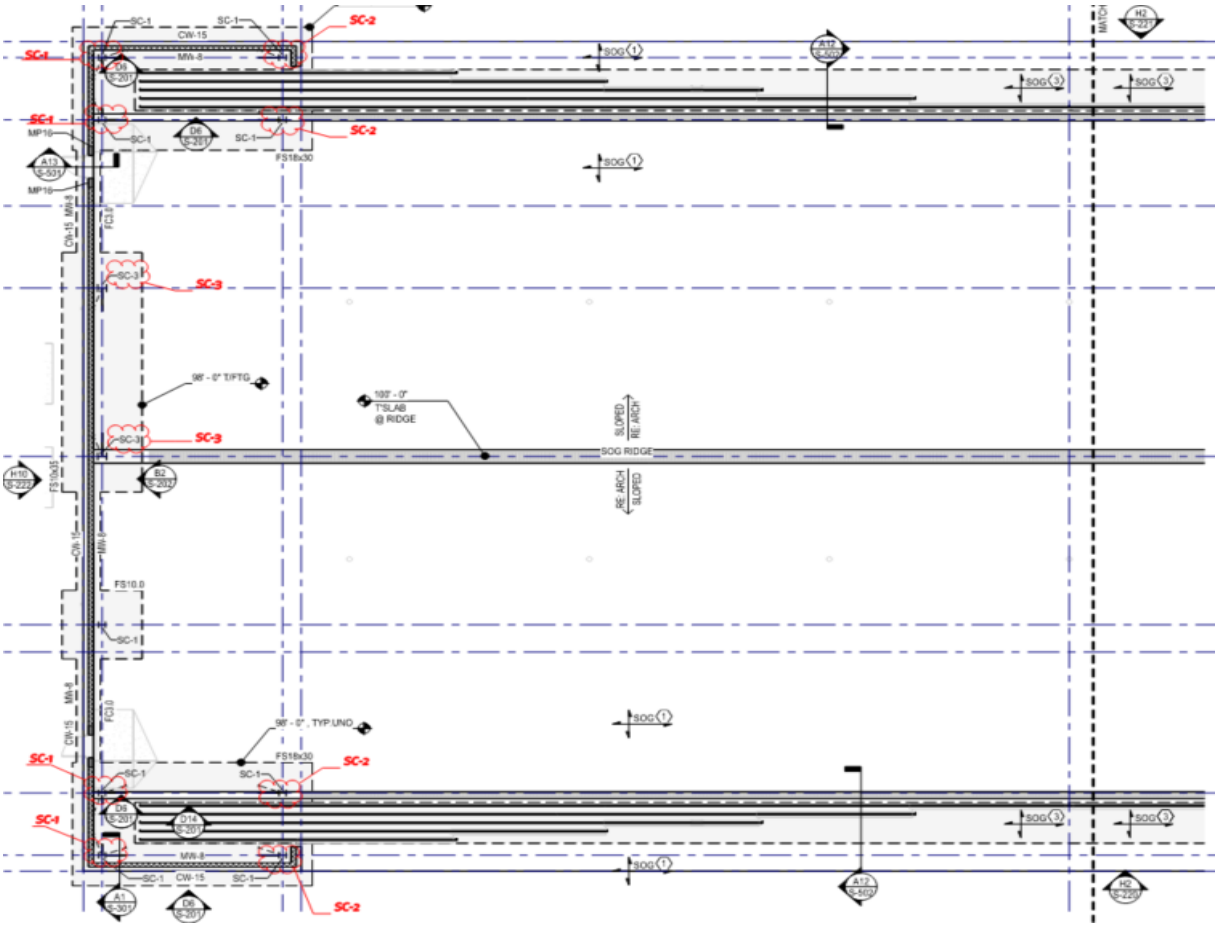
Welding of Gusset Connection

Weld to Beam Size:	5/16	Weld to Column Size:	4/16
Interaction for Vub:	0.185	Interaction for Vuc:	0.950
Interaction for Hub:	0.756	Interaction for Huc:	0.000
Total Interaction:	**0.942**	Total Interaction:	**0.950**

Gusset Design

Buckling Capacity:	276k OK
Yielding Capacity:	670k OK
Block Shear Capacity:	551k OK

BASEPLATE PLAN LOCATIONS



SCBF Column Axial Loads

	Cloumn Label	Axial load D (kips)	Axial load Ex (kips)	Axial load Ey (kips)
Grid 3.9	69	28	17	35
	361	7	16	28
	335	116	2	49
	406	86	1	45
	419	1	16	29
	184	31	17	36
Grid 1.1	479	30	17	36
	430	1	16	29
	404	86	1	45
	475	116	2	48
	486	7	16	28
	483	28	17	35
Grid A.1	5	61	16	8
	29	61	16	8
Grid A.3	91	30	17	14
	93	28	17	14
Grid D.7	154	28	17	14
	162	30	17	14
Grid D.9	108	61	16	8
	110	61	16	8

	HSS 4x4x3/8		HSS 8x8x5/8		HSS 8x8x5/8	
Lc	11.75	ft	36	ft	44	ft
r	1.47	in	2.99	in	2.99	in
Ag	4.78	sq in	16.4	sq in	16.4	sq in
Lc/r	95.918367		144.4816		176.5886	
Ry	1.3		1.3		1.3	
Fy	50	ksi	50	ksi	50	ksi
E	29000	ksi	29000	ksi	29000	ksi
4.71 (E/RyFy)^.5	99.486288		99.48629		99.48629	
Fe	31.109583	ksi	13.71111	ksi	9.178512	ksi
Fne	27.109197	ksi	12.02464	ksi	8.049555	ksi
Pn	130	kip	197	kip	132	kip
RyFyAg	310.7	kip	1066	kip	1066	kip
(1/.877)*Fne*Ag	148	kip	225	kip	151	kip

Base Plate	**SC1**		**SC1**		**SC2**		**SC3**	
Gr Anchor	105	ksi	105	ksi	105	ksi	105	ksi
tp	1.875	in	1.5	in	1.75	in	1.875	in
M	48	in	30		48		45	
N	20	in	20		20		20	
Brace angle	52.81	deg from horiz	52.81	deg from horiz	50.96	deg from horiz	55.51	deg from horiz
P(DL)	28		1		28		86	
P(E)	35		29		17		45	
Pu Column	48		57		12		23	
Tension	897	k	305	k	840	k	901	k
# Anchors	10		6		6		6	
Pu anchor	90	k	51	k	140	k	150	k
As rod	0.95	sq in	0.54	sq in	1.48	sq in	1.59	sq in
phi rod	1.10	sq in	0.83	sq in	1.37	sq in	1.42	sq in
Check base plate in bending to confirm rigid body assumption								
e	1.75	in	1.75	in	1.75	in	1.75	in
Mu	314.1	k-in	177.8	k-in	490.0	k-in	525.8	k-in
Z	12.3	cu in	7.9	cu in	10.7	cu in	12.3	cu in
stress	25.52	ksi	22.57	ksi	45.72	ksi	42.73	ksi
Embed plate								
wu	59.8	k-in	20.3	k-in	56.0	k-in	60.1	k-in
Mu	1682.5	k-in	571.4	k-in	1575.1	k-in	1690.1	k-in
Zx	37.4	cu in	12.7	cu in	35.0	cu in	37.6	cu in
t	1.77	in	1.30	in	1.71	in	1.83	in
Conc Breakout								
d	27	in	27	in	27	in	27	in
bo	224	in	188	in	224	in	218	in
phi Vc	1217	k	1022	k	1217	k	1185	k
Shear	644	k	188	k	671	k	604	k
See shear lug calculation								

Shear Lug Design

SC 1 - HSS8x8x5/8 Brace

Michael Baker INTERNATIONAL

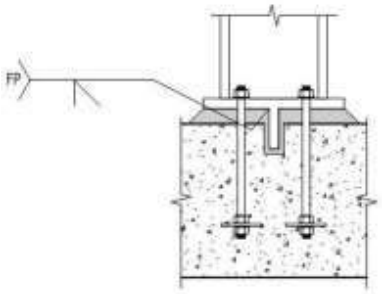
GENERAL DESIGN CRITERIA

Base Plate Length:	48.00 in	Factored Shear Load (Vu):	644 kips
Base Plate Width:	20.00 in	Grout Thickness:	2.00 in
Shear Lug Width:	20.00 in	Concrete Strength (f'c):	4500 psi
Base Plate Thickness:	1.875 in	Lug Yield Strength (Fy):	50 ksi
# of Lugs Used:	2		

ANALYSIS AND DESIGN CALCULATION

Single Shear Lug Design

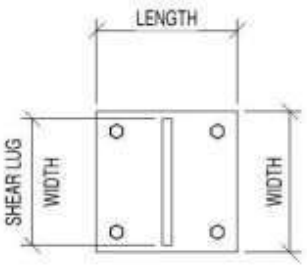
Max Shear (Vu):	322.00	kips
Lug Area Req'd:	129.51	in2
Min Lug Depth:	8.48	in
Actual Lug Depth:	8.50	in
Lug Moment (Mu):	84.53	k-in
Min Lug Thickness:	2.741	in
Lug Thickness:	2.750	in



SHEAR LUG/BASE PLATE SECTION VIEW

Final Shear Lug Design

Shear Lug Size:	**(2) 8.5 in x 2.3/4 in x 20 in.**	
Shear Lug Thickness:	2.750	in.
Shear Lug Depth:	8.50	in.
Lug Face to Base Plate edge:	15.00 in	in.
Spacing Between Lugs:	12.50 in	in.



SHEAR LUG/BASE PLATE PLAN VIEW

Shear Lug Design

SC 2 - HSS8x8x5/8 Brace

Michael Baker INTERNATIONAL

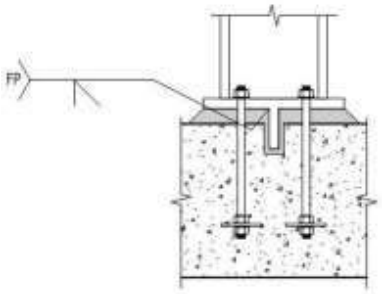
GENERAL DESIGN CRITERIA

Base Plate Length:	48.00 in	Factored Shear Load (Vu):	671 kips
Base Plate Width:	20.00 in	Grout Thickness:	2.00 in
Shear Lug Width:	20.00 in	Concrete Strength (f'c):	4500 psi
Base Plate Thickness:	1.750 in	Lug Yield Strength (Fy):	50 ksi
# of Lugs Used:	2		

ANALYSIS AND DESIGN CALCULATION

Single Shear Lug Design

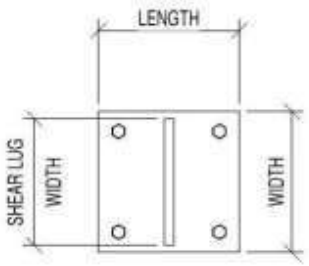
Max Shear (Vu):	335.50	kips
Lug Area Req'd:	134.94	in2
Min Lug Depth:	8.75	in
Actual Lug Depth:	9.00	in
Lug Moment (Mu):	92.26	k-in
Min Lug Thickness:	2.864	in
Lug Thickness:	3.000	in



SHEAR LUG/BASE PLATE SECTION VIEW

Final Shear Lug Design

Shear Lug Size:	**(2) 9 in x 3 in x 20 in.**	
Shear Lug Thickness:	3.000	in.
Shear Lug Depth:	9.00	in.
Lug Face to Base Plate edge:	14.50 in	in.
Spacing Between Lugs:	13.00 in	in.



SHEAR LUG/BASE PLATE PLAN VIEW

Shear Lug Design

SC 3 - HSS8x8x5/8 Brace

Michael Baker INTERNATIONAL

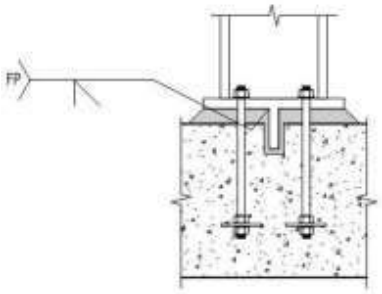
GENERAL DESIGN CRITERIA

Base Plate Length:	45.00 in	Factored Shear Load (Vu):	604 kips
Base Plate Width:	20.00 in	Grout Thickness:	2.00 in
Shear Lug Width:	20.00 in	Concrete Strength (f'c):	4500 psi
Base Plate Thickness:	1.875 in	Lug Yield Strength (Fy):	50 ksi
# of Lugs Used:	2		

ANALYSIS AND DESIGN CALCULATION

Single Shear Lug Design

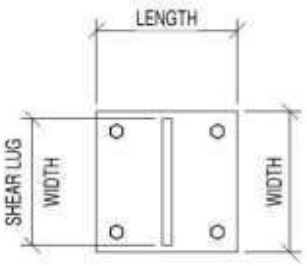
Max Shear (Vu):	302.00	kips
Lug Area Req'd:	121.47	in2
Min Lug Depth:	8.07	in
Actual Lug Depth:	8.50	in
Lug Moment (Mu):	79.28	k-in
Min Lug Thickness:	2.655	in
Lug Thickness:	2.750	in



SHEAR LUG/BASE PLATE SECTION VIEW

Final Shear Lug Design

Shear Lug Size:	**(2) 8.5 in x 2.3/4 in x 20 in.**	
Shear Lug Thickness:	2.750	in.
Shear Lug Depth:	8.50	in.
Lug Face to Base Plate edge:	13.50 in	in.
Spacing Between Lugs:	12.50 in	in.



SHEAR LUG/BASE PLATE PLAN VIEW

Tekla Tedds	Project				Job Ref.	
	Creech DRP 2					
	Section				Sheet no./rev.	
	AMF Grid D/3.9 Base Plate				1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	BT	11/6/2025				

COLUMN BASE PLATE DESIGN

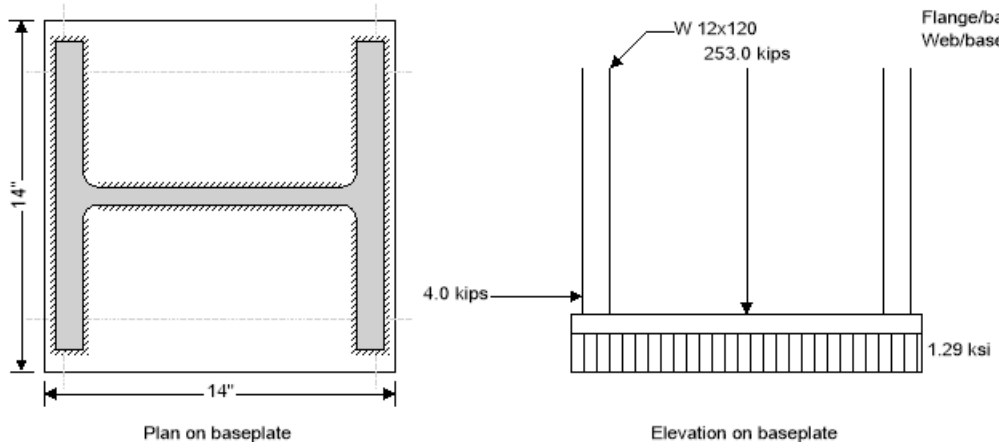
In accordance with AISC Steel Design Guide 1 and AISC 360-22

Tedds calculation version 2.1.14

Design summary

Overall design status PASS

Description	Unit	Required	Provided	Utilization	Result
Plate thickness	(in)	0.587	0.750	0.783	PASS
Bearing strength	(kips)	253.00	974.61	0.260	PASS
Flange weld strength	(kips/in)	0.000	10.441	0.000	PASS
Shear weld strength	(kips/in)	0.184	6.961	0.026	PASS
Frictional shear resistance	(kips)	4.00	68.31	0.059	PASS



Design forces and moments

Axial force	P_u = **253.0** kips (Compression)
Bending moment	M_u = **0.0** kip_in
Shear force	F_v = **4.0** kips

Column details

Column section	W 12x120
Depth	d = **13.100** in
Breadth	b_f = **12.300** in
Flange thickness	t_f = **1.110** in
Web thickness	t_w = **0.710** in

Baseplate details

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

Foundation geometry

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

Dist center of baseplate to top edge foundation	y_{ce2} = **30.000** in
Minimum tensile strength, base plate	F_y = **36** ksi
Minimum tensile strength, column	F_{yCol} = **50** ksi
Compressive strength of concrete	f'_c = **4.5** ksi
Strength reduction factors	
Compression	ϕ_c = **0.65**
Flexure	ϕ_b = **0.90**
Weld shear	ϕ_v = **0.75**
Plate cantilever dimensions	
Minimum distance to edge of concrete	$l_{min} = \min(\min(x_{ce1}, x_{ce2}) - N / 2 , \min(y_{ce1}, y_{ce2}) - B / 2)$ = **23.000** in
Area of base plate	$A_1 = B \times N$ = **196.000** in^2
Maximum area of supporting surface	$A_2 = (N + 2 \times l_{min})^2 \times (B / N)$ = **3600.000** in^2
Nominal strength of concrete under base plate	$P_p = 0.85 \times f'_c \times A_1 \times \min(\sqrt{(A_2 / A_1)}, 2)$ = **1499.4** kips
Plate bending coefficient X	$X = (4 \times d \times b_f) / (d + b_f)^2 \times \min(1.0, P_u / (P_p \times \phi_c))$ = **0.26**
Plate bending coefficient λ	$\lambda_l = \min(1, 2 \times \sqrt{(X)} / (1 + \sqrt{(1 - X)}))$ = **0.55**
Bending line cantilever distance m	$m = (N - 0.95 \times d) / 2$ = **0.778** in
Bending line cantilever distance n	$n = (B - 0.8 \times b_f) / 2$ = **2.080** in
Yield line theory cantilever distance n'	$n' = \sqrt{(d \times b_f)} / 4$ = **3.173** in
Maximum bending line cantilever	$l = \max(m, n, \lambda_l \times n')$ = **2.080** in
Plate thickness	
Required plate thickness	$t_{p,req} = l \times \sqrt{((2 \times P_u) / (\phi_b \times F_y \times B \times N))}$ = **0.587** in
Specified plate thickness	t_p = **0.750** in
	PASS - Thickness of plate exceeds required thickness
Design bearing strength (AISC 360-05-J8)	
Design bearing strength	P_p = **1499.40** kips
Factored bearing strength	$\phi_c P_p$ = **974.61** kips
	PASS - Allowable bearing stress exceeds applied bearing stress
Frictional shear resistance	
Steel / concrete friction coefficient	μ = **0.4**
Frictional shear resistance	$\phi V_n = \min(\phi_v \times \mu \times P_u, \phi_v \times 0.2 \times f'_c \times A_1, \phi_v \times 800psi \times A_1)$ = **68.31** kips
	PASS - Frictional shear resistance exceeds applied shear
Flange weld	
Flange weld leg length	t_{wf} = **0.3125** in
Tension capacity of flange	$P_{tf} = b_f \times t_f \times F_{yCol}$ = **682.7** kips
Force in tension flange	$F_{tf} = M_u / (d - t_f) - P_u \times (b_f \times t_f) / A_{col}$ = **-98.1** kips
Critical force in flange	$F_f = \min(P_{tf}, \max(F_{tf}, 0kips))$ = **0.0** kips
Flange weld force per in	$R_{wf} = F_f / (2 \times b_f - t_w)$ = **0.0** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(90deg))^{1.5})$ = **47.250**ksi
Design strength of weld per in	$\phi R_{nf} = \phi F_{nw} \times t_{wf} / \sqrt{(2)}$ = **10.4** kips/in
	PASS - Available strength of flange weld exceeds force in flange weld

Shear weld

Shear web weld leg length	t_{ww} = **0.3125** in
Shear web weld force per in	$R_{wl} = F_v / (2 \times (d - 2 \times t_f))$ = **0.184** kips/in
Electrode classification number	F_{EXX} = **70.0** ksi
Design weld stress	$\phi F_{nw} = \phi_v \times 0.60 \times F_{EXX} \times (1.0 + 0.5 \times (\sin(0deg))^{1.5})$ = **31.500**ksi
Design strength of weld per in	$\phi R_{nl} = \phi F_{nw} \times t_{ww} / \sqrt{(2)}$ = **7.0** kips/in

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

Hilti PROFIS Engineering 3.1.14

www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Apr 23, 2025 (1)	Date:	4/23/2025
Fastening point:	Wind Grit Anchor		

Specifier's comments:

1 Input data

Anchor type and diameter:	**HY 270 + threaded rod 5.8 5/8**
Item number:	385428 HAS 5.8 5/8"x8" (element) / 2194247 HIT-HY 270 (adhesive)
Specification text:	Hilti ∅ 5/8 HY 270 + threaded rod 5.8 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII,
Effective embedment depth:	h_{ef} = 5.625 in.
Material:	5.8
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	
Profile:	
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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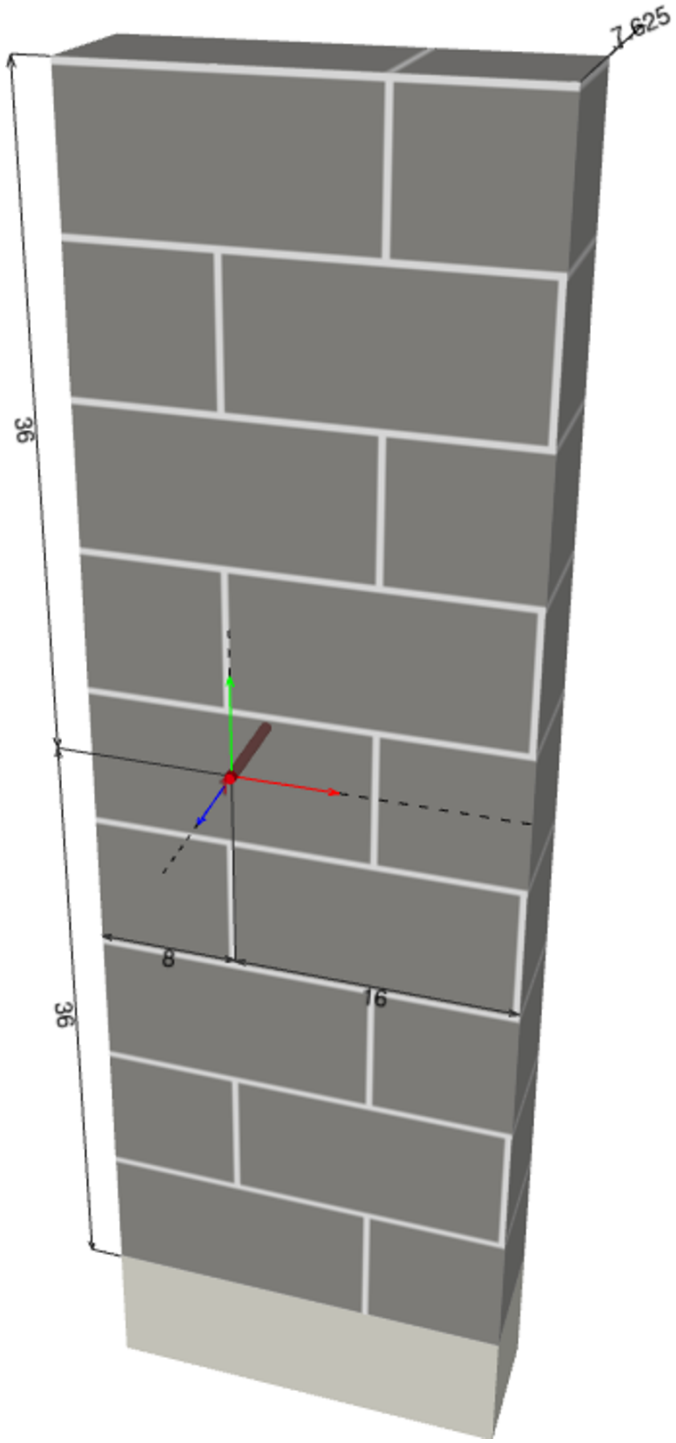
Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Apr 23, 2025 (1)	Date:	4/23/2025
Fastening point:	Wind Grit Anchor		

Geometry [in.]



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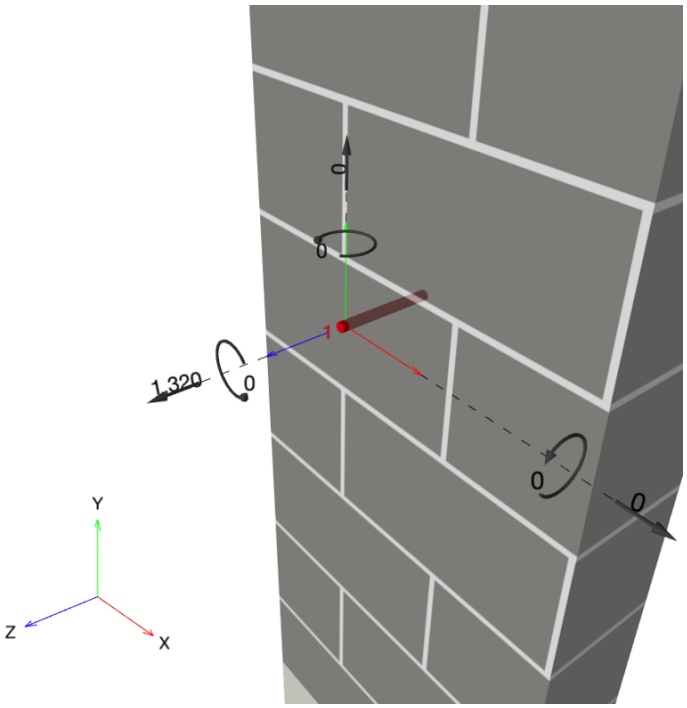
Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Apr 23, 2025 (1)	Date:	4/23/2025
Fastening point:	Wind Grit Anchor		

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 = 1,320; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	65

Wp = 126x10x16/12 = 1700 lb
Ka = 1+Lf/100 = 1+10/100 = 1.1
Fp = 0.4 SDS Ka Ie Wp = 0.4x0.59x1.1x1.25x1700 = 552 lb

2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1,320	0	0	0

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Apr 23, 2025 (1)	Date:	4/23/2025
Fastening point:	Wind Grit Anchor		

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Steel strength	1,320	7,340	18	OK
Bond strength	1,320	2,061	65	OK

3.1 Steel strength

$P_{t,s}$ = Value refer to Hilti Technical Data
$P_{t,s} \geq P_s$

Results

$P_{t,s}$ [lb]	P_s [lb]
7,340	1,320

3.2 Bond strength

$P_{t,b,Base}$ = Value refer to Hilti Technical Data
$P_{t,b} = P_{t,b,Base} \cdot f_{red,E} \cdot f_{red,s} \cdot f_{red,Temp} \cdot f_{red,Bedjoint}$
$P_{t,b} \geq P_s$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	22.500	68

Results

$P_{t,b}$ [lb]	$P_{t,b,Base}$ [lb]	P_s [lb]	$f_{red,E}$	$f_{red,S}$	$f_{red,Temp}$	$f_{red,Bedjoint}$
2,061	2,840	1,320	0.726	1.000	1.000	1.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Apr 23, 2025 (1)	Date:	4/23/2025
Fastening point:	Wind Grit Anchor		

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Overall strength	N/A	N/A	N/A	N/A

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 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 CBFEM 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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Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Apr 23, 2025 (1)	Date:	4/23/2025
Fastening point:	Wind Grit Anchor		

6 Installation data

Profile: -

Anchor type and diameter: HY 270 + threaded rod 5.8 5/8
Item number: 385428 HAS 5.8 5/8"x8" (element) / 2194247
HIT-HY 270 (adhesive)

Hole diameter in the fixture: -

Maximum installation torque: 90 in.lb

Plate thickness (input): -

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 5.625 in.

Drilling method: Drilled in hammer mode

Minimum thickness of the base material: 7.625 in.

Hilti ∅ 5/8 HY 270 + threaded rod 5.8 with 5.625 in nominal embedment depth per Technical data , Hammer drilled installation per MPII

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	8.000	16.000	36.000	36.000

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Apr 23, 2025 (1)	Date:	4/23/2025
Fastening point:	Wind Grit Anchor		

7 Remarks; Your Cooperation Duties

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S.O. No. _____

Subject: Hangar - Extreme Torsional Irregularity and Drift Check

Sheet No. _____ of _____

Drawing No. _____

Computed by BT Checked by _____ Date 10/10/2025

Michael Baker INTERNATIONAL

$\delta_{ave} = \frac{\delta_{max} + \delta_{min}}{2}$ $\frac{\delta_{max}}{\delta_{ave}} > 1.4 \rightarrow$ extreme

under Ex (Seismic E/W direction) in ETABS:

$\delta_{max} = 0.078''$ $\delta_{min} = 0.05''$ $\delta_{ave} = 0.064$

$\frac{\delta_{max}}{\delta_{ave}} < 1.4$ ✓

under Ey (Seismic N/S direction) in ETABS:

$\delta_{max} = 0.15''$ $\delta_{min} = 0.095''$ $\delta_{ave} = 0.1225''$

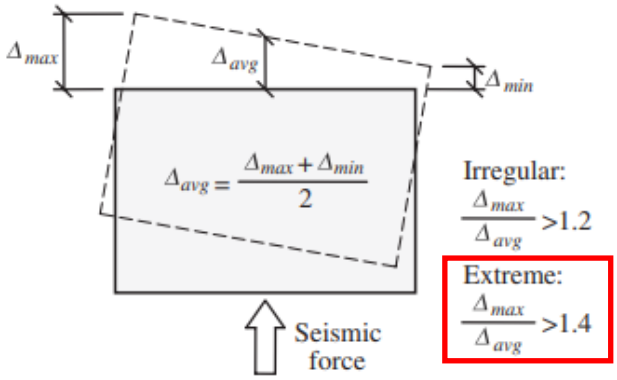
$\frac{\delta_{max}}{\delta_{ave}} < 1.4$ ✓

Allowable story drift for Seismic:

Risk category III $\rightarrow \Delta_a = 0.015 h_x = 6.12'' > 0.15''$ ✓

max drift under wind from ETABS = 0.5"

Allow drift for wind = $H/600 = \frac{48 \times 12}{600} = 0.96'' > 0.5''$ ✓



Type 1. Torsional

ties required by this section shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective for providing the ties or struts required by this section.

12.11.2.2.4 Metal Deck Diaphragms. In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

12.11.2.2.5 Embedded Straps. Diaphragm to structural wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

12.12.3 Structural Separation. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact as set forth in this section.

Separations shall allow for the maximum inelastic response displacement (δ_M). δ_M shall be determined at critical locations with consideration for translational and torsional displacements of the structure including torsional amplifications, where applicable, using the following equation:

$$\delta_M = \frac{C_d \delta_{max}}{I_e} \qquad (12.12\text{-}1)$$

where δ_{max} = maximum elastic displacement at the critical location.

Table 12.12-1 Allowable Story Drift, Δ_a [a,b]

Structure	Risk Category I or II	Risk Category III	Risk Category IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

CC.2.2 Drift of Walls and Frames. Drifts (lateral deflections) of concern in serviceability checking arise primarily from the effects of wind. Drift limits in common usage for building design are on the order of 1/600 to 1/400 of the building or story height (ASCE Task Committee on Drift Control of Steel Building Structures 1988; Griffis 1993). These limits generally are

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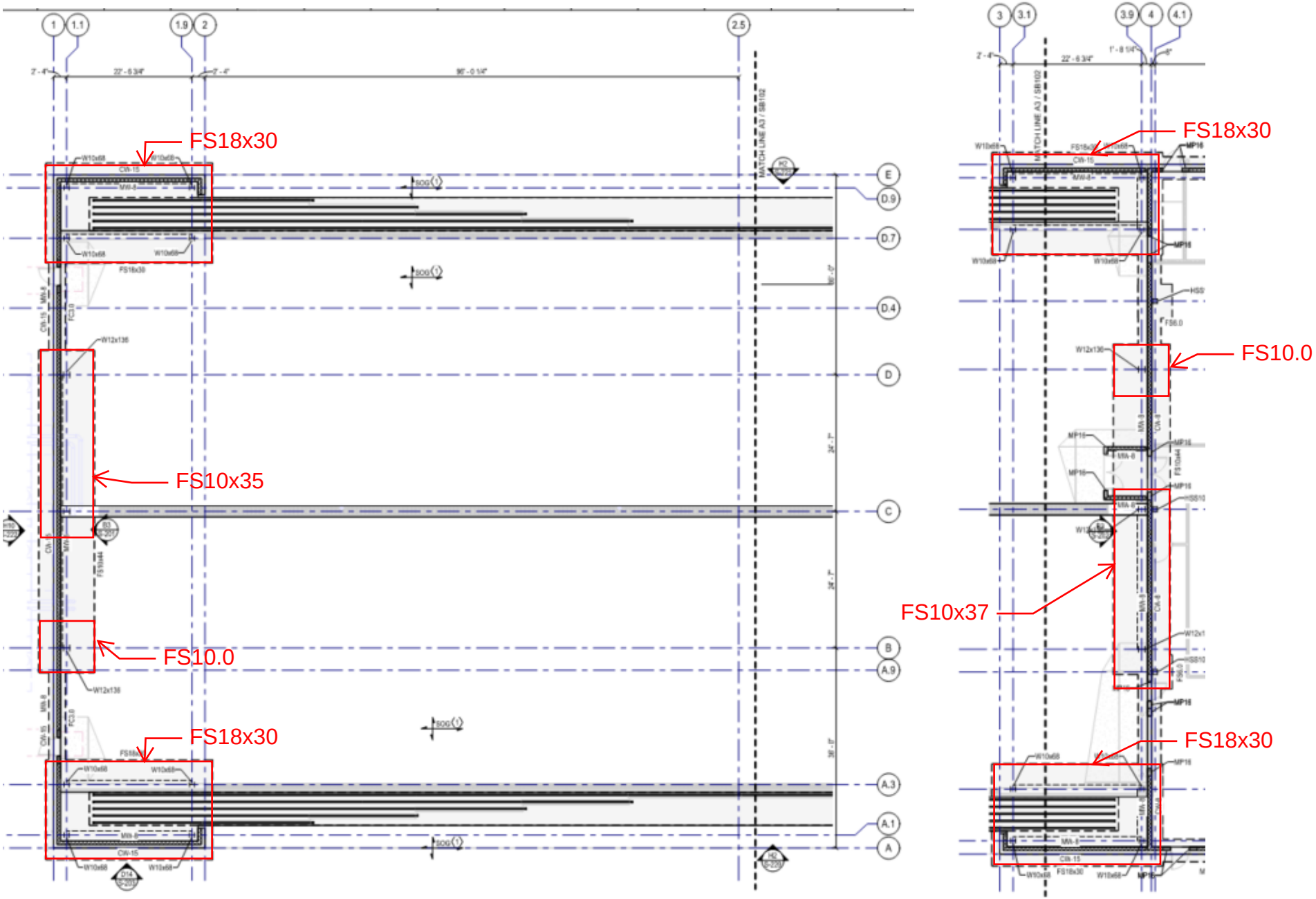
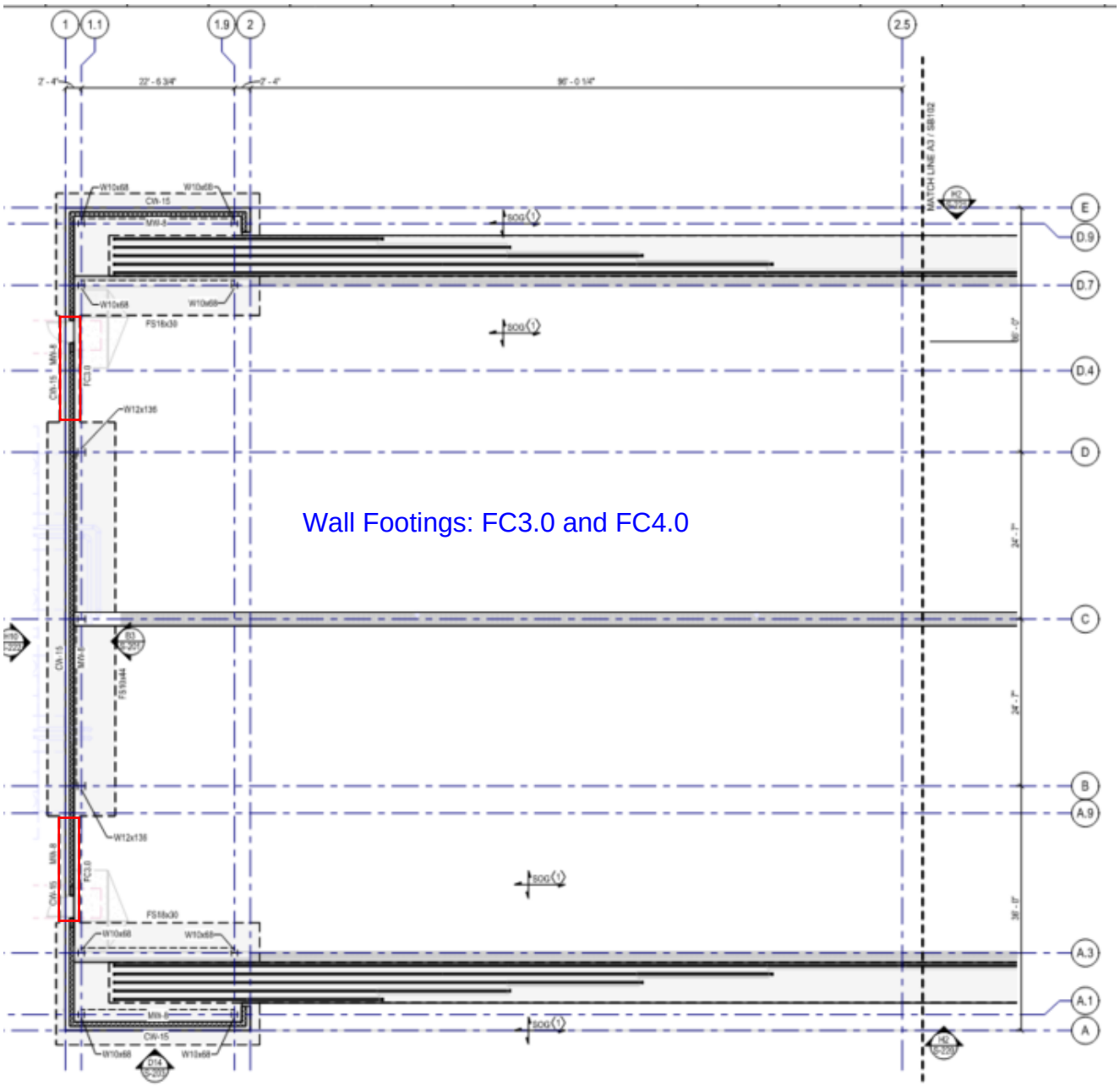
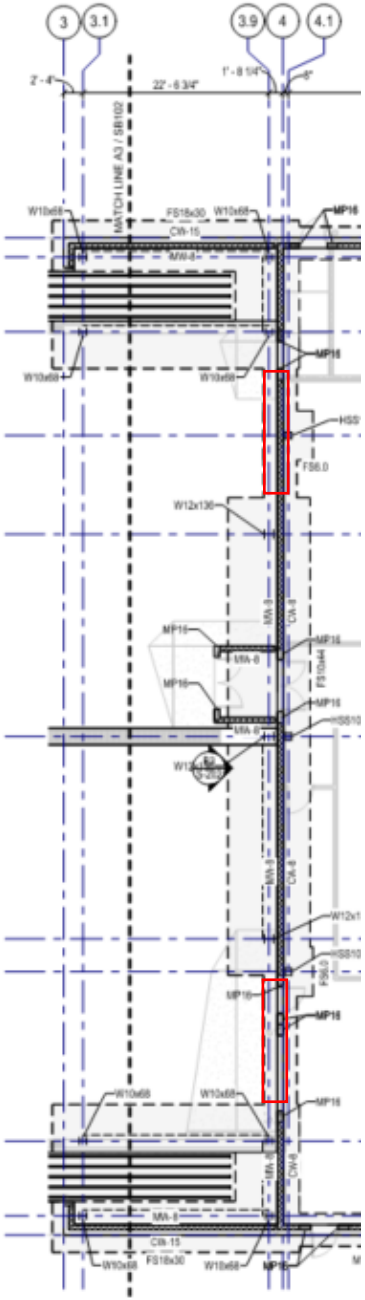
Foundation Calculations

Overall Foundation Plan with Labels



Wall Loads and Wall Footing Layout





As CMU walls are only self-weight and do not act as bearing or shear walls, wall loads are only their self-weight.

8'' CMU walls: DL = 126x29.5 = 3717 plf

Tekla Tedds	Project: Creech Part 2 - Hangar				Job Ref.	
	Section: CMU Wall Footing				Sheet no./rev.: 1	
	Calc. by: BT	Date: 10/10/2025	Chk'd by	Date	App'd by	Date

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status PASS

Overall design utilisation 0.706

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	5.1			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.119	3	0.706	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, y-direction	kip_ft	1.4	11.8	0.116	Pass
Shear, one-way, y-direction	kips	0.9	5.7	0.159	Pass
Min.area of reinf, bot., y-direction	in^2	0.259	0.310		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	12.0		Pass
Min.area of reinf, S&T, x-direction	in^2	0.778	0.930		Pass
Max.reinf.spacing, S&T, x-direction	in	18.0	12.0		Pass

Strip footing details - considering a one foot strip

Length of footing L_x = **1** ft

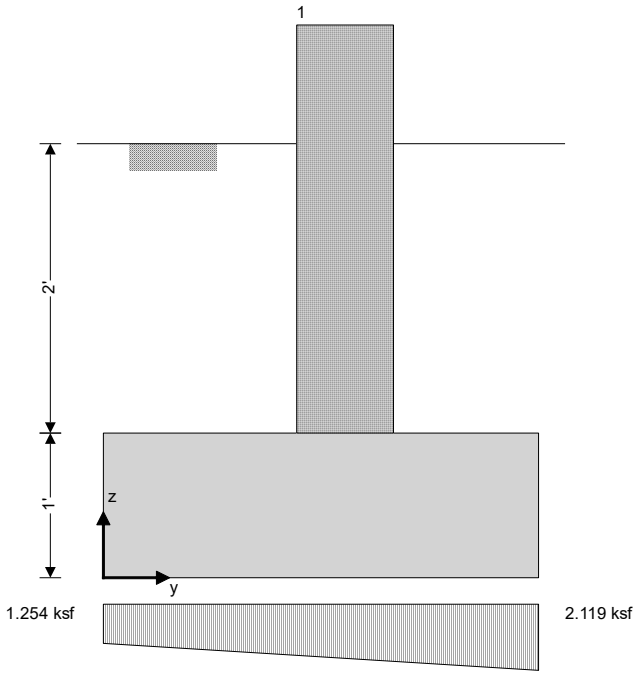
Width of footing L_y = **3** ft

Footing area $A = L_x \times L_y$ = **3** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **24** 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 = **20** 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	$F_{swt} = h \times \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **240** psf

Wall no.1 loads per linear foot

Dead load in z	F_{Dz1} = **1.5** kips
Live load in z	F_{Lz1} = **2.5** kips
Live roof load in z	F_{Lrz1} = **0.9** kips
Snow load in z	F_{Sz1} = **0.8** kips
Rain load in z	F_{Rz1} = **0.3** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.329)
1.0D + 1.0L (0.699)
1.0D + 1.0Lr (0.462)
1.0D + 0.7S (0.406)

1.0D + 1.0R (0.373)
1.0D + 0.75L + 0.75Lr (0.706)
1.0D + 0.75L + 0.525S (0.665)
1.0D + 0.75L + 0.75R (0.640)
1.0D + 0.6W (0.329)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.355)
1.0D + 0.75L + 0.75Lr + 0.45W (0.706)
1.0D + 0.75L + 0.525S + 0.45W (0.665)
1.0D + 0.75L + 0.75R + 0.45W (0.640)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.637)
0.6D + 0.6W (0.197)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.170)

Combination 6 results: 1.0D + 0.75L + 0.75Lr

Forces on footing per linear foot

Force in z-axis

$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1}$ = **5.1** kips

Moments on footing per linear foot

Moment in y-axis, about y is 0

$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **8.2** kip_ft

Uplift verification

Vertical force

F_{dz} = **5.06** kips

PASS - Footing is not subject to uplift

Stability against sliding

Resistance due to base friction

$F_{RFriction} = \max(F_{dz}, 0\ kN) \times \tan(\delta_{bb})$ = **2.921** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

$e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **1.538** in

Strip base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **1.254** ksf

$q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **2.119** ksf

Minimum base pressure

$q_{min} = \min(q_1, q_2)$ = **1.254** ksf

Maximum base pressure

$q_{max} = \max(q_1, q_2)$ = **2.119** ksf

Allowable bearing capacity

Allowable bearing capacity

$q_{allow} = q_{allow_Gross}$ = **3** ksf

q_{max} / q_{allow} = **0.706**

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 = **4000** 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**
Wall type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-22

1.4D (0.049)
1.2D + 1.6L + 0.5Lr (0.159)
1.2D + 1.6L + 0.3S (0.153)
1.2D + 1.6L + 0.5R (0.151)
1.2D + 1.0L + 1.6Lr (0.146)
1.2D + 1.0L + 1.0S (0.127)
1.2D + 1.0L + 1.6R (0.120)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing per linear foot

Ultimate force in z-axis

$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1}$ = **7.5** kips

Moments on footing per linear foot

Ultimate moment in y-axis, about y is 0

$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1)$ = **12.2** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

$e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **1.624** in

Strip base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1\text{ ft})$ = **1.814** ksf

$q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1\text{ ft})$ = **3.16** ksf

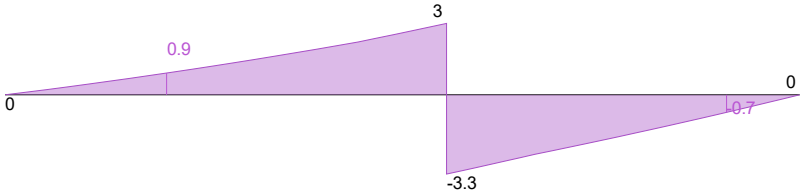
Minimum ultimate base pressure

$q_{umin} = \min(q_{u1}, q_{u2})$ = **1.814** ksf

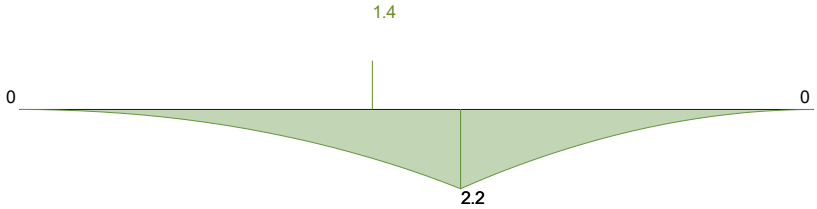
Maximum ultimate base pressure

$q_{umax} = \max(q_{u1}, q_{u2})$ = **3.16** ksf

Shear diagram (kips)



Moment diagram (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **1.375** kip_ft
Tension reinforcement provided	No.5 bars at 12.0 in c/c bottom
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **0.31** in^2
Minimum area of reinforcement (7.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **0.259** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (7.7.2.3)	$s_{max} = \min(3 \times h, 18\text{ in})$ = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom_b} - \phi_{y.bot} / 2$ = **8.688** in
Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.456** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.536** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04559**
Minimum tensile strain(7.3.3.1)	$\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **13.112** 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	$\phi M_n = \phi_f \times M_n$ = **11.801** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.116**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **0.904** kips
Depth to reinforcement	$d_v = h - c_{nom_b} - \phi_{y.bot} / 2$ = **8.687** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = A_{sy.bot.prov} / (L_x \times d_v)$ = **0.00297**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v)$ = **7.585** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **5.689** kips
	$V_{u.y} / \phi V_n$ = **0.159**
	PASS - Design shear capacity exceeds ultimate shear load

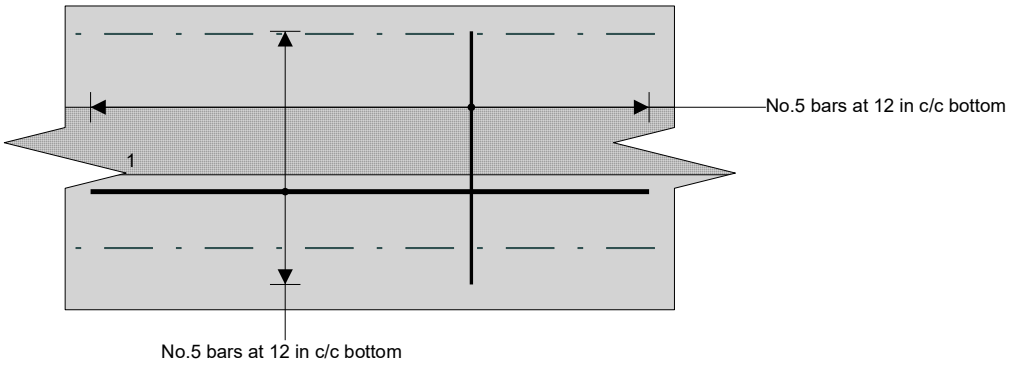
Shrinkage and temperature requirements, x direction

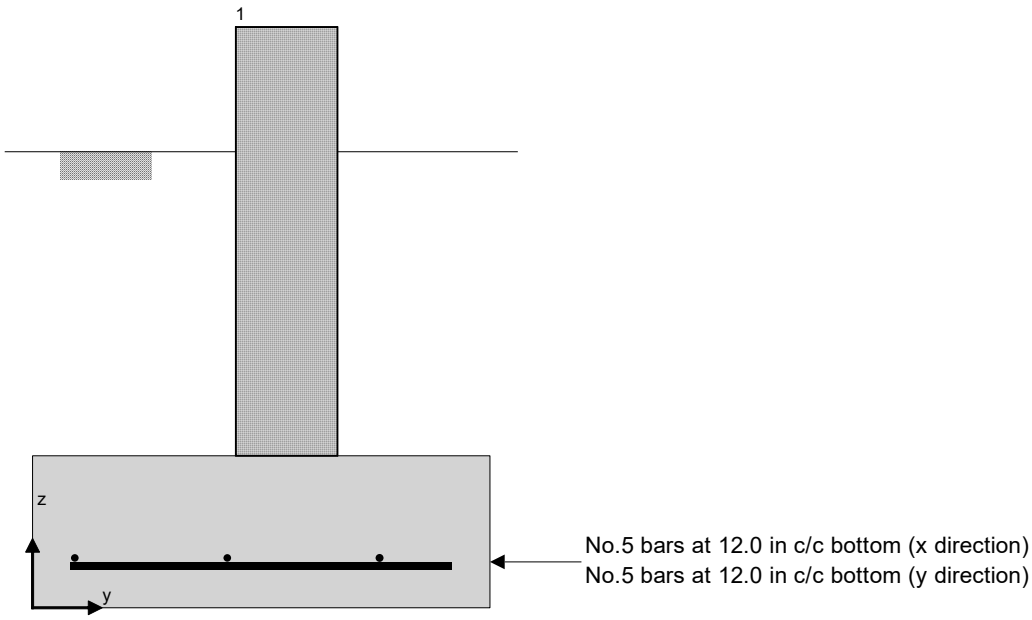
Reinforcement provided	No.5 bars at 12.0 in c/c bottom
Reinforcement area provided in full width	$A_{sx.prov}$ = **0.93** in^2
Minimum area of reinforcement (24.4.4.3.2)	$A_{s.min.ST} = 0.0018 \times L_y \times h$ = **0.778** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum reinforcement spacing (24.4.4.3.3) $s_{max.ST}$ = min(5 × h, 18 in) = **18** in

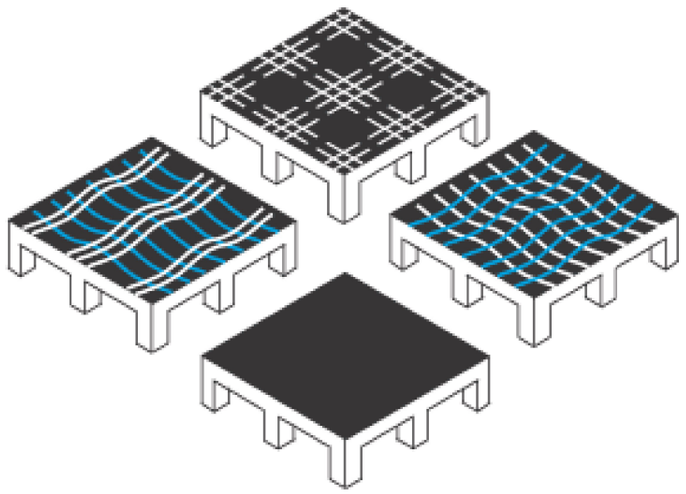
PASS - Maximum permissible reinforcement spacing exceeds actual spacing





Creech DRP Part 2

Mat Foundations



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Tue Nov 4 2025

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Units

Geometry Units

Plan Dimensions: feet
Angles: degrees
Slab Thickness: inches
Elevations: inches
Support Dimensions: inches
Support Height: feet

Loading and Reaction Units

Point Force: Kips
- Report As Zero: 0 Kips

Point Moment: kip-ft
- Report As Zero: 0 kip-ft

Line Force: kips/ft
- Report As Zero: 0 kips/ft

Line Moment: Kips
- Report As Zero: 0 Kips

Area Force: psf
- Report As Zero: 0 psf

Area Moment: #/foot
- Report As Zero: 0 #/foot

Spring and Stiffness Units

Point Force Spring: kips/in
Point Moment Spring: k-ft/°
Line Force Spring: ksi
Line Moment Spring: k/°
Area Force Spring: pci
Area Moment Spring: k/ft°

Slab Analysis Units

Force: Kips
- Report As Zero: 0 Kips

Force Per Width: kips/ft
- Report As Zero: 0 kips/ft

Moment: kip-ft
- Report As Zero: 0 kip-ft

Moment Per Width: Kips
- Report As Zero: 0 Kips

Concrete Stress: psi
- Report As Zero: 0 psi

Deflection: inches
- Report As Zero: 0 inches

Materials Units

Concrete Volume: yd³
Tendon Force: Kips
Reinforcing Stress: ksi
Reinforcing Area: in²
Tendon Force Per Width: kips/ft
PT Weight: pounds
Reinforcement Weight: tons
Tendon Profile: inches
Cover: inches

Miscellaneous Units

Floor Area: ft²
Tendon Angles (for friction): radians
Density: pcf
Temperature Change: °F
Elongations: inches

Materials

Concrete Mix

Mix Name	Density (pcf)	Density For Loads (pcf)	f'ci (psi)	f'c (psi)	fcui (psi)	fcu (psi)	Poissons Ratio	Thermal Exp Coeff	Ec Calc	User Eci (psi)	User Ec (psi)
3000 psi	150	150	3000	3000	3725	3725	0.2	5.556e-6	Code	2500000	3000000
4000 psi	150	150	3000	4000	3725	4975	0.2	5.556e-6	Code	2500000	3000000
5000 psi	150	150	3000	5000	3725	6399	0.2	5.556e-6	Code	2500000	3000000
6000 psi	150	150	3000	6000	3725	7450	0.2	5.556e-6	Code	2500000	3000000
4500 psi	150	150	3000	4500	3725	5590	0.2	5.556e-6	Code	2500000	3000000

PT Systems

System Name	Strand	Duct	Anchor	fse (ksi)	Long-Term Losses (ksi)	Min Radius (feet)
½" Unbonded	½" 7-Wire Strand	½" Unbonded Sheathing	Monostrand Anchor	175	22	6
0.6" Unbonded	0.6" 7-Wire Strand	0.6" Unbonded Sheathing	Monostrand Anchor	175	22	8
½" Bonded Flat Duct	½" 7-Wire Strand	½" Bonded Flat Duct	Flat Duct Anchor	175	22	6
0.6" Bonded Flat Duct	0.6" 7-Wire Strand	0.6" Bonded Flat Duct	Flat Duct Anchor	175	22	8

Strand Materials

Strand Name	Aps (in²)	Eps (ksi)	fpy (ksi)	fpu (ksi)
½" 7-Wire Strand	0.153	28000	243	270
0.6" 7-Wire Strand	0.217	28000	243	270

Duct Systems

Duct Name	Type	Shape	Material	Duct Height (inches)	Duct Width (inches)	Max Strands Per Duct	Wobble Friction (1/feet)	Angular Friction (1/radians)
½" Unbonded Sheathing	unbonded	Round	Tightly Sheathed	0.5	0.5	1	0.0014	0.07
0.6" Unbonded Sheathing	unbonded	Round	Tightly Sheathed	0.6	0.6	1	0.0014	0.07
½" Bonded Flat Duct	bonded	Flat	Corrugated Plastic	1	3	5	0.0002	0.14
0.6" Bonded Flat Duct	bonded	Flat	Corrugated Plastic	1	3	4	0.0002	0.14

Anchor Systems

Anchor Name	Type	Jacking Stress (ksi)	Anchor Friction	Seating Distance (inches)
Monostrand Anchor	Monostrand	216	0	0.25
Flat Duct Anchor	Flat Multi-Plane	216	0.02	0.25

Materials (2)

Reinforcing Bars

Bar Name	*As (in²)*	*Es (ksi)*	*Fy (ksi)*	*Coating*	*Straight Ld/Db*	*90 Hook Ld/Db*	*180 Hook Ld/Db*
#3	0.11	29000	60	None	Code	Code	Code
#4	0.2	29000	60	None	Code	Code	Code
#5	0.31	29000	60	None	Code	Code	Code
#6	0.44	29000	60	None	Code	Code	Code
#7	0.6	29000	60	None	Code	Code	Code
#8	0.79	29000	60	None	Code	Code	Code
#9	1	29000	60	None	Code	Code	Code
#10	1.27	29000	60	None	Code	Code	Code
#11	1.56	29000	60	None	Code	Code	Code

SSR Systems

SSR System Name	*Stud Area (in²)*	*Head Area (in²)*	*Min Clear Head Spacing (inches)*	*Specified Stud Spacing (inches)*	*Fy (ksi)*	*Stud Spacing Rounding Increment (inches)*	*Min Studs Per Rail*	*System Type*
3/8" SSR	0.11	1.11	0.5	infinite	50	0.25	2	Rail
1/2" SSR	0.196	1.96	0.5	infinite	50	0.25	2	Rail
5/8" SSR	0.307	3.07	0.5	infinite	50	0.25	2	Rail
3/4" SSR	0.442	4.42	0.5	infinite	50	0.25	2	Rail

Loadings

Loading Name	*Type*	*Analysis*	*On-Pattern Factor*	*Off-Pattern Factor*
Self-Dead Loading	Self-Weight	Normal	1	1
Balance Loading	Balance	Normal	1	1
Hyperstatic Loading	Hyperstatic	Hyperstatic	1	1
Temporary Construction (At Stressing) Loading	Stressing Dead	Normal	1	1
Other Dead Loading	Dead	Normal	1	1
Live (Reducible) Loading	Live (Reducible)	Normal	1	0
Live (Unreducible) Loading	Live (Unreducible)	Normal	1	0
Live (Storage) Loading	Live (Storage)	Normal	1	0
Live (Parking) Loading	Live (Parking)	Normal	1	0
Live (Roof) Loading	Live (Roof)	Normal	1	0
Snow Loading	Snow	Normal	1	1
Snow unbalanced 1	Snow	Normal	1	1
Snow unbalanced 2	Snow	Normal	1	1
Wy	Ultimate Wind 1	Normal	1	1
Wyy	Ultimate Wind 1	Normal	1	1
Wx	Ultimate Wind 2	Normal	1	1
Wxx	Ultimate Wind 2	Normal	1	1
Ey	Ultimate Seismic 1	Normal	1	1
Ex	Ultimate Seismic 2	Normal	1	1

Load Combinations

All Dead LC

Active Design Criteria: <none>
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Other Dead Loading	1

Dead + Balance LC

Active Design Criteria: <none>
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1

Initial Service LC

Active Design Criteria: Initial Service Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1.13
Temporary Construction (At Stressing) Loading	1

Service LC: D + L

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Design, Soil Bearing Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	1
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1

Service LC: D

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Design, Soil Bearing Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1

Load Combinations (2)

Service LC: D + Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Design, Soil Bearing Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Roof) Loading	1

Service LC: D + S

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Design, Soil Bearing Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Snow Loading	1

Service LC: D + 0.75L + 0.75Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Design, Soil Bearing Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Live (Roof) Loading	0.75

Service LC: D + 0.75L + 0.75S

Active Design Criteria: User Minimum Design, Code Minimum Design, Service Design, Soil Bearing Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Snow Loading	0.75

Load Combinations (3)

Service Wind LC: D + 0.6W

Active Design Criteria: Soil Bearing Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 0.6

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1

Service Wind LC: D - 0.6W

Active Design Criteria: Soil Bearing Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -0.6

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1

Service Wind LC: D + 0.75L + 0.75Lr + 0.45W

Active Design Criteria: Soil Bearing Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 0.45

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Live (Roof) Loading	0.75

Service Wind LC: D + 0.75L + 0.75Lr - 0.45W

Active Design Criteria: Soil Bearing Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -0.45

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Live (Roof) Loading	0.75

Load Combinations (4)

Service Wind LC: D + 0.75L + 0.75S + 0.45W

Active Design Criteria: Soil Bearing Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 0.45

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Snow Loading	0.75

Service Wind LC: D + 0.75L + 0.75S - 0.45W

Active Design Criteria: Soil Bearing Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -0.45

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Snow Loading	0.75

Service Wind LC: 0.6D + 0.6W

Active Design Criteria: Soil Bearing Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 0.6

Loading	*Standard Factor*
Self-Dead Loading	0.6
Balance Loading	1
Other Dead Loading	0.6

Load Combinations (5)

Service Wind LC: 0.6D - 0.6W

Active Design Criteria: Soil Bearing Design

Analysis: Zero-Tension

Key Lateral Loading: Wind-Ultimate Standard Factor: -0.6

Loading	*Standard Factor*
Self-Dead Loading	0.6
Balance Loading	1
Other Dead Loading	0.6

Service Seismic LC: D + 0.7E

Active Design Criteria: Soil Bearing Design

Analysis: Zero-Tension

Key Lateral Loading: Seismic-Ultimate Standard Factor: 0.7

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1

Service Seismic LC: D - 0.7E

Active Design Criteria: Soil Bearing Design

Analysis: Zero-Tension

Key Lateral Loading: Seismic-Ultimate Standard Factor: -0.7

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1

Service Seismic LC: D + 0.75L + 0.75S + 0.525E

Active Design Criteria: Soil Bearing Design

Analysis: Zero-Tension

Key Lateral Loading: Seismic-Ultimate Standard Factor: 0.525

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Snow Loading	0.75

Load Combinations (6)

Service Seismic LC: D + 0.75L + 0.75S - 0.525E

Active Design Criteria: Soil Bearing Design

Analysis: Zero-Tension

Key Lateral Loading: Seismic-Ultimate Standard Factor: -0.525

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.75
Live (Unreducible) Loading	0.75
Live (Storage) Loading	0.75
Live (Parking) Loading	0.75
Snow Loading	0.75

Service Seismic LC: 0.6D + 0.7E

Active Design Criteria: Soil Bearing Design

Analysis: Zero-Tension

Key Lateral Loading: Seismic-Ultimate Standard Factor: 0.7

Loading	*Standard Factor*
Self-Dead Loading	0.6
Balance Loading	1
Other Dead Loading	0.6

Service Seismic LC: 0.6D - 0.7E

Active Design Criteria: Soil Bearing Design

Analysis: Zero-Tension

Key Lateral Loading: Seismic-Ultimate Standard Factor: -0.7

Loading	*Standard Factor*
Self-Dead Loading	0.6
Balance Loading	1
Other Dead Loading	0.6

Sustained Service LC

Active Design Criteria: Sustained Service Design

Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1
Balance Loading	1
Other Dead Loading	1
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	0.5
Live (Storage) Loading	1
Live (Parking) Loading	0.5
Live (Roof) Loading	0.5

Load Combinations (7)

Factored LC: 1.4D

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1.4
Hyperstatic Loading	1
Other Dead Loading	1.4

Factored LC: 0.9D

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9

Factored LC: 1.2D + 1.6L + 0.5Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	1.6
Live (Unreducible) Loading	1.6
Live (Storage) Loading	1.6
Live (Parking) Loading	1.6
Live (Roof) Loading	0.5

Factored LC: 1.2D + 1.6L + 0.5S

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	1.6
Live (Unreducible) Loading	1.6
Live (Storage) Loading	1.6
Live (Parking) Loading	1.6
Snow Loading	0.5

Load Combinations (8)

Factored LC: 0.9D + 1.6L + 0.5Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9
Live (Reducible) Loading	1.6
Live (Unreducible) Loading	1.6
Live (Storage) Loading	1.6
Live (Parking) Loading	1.6
Live (Roof) Loading	0.5

Factored LC: 0.9D + 1.6L + 0.5S

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9
Live (Reducible) Loading	1.6
Live (Unreducible) Loading	1.6
Live (Storage) Loading	1.6
Live (Parking) Loading	1.6
Snow Loading	0.5

Factored LC: 1.2D + f1L + 1.6Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Live (Roof) Loading	1.6

Load Combinations (9)

Factored LC: 1.2D + f1L + 1.6S

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Snow Loading	1.6

Factored LC: 0.9D + f1L + 1.6Lr

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Live (Roof) Loading	1.6

Factored LC: 0.9D + f1L + 1.6S

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Snow Loading	1.6

Load Combinations (10)

Factored Wind LC: 1.2D + f1L + 0.5Lr + 1.0W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 1

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Live (Roof) Loading	0.5

Factored Wind LC: 1.2D + f1L + 0.5Lr - 1.0W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -1

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Live (Roof) Loading	0.5

Factored Wind LC: 1.2D + f1L + 0.5S + 1.0W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 1

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Snow Loading	0.5

Load Combinations (11)

Factored Wind LC: 1.2D + f1L + 0.5S - 1.0W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -1

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Snow Loading	0.5

Factored Wind LC: 1.2D + 1.6Lr + 0.5W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 0.5

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Roof) Loading	1.6

Factored Wind LC: 1.2D + 1.6Lr - 0.5W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -0.5

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Roof) Loading	1.6

Factored Wind LC: 1.2D + 1.6S + 0.5W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 0.5

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Snow Loading	1.6

Load Combinations (12)

Factored Wind LC: 1.2D + 1.6S - 0.5W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -0.5

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Snow Loading	1.6

Factored Wind LC: 0.9D + W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: 1

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9

Factored Wind LC: 0.9D - W

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Wind-Ultimate Standard Factor: -1

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9

Factored Seismic LC: 1.2D + f1L + 0.2S + E

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Seismic-Ultimate Standard Factor: 1

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Snow Loading	0.2

Load Combinations (13)

Factored Seismic LC: 1.2D + f1L + 0.2S - E

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Seismic-Ultimate Standard Factor: -1

Loading	*Standard Factor*
Self-Dead Loading	1.2
Hyperstatic Loading	1
Other Dead Loading	1.2
Live (Reducible) Loading	0.5
Live (Unreducible) Loading	1
Live (Storage) Loading	1
Live (Parking) Loading	1
Snow Loading	0.2

Factored Seismic LC: 0.9D + E

Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Seismic-Ultimate Standard Factor: 1

Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9

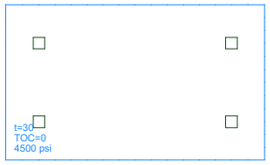
Factored Seismic LC: 0.9D - E

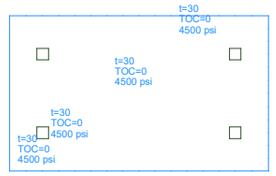
Active Design Criteria: User Minimum Design, Code Minimum Design, Strength Design, Ductility Design
Analysis: Zero-Tension
Key Lateral Loading: Seismic-Ultimate Standard Factor: -1

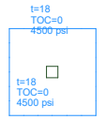
Loading	*Standard Factor*
Self-Dead Loading	0.9
Hyperstatic Loading	1
Other Dead Loading	0.9

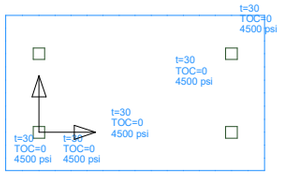
Element: Slab Summary Plan

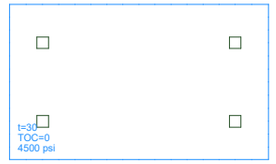
Element: User Lines; User Notes; User Dimensions; Wall Elements Below; Wall Elements Above; Column Elements Below; Column Elements Above; Point Springs; Point Spring Icons; Line Springs; Line Spring Icons; Slab Elements; Slab Element Outline Only; Slab Element Thicknesses; Slab Element Elevations; Slab Element Concrete Models;
Scale = 1:250





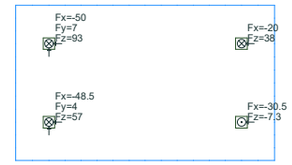


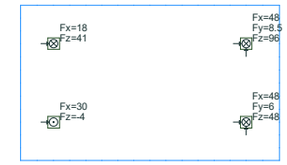


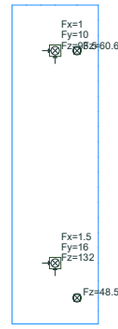


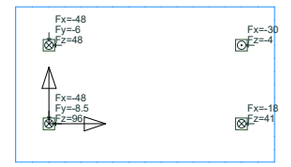
Other Dead Loading: All Loads Plan

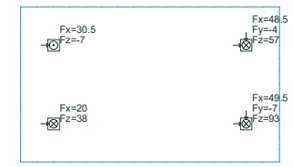
Other Dead Loading: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250









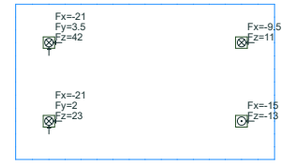


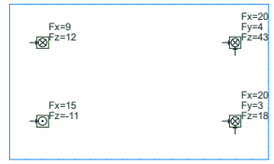
Live (Roof) Loading: All Loads Plan

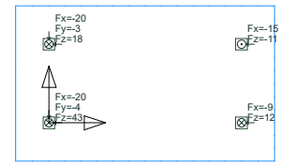
Live (Roof) Loading; User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values; Element; Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250

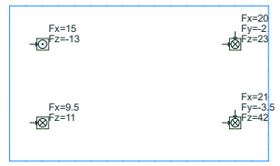
Snow Loading: All Loads Plan

Snow Loading; User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250



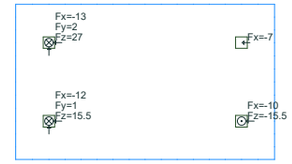


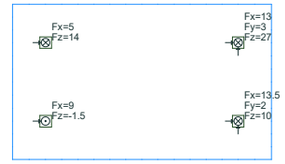


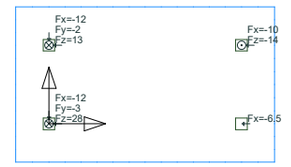


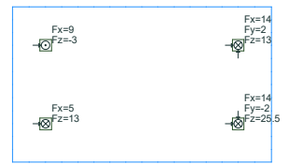
Snow unbalanced 1: All Loads Plan

Snow unbalanced 1: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250



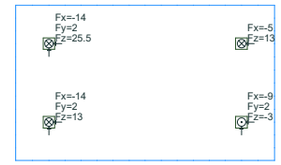


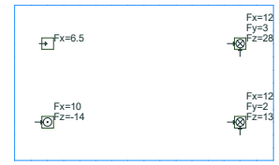


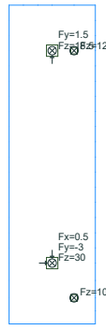


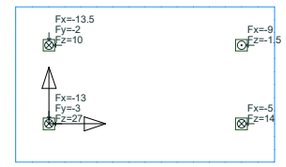
Snow unbalanced 2: All Loads Plan

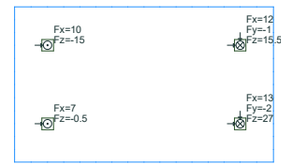
Snow unbalanced 2: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250









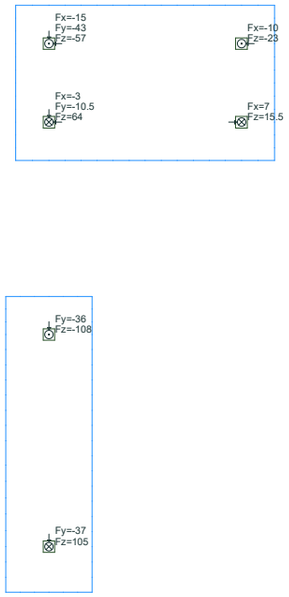


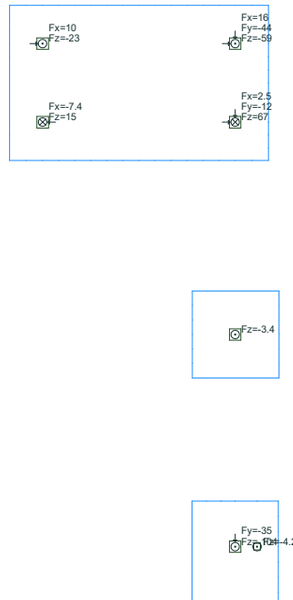
Wxx: All Loads Plan

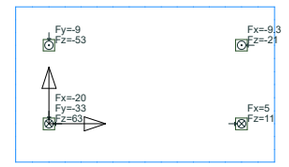
Wxx: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250

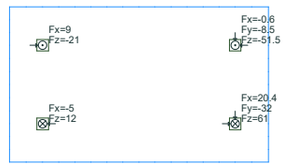
Wyy: All Loads Plan

Wyy: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250



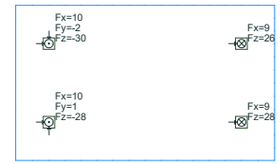


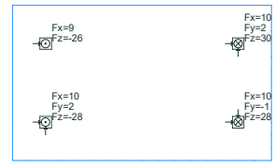


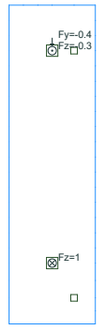


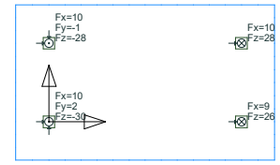
Ex: All Loads Plan

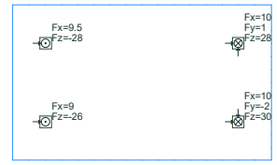
Ex: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250





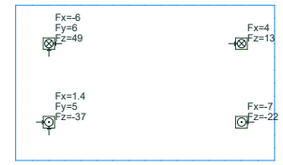


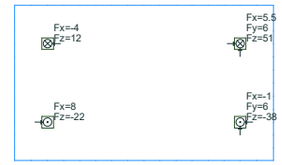


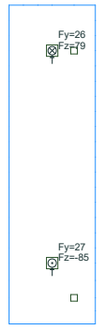


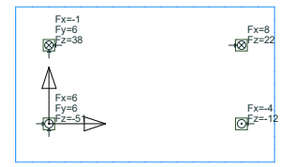
Ey: All Loads Plan

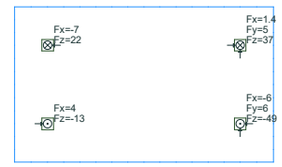
Ey: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250





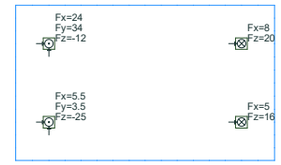


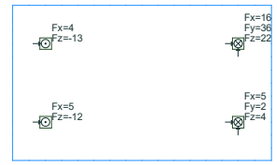


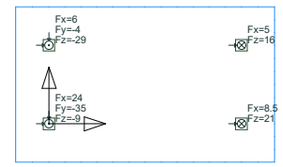


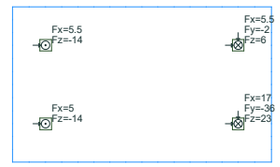
Wx: All Loads Plan

Wx: User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250









Wy: All Loads Plan

Wy; User Lines; User Notes; User Dimensions; Point Loads; Point Load Icons; Point Load Values; Line Loads; Line Load Icons; Line Load Values; Area Loads; Area Load Icons; Area Load Values;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
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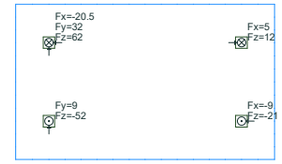

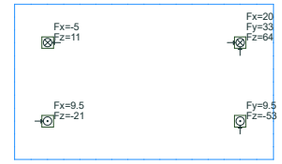

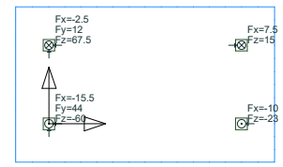

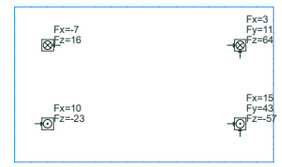

Factored Wind LC: 1.2D + f1L + 0.5Lr + 1.0W: Max Mx Plan

Factored Wind LC: 1.2D + f1L + 0.5Lr + 1.0W: User Lines; User Notes; User Dimensions;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250
Factored Wind LC: 1.2D + f1L + 0.5Lr + 1.0W - Bending Moment Plot (Maximum Values) (X-Axis Direction)
One Contour = 2 Kips
Min Value = -56.11 Kips @ (246.3,45.12) Max Value = 29 Kips @ (0,-0.7421)

Code Minimum Design: Status Plan

Code Minimum Design; User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
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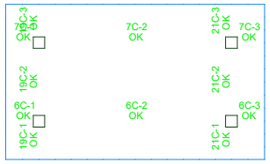

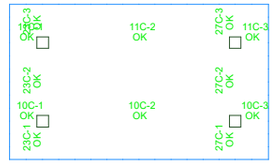

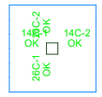

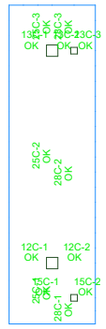

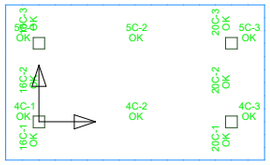

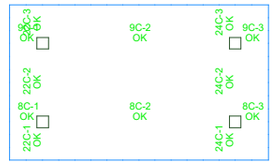

Service Design: Status Plan

Service Design: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
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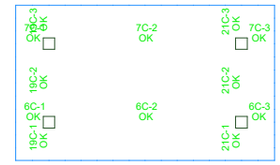

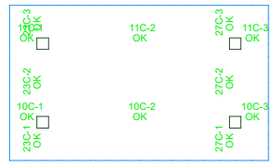

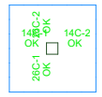

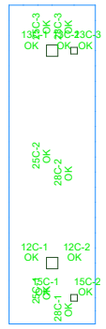

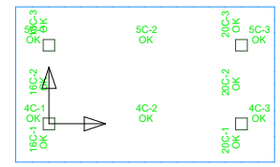

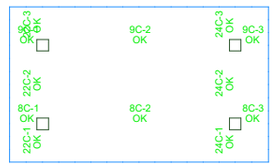

Sustained Service Design: Status Plan

Sustained Service Design; User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
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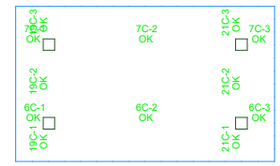
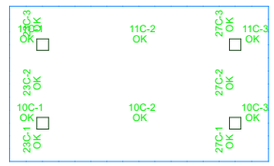
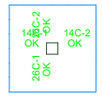
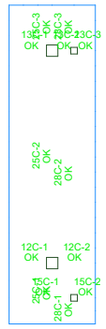
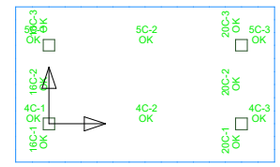
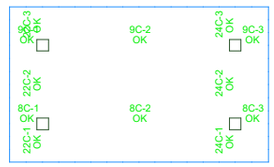






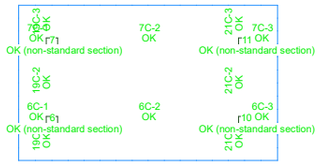


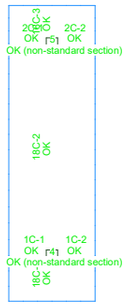


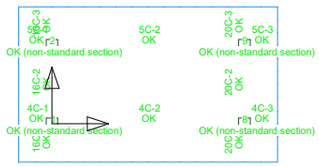


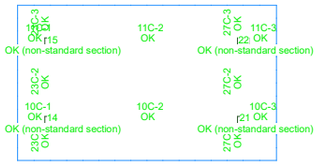
Strength Design: Status Plan

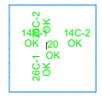
Strength Design; User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status; Element; Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250

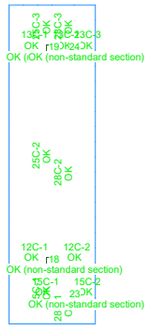


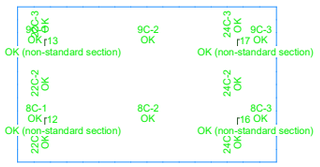










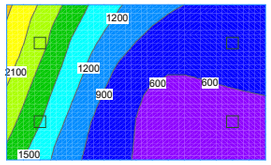


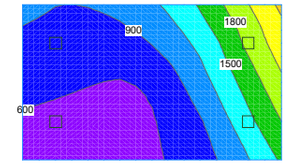
Soil Bearing Design: Max Soil Bearing Pressure Plan

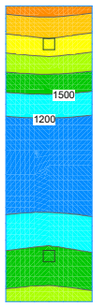
Soil Bearing Design: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Latitude DS Designs; Longitude DS Designs; PC Designs;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250
Soil Bearing Design - Area Spring Vertical Reactions Plot (Maximum Values)

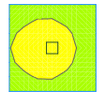
600 900 1200 1500 1800 2100 2400 2700 3000

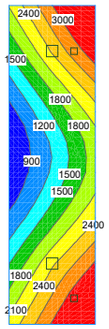
Min Value = 356.9 psf @ (22.3,103) Max Value = 3830 psf @ (246.3,26.66)

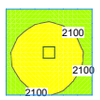


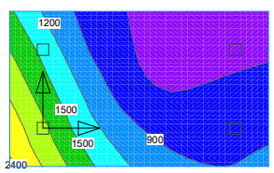






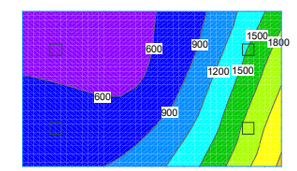






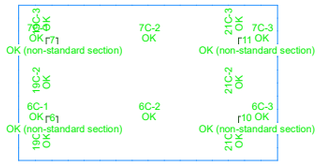
To check sliding: seismic mass x coefficient of friction = 0.36W = 0.36x1220 = 439 K
Seismic base shear = seismic mass x Cs = 0.123W < 0.36W
Wind base shear = 375 K < 439 K

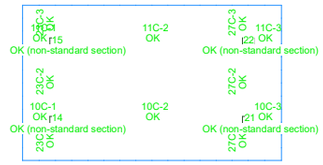
There is no tension in the soil, and the foundation is stable against overturning.

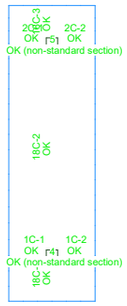


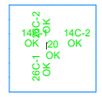
Design Status: Status Plan

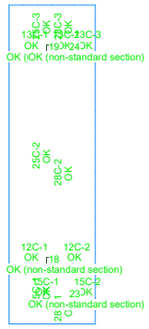
Design Status: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Numbers; Span Design Status; Latitude DS Designs; Longitude DS Designs; DS Design Numbers; DS Design Status; PC Designs; PC Design Numbers; PC Design Status; Latitude Span Segment Deflection Checks; Longitude Span Segment Deflection Checks; Span Segment Deflection Check Status; Latitude Deflection Checks; Longitude Deflection Checks; Deflection Check Status; Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Scale = 1:250

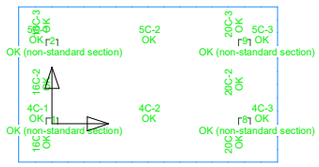


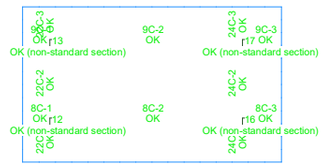






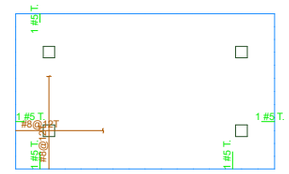


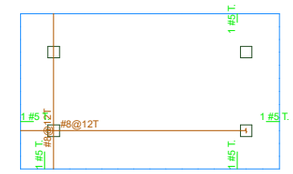


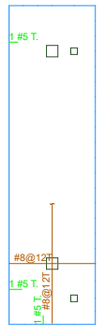


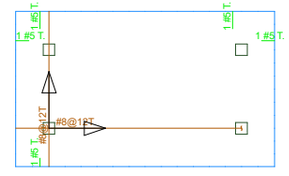
Design Status: Top Reinforcement Plan

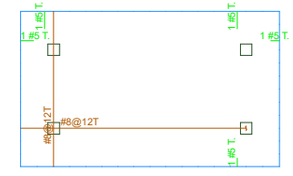
Design Status: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Top Bars; Span Design Bar Descriptions; Latitude DS Designs; Longitude DS Designs; DS Design Top Bars;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Reinforcement: Top Face Concentrated Reinf.; Both Faces Concentrated Reinf.; Auto Face Concentrated Reinf.; Concentrated Reinf. Descriptions; Top Face Distributed Reinf.; Both Faces Distributed Reinf.; Auto Face Distributed Reinf.; Distributed Reinf. Descriptions; Latitude User Concentrated Reinf.; Longitude User Concentrated Reinf.; Latitude User Distributed Reinf.; Longitude User Distributed Reinf.;
Scale = 1:250





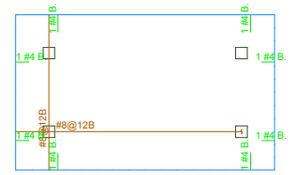


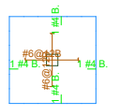


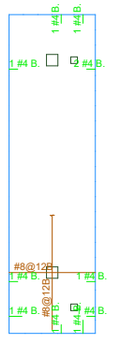


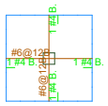
Design Status: Bottom Reinforcement Plan

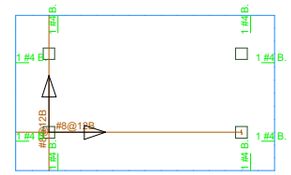
Design Status: User Lines; User Notes; User Dimensions; Latitude Span Designs; Longitude Span Designs; Span Design Bottom Bars; Span Design Bar Descriptions; Latitude DS Designs; Longitude DS Designs; DS Design Bottom Bars;
Element: Wall Elements Below; Wall Elements Above; Wall Element Outline Only; Column Elements Below; Column Elements Above; Slab Elements; Slab Element Outline Only;
Reinforcement: Bottom Face Concentrated Reinf.; Both Faces Concentrated Reinf.; Auto Face Concentrated Reinf.; Concentrated Reinf. Descriptions; Bottom Face Distributed Reinf.; Both Faces Distributed Reinf.; Auto Face Distributed Reinf.; Distributed Reinf. Descriptions; Latitude User Concentrated Reinf.; Longitude User Concentrated Reinf.; Latitude User Distributed Reinf.; Longitude User Distributed Reinf.;
Scale = 1:250





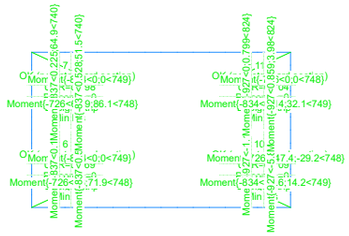


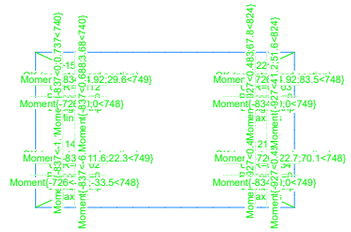


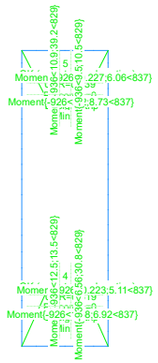


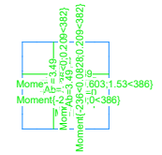
Design Status: Punching Shear Status Plan

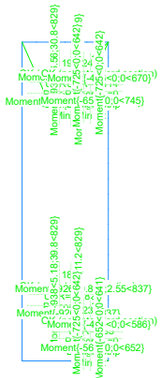
Design Status: PC Designs; PC Design Numbers; PC Design Status; PC Design Controlling; PC Design Stress Ratios; PC Design Prestress; PC Design Sections; SSR; PC Design Flexural Sections; PC Design Flexural Section Design; PC Design Flexural Section Analysis; User Notes; User Lines; User Dimensions; Element: Wall Elements Above; Wall Elements Below; Wall Element Outline Only; Column Elements Above; Column Elements Below; Slab Elements; Slab Element Outline Only;
Scale = 1:250

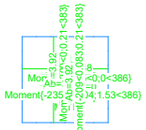


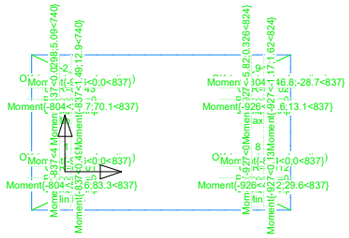


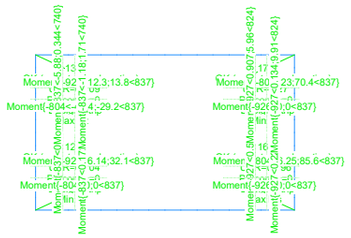












Tekla Tedds	Project				Job Ref.	
	Creech DRP PH 2					
	Section				Sheet no./rev.	
	Column B-1.1				1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	BT	10/27/2025				

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status PASS

Overall design utilisation 0.937

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	227.3			Pass
Overturning stability, x	kip_ft	3.75	-1136.60	303.09	Pass
Sliding stability, x	kips	2.5	131.2	52.497	Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.296	3	0.765	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	252.1	269.0	0.937	Pass
Moment, positive, y-direction	kip_ft	250.0	283.9	0.881	Pass
Shear, one-way, x-direction	kips	83.2	92.7	0.898	Pass
Shear, one-way, y-direction	kips	82.6	92.7	0.891	Pass
Shear, two-way, Col 1	psi	156.962	201.246	0.788	Pass
Min.area of reinf, bot., x-direction	in^2	3.888	4.400		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	12.5		Pass
Min.area of reinf, bot., y-direction	in^2	3.888	4.400		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	12.5		Pass

Pad footing details

Length of footing L_x = **10** ft

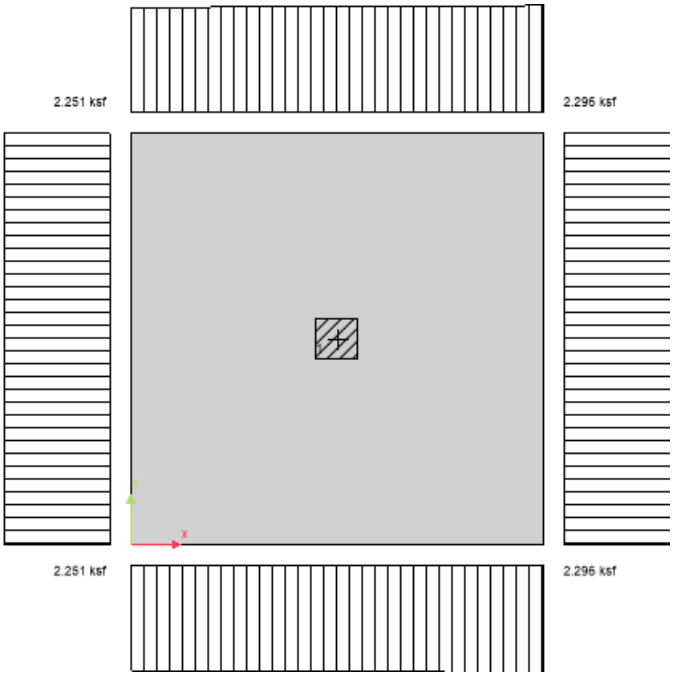
Width of footing L_y = **10** ft

Footing area $A = L_x \times L_y$ = **100** ft^2

Depth of footing h = **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} = **12.00** in
Width of column	l_{y1} = **12.00** 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(δ_{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} = **124.0** kips
Live roof load in z	F_{Lrz1} = **60.0** kips
Snow load in z	F_{Sz1} = **63.0** kips
Wind load in z	F_{Wz1} = **-5.0** kips
Seismic load in z	F_{Ez1} = **-6.0** kips
Dead load in x	F_{Dx1} = **1.5** kips
Live roof load in x	F_{Lrx1} = **1.0** kips
Snow load in x	F_{Sx1} = **1.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.552)
1.0D + 1.0L (0.552)
1.0D + 1.0Lr (0.755)
1.0D + 1.0S (0.765)
1.0D + 1.0R (0.552)
1.0D + 0.75L + 0.75Lr (0.704)
1.0D + 0.75L + 0.75S (0.712)
1.0D + 0.75L + 0.75R (0.552)
1.0D + 0.6W (0.542)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.616)
1.0D + 0.75L + 0.75Lr + 0.45W (0.697)
1.0D + 0.75L + 0.75S + 0.45W (0.704)
1.0D + 0.75L + 0.75R + 0.45W (0.545)
$(1.0 + 0.10 \times S_{DS})D + 0.75L + 0.75S + 0.525E$ (0.757)
0.6D + 0.6W (0.321)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.240)

Combination 4 results: 1.0D + 1.0S

Forces on footing

Force in x-axis: $F_{dx} = \gamma_D \times F_{Dx1} + \gamma_S \times F_{Sx1}$ = **2.5** kips

Force in z-axis: $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_S \times F_{Sz1}$ = **227.3** kips

Moments on footing

Moment in x-axis, about x is 0: $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1 + F_{Dx1} \times (h)) + \gamma_S \times (F_{Sz1} \times x_1 + F_{Sx1} \times (h))$ = **1140.3** kip_ft

Moment in y-axis, about y is 0: $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **1136.6** kip_ft

Uplift verification

Vertical force: F_{dz} = **227.32** kips

PASS - Footing is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment: $M_{OTxL} = \gamma_D \times (F_{Dx1} \times (h)) + \gamma_S \times (F_{Sx1} \times (h))$ = **3.75** kip_ft

Resisting moment: $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times (x_1 - L_x)) + \gamma_S \times (F_{Sz1} \times (x_1 - L_x))$ = **-1136.6** kip_ft

Factor of safety: $abs(M_{RxL} / M_{OTxL})$ = **303.093**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction: $F_{RFriction} = max(F_{dz}, 0\ kN) \times tan(\delta_{bb})$ = **131.243** kips

Stability against sliding in x direction

Total sliding resistance: $F_{Rx} = F_{RFriction}$ = **131.243** kips

Factor of safety: $abs(F_{RTx} / F_{dx})$ = **52.5**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0.198** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.251** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.251** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.296** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **2.296** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **2.251** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **2.296** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow_Gross} =$ **3** ksf

$q_{max} / q_{allow} =$ **0.765**

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)	$\varepsilon_{ty} =$ **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	$\lambda =$ **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.657)
1.2D + 1.6L + 0.5Lr (0.678)
1.2D + 1.6L + 0.5S (0.683)
1.2D + 1.6L + 0.5R (0.569)
1.2D + 1.0L + 1.6Lr (0.929)
1.2D + 1.0L + 1.6S (0.907)
1.2D + 1.0L + 1.6R (0.788)
1.2D + 1.6Lr + 0.5W (0.919)
1.2D + 1.6S + 0.5W (0.937)
1.2D + 1.6R + 0.5W (0.788)
1.2D + 1.0L + 0.5Lr + 1.0W (0.788)
1.2D + 1.0L + 0.5S + 1.0W (0.788)

1.2D + 1.0L + 0.5R + 1.0W (0.788)

(1.2 + 0.2 × S_{DS})D + 1.0L + 0.2S + 1.0E (0.788)

0.9D + 1.0W (0.788)

(0.9 - 0.2 × S_{DS})D + 1.0E (0.788)

Combination 9 results: 1.2D + 1.6S + 0.5W

Forces on footing

Ultimate force in x-axis

$F_{ux} = \gamma_D \times F_{Dx1} + \gamma_S \times F_{Sx1}$ = **3.4** kips

Ultimate force in z-axis

$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_S \times F_{Sz1} + \gamma_W \times F_{Wz1}$ = **295.5** kips

Moments on footing

Ultimate moment in x-axis, about x is 0

$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1 + F_{Dx1} \times (h)) + \gamma_S \times (F_{Sz1} \times x_1 + F_{Sx1} \times (h)) + \gamma_W \times (F_{Wz1} \times x_1)$ = **1482.5** kip_ft

Ultimate moment in y-axis, about y is 0

$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **1477.4** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0.207** in

Eccentricity of base reaction in y-axis

$e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.924** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.924** ksf

$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.985** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.985** ksf

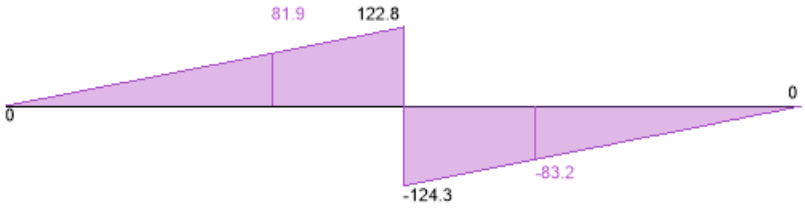
Minimum ultimate base pressure

$q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.924** ksf

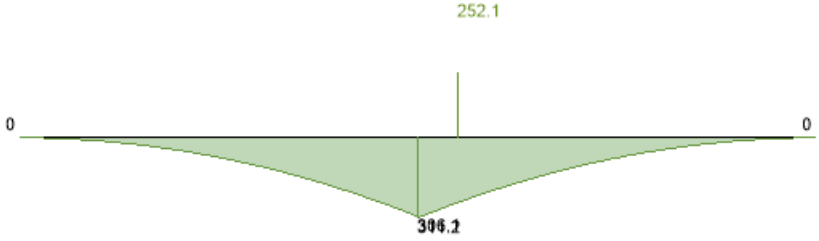
Maximum ultimate base pressure

$q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.985** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment

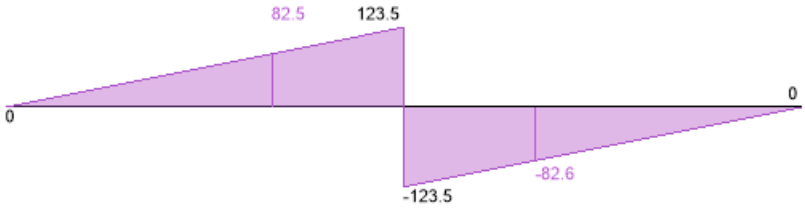
$M_{u.x.max}$ = **252.139** kip_ft

Tension reinforcement provided	10 No.6 bottom bars (12.5 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **4.4** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min}$ = 0.0018 × L_y × h = **3.888** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 × h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	d = h - c_{nom_b} - $\phi_{x.bot}$ / 2 = **14.625** in
Depth of compression block	a = $A_{sx.bot.prov}$ × f_y / (0.85 × f'$_c$ × L_y) = **0.575** in
Neutral axis factor	β_1 = **0.83**
Depth to neutral axis	c = a / β_1 = **0.697** in
Strain in tensile reinforcement	ε_t = 0.003 × d / c - 0.003 = **0.05993**
Minimum tensile strain(8.3.3.1)	ε_{min} = ε_{ty} + 0.003 = **0.00500**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	M_n = $A_{sx.bot.prov}$ × f_y × (d - a / 2) = **315.423** kip_ft
Flexural strength reduction factor	ϕ_f = min(max(0.65 + 0.25 × (ε_t - ε_{ty}) / (0.003), 0.65), 0.9) = **0.900**
Design moment capacity	ϕM_n = ϕ_f × M_n = **283.881** kip_ft
	$M_{u.x.max}$ / ϕM_n = **0.888**
	PASS - Design moment capacity exceeds ultimate moment load

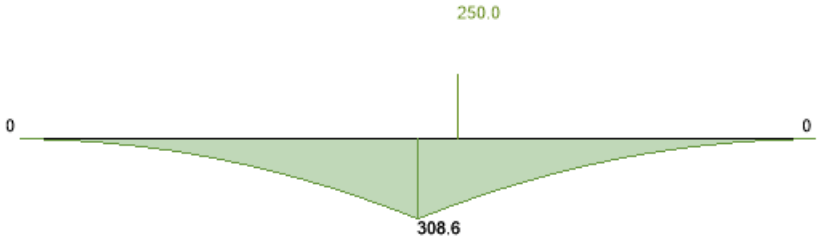
One-way shear design, x direction

Ultimate shear force	$V_{u.x}$ = **83.233** kips
Depth to reinforcement	d_v = h - c_{nom_b} - $\phi_{x.bot}$ / 2 = **14.625** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	ρ_w = $A_{sx.bot.prov}$ / (L_y × d_v) = **0.00251**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	V_n = min(8 × λ_s × λ × (ρ_w)$^{1/3}$ × √(f'$_c$ × 1 psi) × L_y × d_v, 5 × λ × √(f'$_c$ × 1 psi) × L_y × d_v) = **127.948** kips
Design shear capacity	ϕV_n = ϕ_v × V_n = **95.961** kips
	$V_{u.x}$ / ϕV_n = **0.867**
	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	$M_{u.y.max}$ = **249.97** kip_ft
Tension reinforcement provided	10 No.6 bottom bars (12.5 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **4.4** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **3.888** 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	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.575** in
Neutral axis factor	β_1 = **0.83**
Depth to neutral axis	$c = a / \beta_1$ = **0.697** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.05671**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **298.923** 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	$\phi M_n = \phi_f \times M_n$ = **269.031** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.929**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

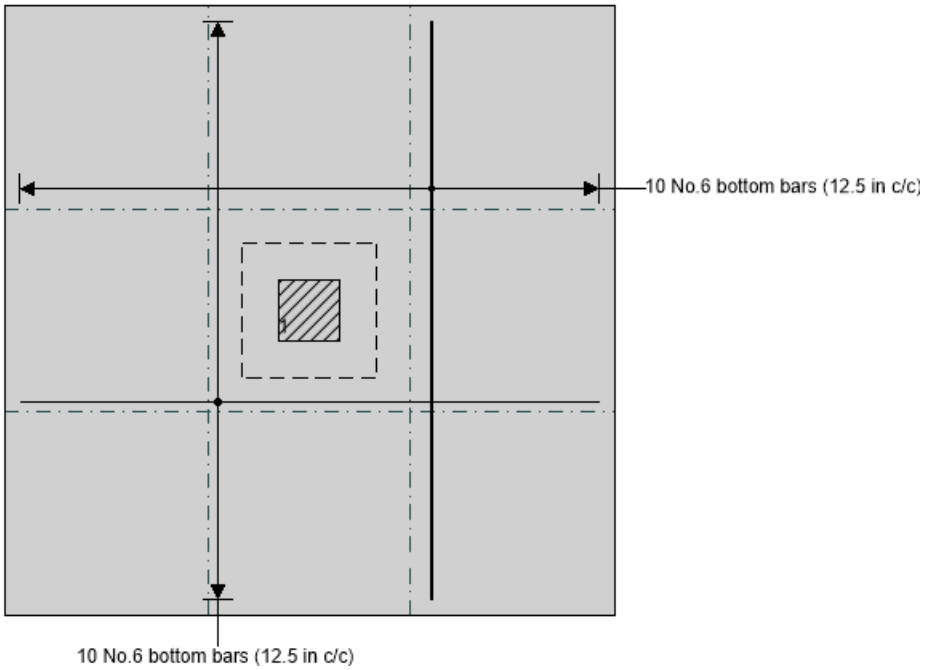
Ultimate shear force	$V_{u.y}$ = **82.552** kips
Depth to reinforcement	$d_v = h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **13.875** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = A_{sy.bot.prov} / (L_x \times d_v)$ = **0.00264**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v)$ = **123.535** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **92.651** kips
	$V_{u.y} / \phi V_n$ = **0.891**
	PASS - Design shear capacity exceeds ultimate shear load

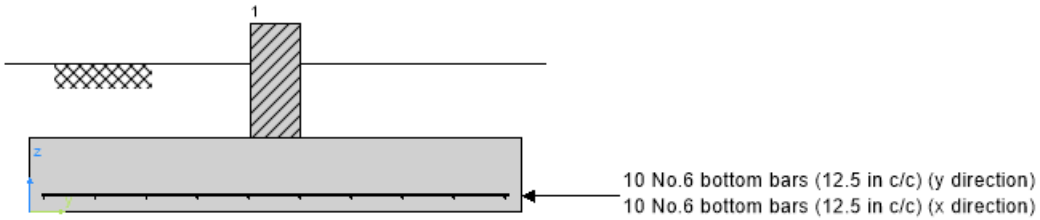
Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **14.25** in
Shear perimeter length (22.6.4)	l_{xp} = **26.250** in

Shear perimeter width (22.6.4)	l_{yp} = **26.250** in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **105.000** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **689.063** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1}$ = **545.063** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **2.955** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_S \times F_{Sz1} + \gamma_W \times F_{Wz1} + \gamma_D \times A_p \times F_{swt}$ + $\gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **234.854** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0$ psi) = **156.962** 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)	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **402.492** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **498.324** 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	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
	$v_{ug} / \phi v_n$ = **0.780**

PASS - Design shear stress capacity exceeds ultimate shear stress load




1
z
10 No.6 bottom bars (12.5 in c/c) (y direction)
10 No.6 bottom bars (12.5 in c/c) (x direction)

S.O. No. _____

Subject: Creech DRP Phase 2, Hangar SOG

_____ Sheet No. _____ of _____

_____ Drawing No. _____

Computed by _____ Checked by _____ Date 10/10/2025

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HANGAR BAY SOG CRITERIA

- From RFP: 01 10 10.06 Paragraph 1.4.8.6:
"Design the AOF hangar bay slabs on ground for the minimum wheel loads in accordance with latest edition of UFC 4-211-01. Determine the concrete slab on ground thickness for the AOF hangar bay slabs in accordance with the latest edition of UFC 3-260-02, utilizing a non-reinforced concrete section."

- From RFP Bidder Inquiry Response #10824885:
"The minimum design wheel load for the hangar slab is 30,000 pounds from a tire with a 240 psi pressure. The trench cover plate must be designed to withstand a minimum proof-load of 50,000 pounds from a tire with a 250 psi pressure. By: James Elsberry; SPL PPMD - Military Support Branch; (702-652-8443) On: May 28 2024"

-From UFC 4-211-01:

3-4.2.3.1 Aircraft Maintenance Bay - Slab on Ground Design

Determine the concrete slab on ground thickness for the Aircraft Maintenance Bay in accordance with UFC 3-260-02, utilizing a non-reinforced concrete section with the Service-specific minimum loads shown in this UFC. It is acceptable to design and install reinforcing for temperature and/or crack control but the reinforcing cannot be used to decrease the slab on ground thickness from the calculated non-reinforced slab thickness. If point loads other than wheel loads require a thicker slab section, then reinforcing can be utilized in the area of the point load to keep from increasing the slab thickness for the point load. A typical cross-section is shown in

- Minimum concrete slab on ground thickness will be 8 inches. Increase slab thickness in 1/2" increments.
- Minimum concrete flexural strength will be 550 pounds per square inch (psi) at 90 days.
- Maximum concrete flexural strength will be 650 psi at 90 days.

  \1\
- In hangar bays that allow for multiple aircraft parking positions, provide a uniform slab on ground design throughout the hangar bay. Do not reduce the slab on ground thickness in areas not subject to the design aircraft wheel loads.

S.O. No. _____

Subject: Creech DRP Phase 2, Hangar SOG

_____ Sheet No. _____ of _____

_____ Drawing No. _____

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-From UFC 4-211-01:

3-4.2.3.2 Aircraft Maintenance Bay - Concrete Slab on Ground Joint Spacing

Space the Aircraft Maintenance Bay slab on ground joints in accordance with Table 3-1: Aircraft Maintenance Bay Slab on Ground Joint Spacing. Locate the joints considering the column line spacing while not exceeding the maximums listed in Table 3-1: Aircraft Maintenance Bay Slab on Ground Joint Spacing.

Table 3-1: Aircraft Maintenance Bay Slab on Ground Joint Spacing

SLAB TYPE	CONCRETE THICKNESS* (T) MILLIMETERS (INCHES)	SPACING METERS (FEET)
NONREINFORCED SLAB	205 (8) ≤ T ≤ 230 (9) 230 (9) < T ≤ 305 (12) T > 305 (12)	3.8 TO 4.6 (12.5 TO 15) 4.6 TO 6.1 (15 TO 20) 6.1 (20) MAX
SLAB WITH REINFORCEMENT (MINIMUM 0.05% STEEL IN BOTH DIRECTIONS)	ALL THICKNESSES	7.6 (25) MAX

Table 3-2: Concrete Slab on Ground Dowel Size and Spacing

CONCRETE SLAB THICKNESS IN INCHES	\2\ MINIMUM DOWEL LENGTH IN INCHES /2/	\2\ MAXIMUM DOWEL SPACING IN INCHES /2/	DOWEL DIAMETER AND TYPE
8 TO AND INCLUDING 11.5	16	12	1 INCH BAR
12 TO AND INCLUDING 15.5	20	15	1 TO 1-1/4 INCH BAR
16 TO AND INCLUDING 20.5	20	18	1-1/4 TO 1-1/2 INCH BAR
21 TO AND INCLUDING 25.5	24	18	2 INCH BAR
OVER 26	30	18	3 INCH BAR

S.O. No. _____

Subject: Creech DRP Phase 2, Hangar SOG

_____ Sheet No. _____ of _____

_____ Drawing No. _____

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-From UFC 4-211-01:

5-4 STRUCTURAL

5-4.1 Aircraft Maintenance Bay - Slab on Ground Loading

Air Force traffic loadings are designated in Table 3-1 of UFC 3-260-02 Pavement Design for Airfields. Air Force hangar pavement thickness will be designed in accordance with UFC 3-260-02, Chapter 3, paragraph 2c, titled Type C Traffic Areas. Obtain approval for deviations of aircraft traffic mixes or deviation of airfield design type (heavy, modified-heavy, medium, light) from the AFCEC pavements engineer.

-From Geotech report: k = 120 pci

-From UFC 3-260-02:

Figure 12-6 - Plain concrete design curves for Air Force light-load pavements
Use 12.5" unreinforced pavement thickness (see next page)

Confirmed passing with ACI 360R check on following pages

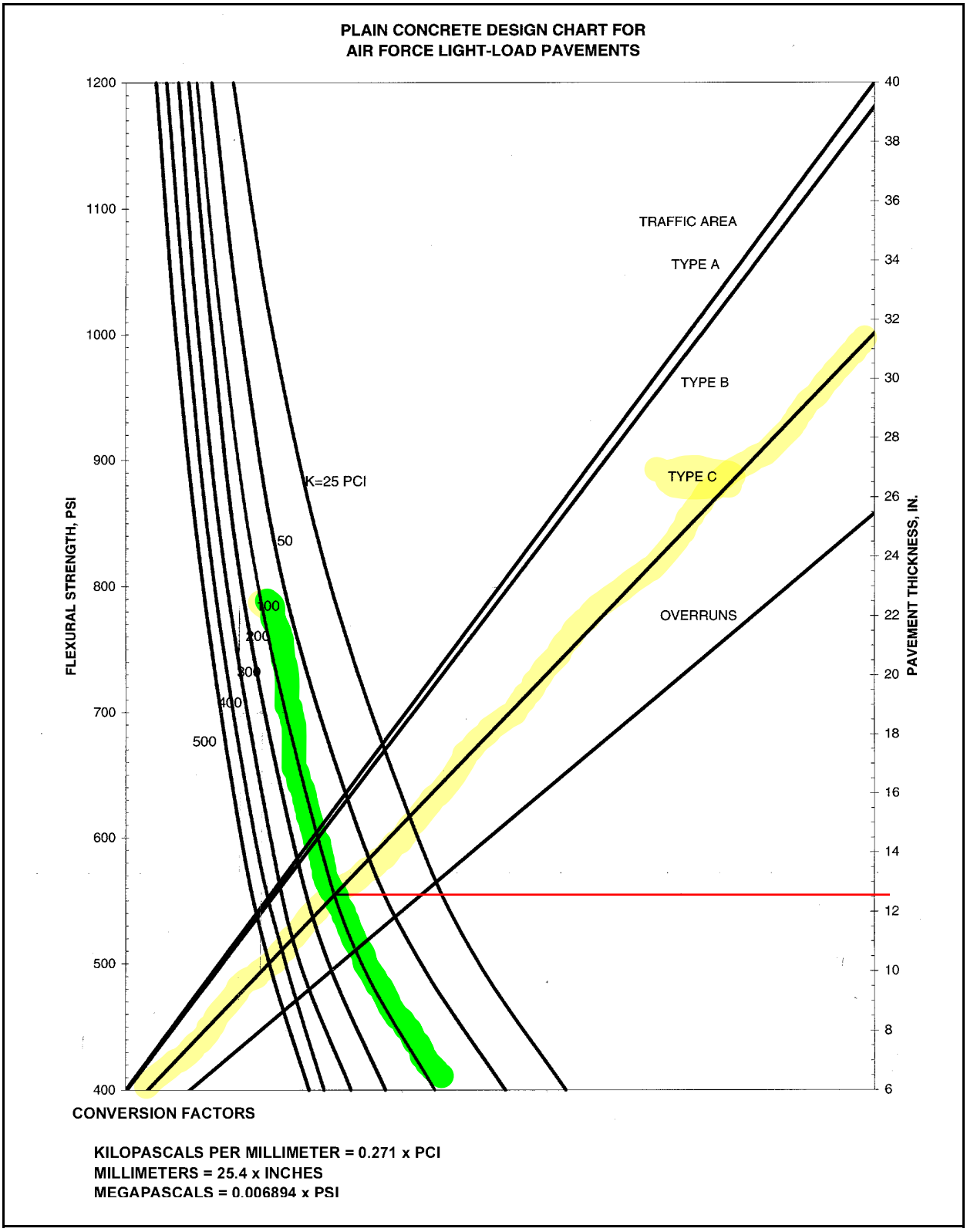


Figure 12-6. Plain concrete design curves for Air Force light-load pavements

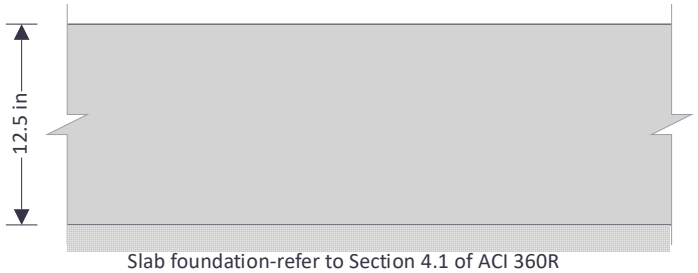
SLAB ON GROUND (ACI 360R)

In accordance with Guide to Design of Slab-on-Ground per ACI 360R-10

Tedds calculation version 1.0.03

Design summary

Description	Unit	Provided	Required	Utilization	Result
Slab thickness	in	12.5	9.02	0.722	PASS
Joint spacing	ft	20	22.25	0.899	PASS



Slab foundation-refer to Section 4.1 of ACI 360R

Design method

Design method publisher — **Wire Reinforcement Institute**

Materials and site properties

Slab thickness — h = **12.5** in

Specified compressive strength of concrete — f'_c = **5000** psi

Subgrade modulus — k = **120** lb/in^3

Concrete modulus of elasticity — E = **3605** ksi

Wheel specifications

Axle type — **Single-axle**

Number of wheels at end of each axle — **Single wheel**

Wheel center to center spacing — S = **40** in

Wheel contact area — A_{wheel} = **125** in^2

Diameter of equivalent circle — $d_{eqv_circle} = \sqrt{((A_{wheel} \times 4) / \pi)}$ = **12.6** in

Loading details

Load location — **Interior**

Axle load — P_a = **30** kips

Wheel load — $P_{wheel} = P_a / 2$ = **15** kips

Safety factor — FS = **2**

Slab thickness design

By iteration assumend trial thickness — h_{trial} = **9** in

The following output is based on the use of this trial thickness in the design charts in Appendix A of ACI 360R

Relative stiffness parameter (Fig. A2.1) — Dbyk = **1.94×10^6** in^4

Modulus of rupture of concrete — $f_r = 9 \times \sqrt{(f'_c \times 1\ psi)}$ = **636.4** psi

Allowable tensile stress — $f_{t_allow} = f_r / FS$ = **318.2** psi

Basic bending moments (Fig. A2.2) — M_{basic} = **235.84** lb_in/in/kips

Additional moments (Fig. A2.2) — $M_{additional}$ = **49.5** lb_in/in/kips

Total unit moments — $M_{unit} = M_{basic} + M_{additional}$ = **285.34** lb_in/in/kips

Design bending moments	$M_{design} = M_{unit} \times P_{wheel} =$ **4280** lb_ft/ft
Required slab thickness (Fig. A2.3)	$h_{min} =$ **9.02** in
	$h_{min} / h =$ **0.722**
	PASS - Slab thickness is adequate to avoid live load-induced cracks

Crack control options

Reinforcement type	**Unreinforced**
Concrete type	**Typical concrete**
Actual joint spacing	$S_{joint} =$ **20.0** ft
Recommended joint spacing (Fig.6.6)	$S_{joint_max} =$ **22.3** ft
	$S_{joint} / S_{joint_max} =$ **0.899**
	PASS - Joint spacing is adequate to avoid live load-induced cracks

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Creech DRP Phase 2

Aircraft Maintenance Facility (AMF)

Admin Structural Calculations

to be updated in 100% Submission

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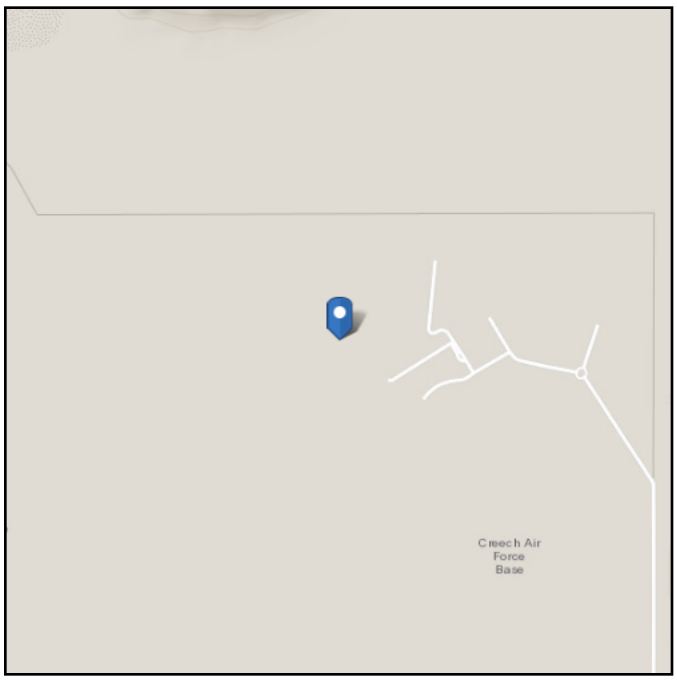
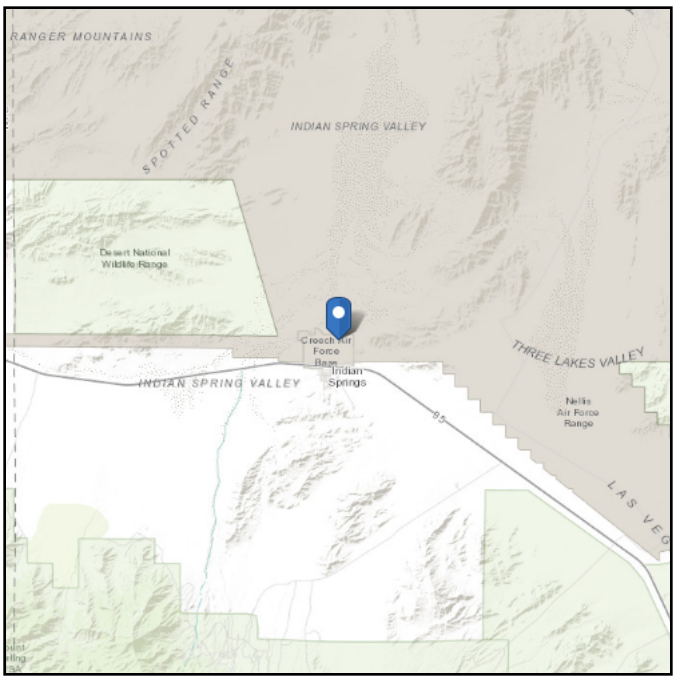
Criteria and Model Information

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

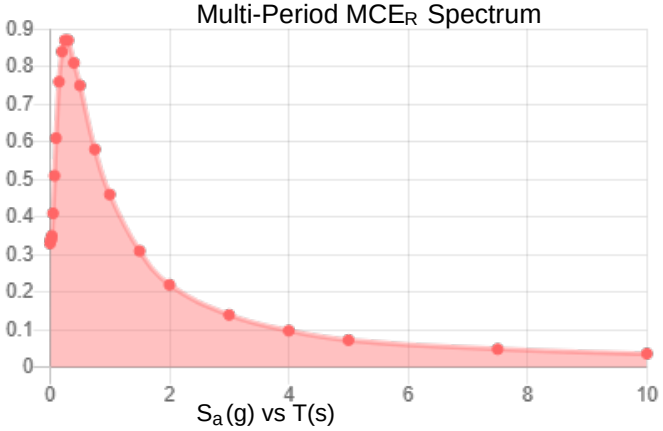
Site Soil Class: D - Stiff Soil

Results:

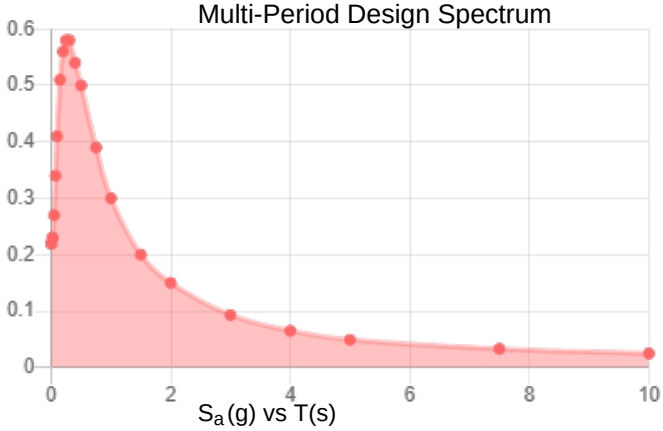
PGA_M :	0.31	T_L :	6
S_{MS} :	0.79	S_S :	0.58
S_{M1} :	0.46	S_1 :	0.17
S_{DS} :	0.52	V_{S30} :	260
S_{D1} :	0.3		

Seismic Design Category: D

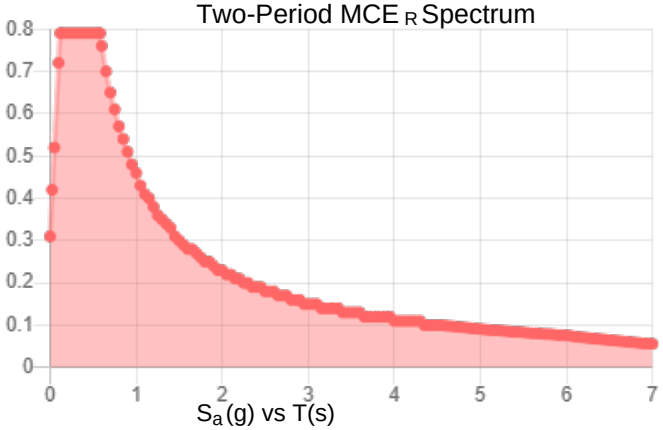
Multi-Period MCE_R Spectrum



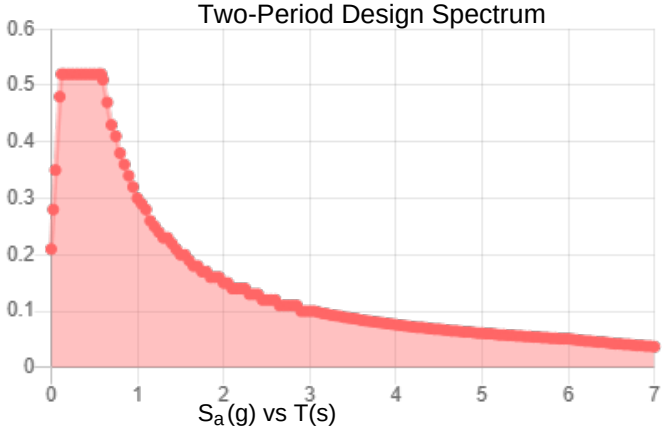
Multi-Period Design Spectrum



Two-Period MCE_R Spectrum



Two-Period Design Spectrum



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

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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JOB TITLE Creech 2 - Admin Criteria

JOB NO. | **SHEET NO.**
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www.struware.com

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

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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

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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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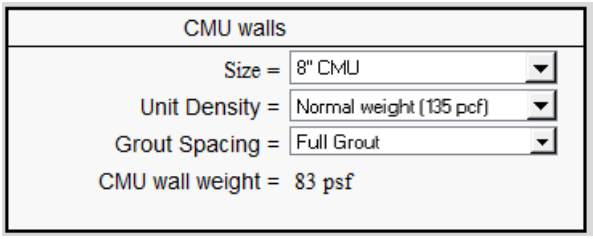
Wall Design Load #1

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



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JOB TITLE	Creech 2 - Admin Criteria		
JOB NO.		**SHEET NO.**	
CALCULATED BY	ACV	**DATE**	10/7/25
CHECKED BY		**DATE**	

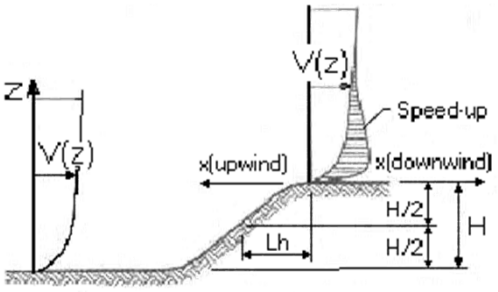
Wind Loads : ASCE 7- 22

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

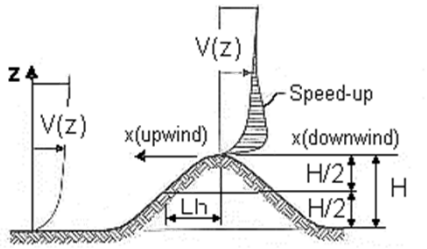
Topographic Factor (Kzt)

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

$\bar{e}$ =	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		

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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)

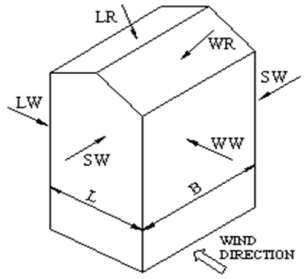
Parapet

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

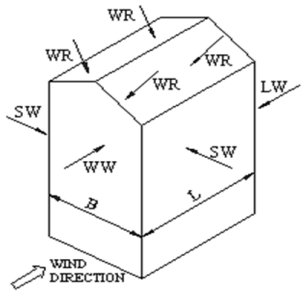
Windward parapet: 37.3 psf (GCpn = +1.5)
Leeward parapet: -24.9 psf (GCpn = -1.0)

Windward Wall Pressures at "z" (psf)

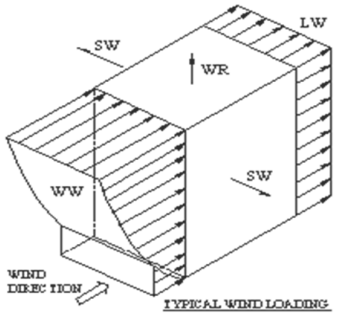
	z	Kz	Kzt	Windward Wall q_zGC_p	Windward Wall w/+q_iGC_{pi}	Windward Wall w/-q_hGC_{pi}	Combined WW + LW Wind Normal to Ridge	Combined WW + LW Wind Parallel to Ridge
	0 to 15'	0.85	1.00	13.9	9.5	18.3	24.2	21.9
	20.0 ft	0.90	1.00	14.7	10.4	19.1	25.0	22.8
	25.0 ft	0.94	1.00	15.4	11.0	19.8	25.7	23.4
	30.0 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 NORMAL TO RIDGE



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

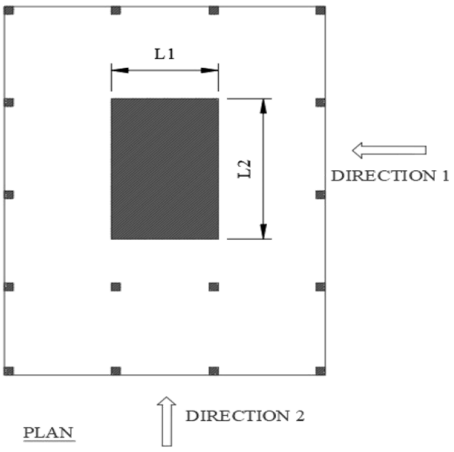
Direction 1 L/B = 0.64 Max L/B = 0.500 OK
Direction 2 L/B = 1.55 Max L/B = 0.500 OK
Meets geometry Limitation No 1 for both directions

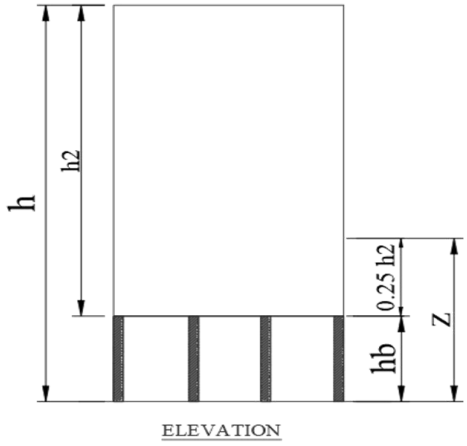
Elevated Building Geometry limitation 2

Direction 1		Direction 2	
Projected width of all columns facing direction 1 =	32.0 ft	Projected width of columns direction 2 =	30.0 ft
Projected L2 width of enclosed areas below bldg =	40.0 ft	Projected L1 width of enclosed areas =	42.0 ft
Total projected width below bldg (width) =	72.0 ft	Total projected width below bldg (width) =	72.0 ft
Projected area ratio =	59.4% OK	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





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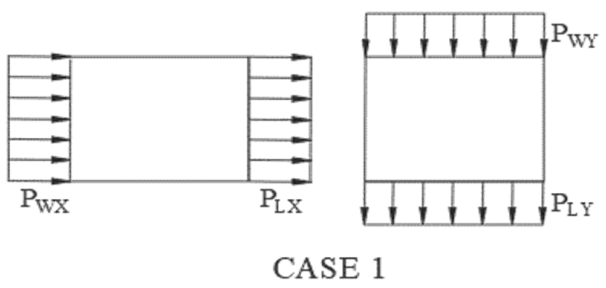
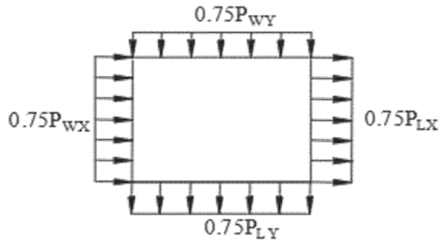
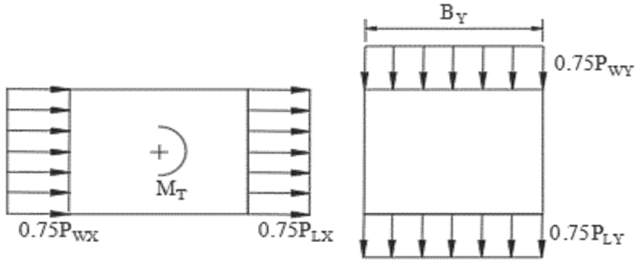
NOTE: ASCE 7 requires the application of full and partial loading of the wind pressures per the 4 cases below.



CASE 1



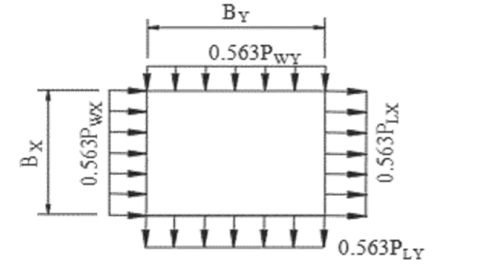
CASE 3



$M_T = 0.75(P_{WX}+P_{LX})B_X e_X$ $\quad e_X = \pm 0.15\ B_X$

$M_T = 0.75(P_{WY}+P_{LY})B_Y e_Y$ $\quad e_Y = \pm 0.15\ B_Y$

CASE 2



$M_T = 0.563(P_{WX}+P_{LX})B_X e_X + 0.563(P_{WY}+P_{LY})B_Y e_Y$

$e_X = \pm 0.15\ B_X$ $\quad e_Y = \pm 0.15\ B_Y$

CASE 4

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			

	Elevation Above	Height of Centroid	Wind Normal to Ridge						Wind Parallel to Ridge				
Level	Grade (ft)	to Fdn (ft)	L	B	Area (sf)	Applied Force (k)	Story Shear (k)	Overturning Moment ('k)	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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JOB TITLE Creech 2 - Admin Criteria
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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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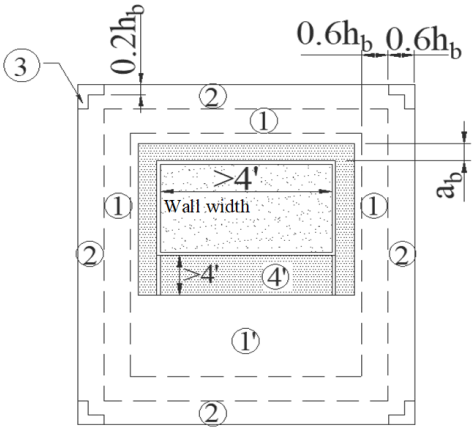
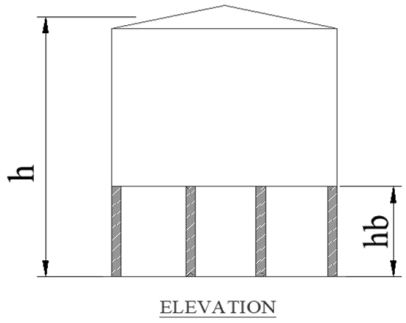
Bottom Horizontal Surface of Elevated Buildings

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

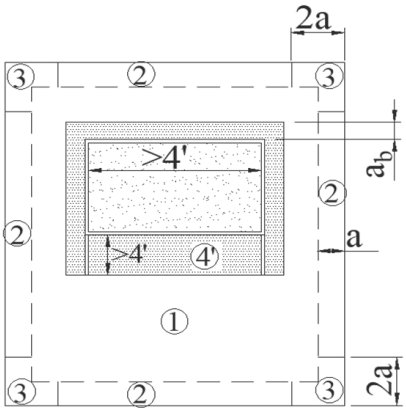
	GCp				Surface Pressure (psf)				User input	
Area	10 sf	100 sf	500 sf	1000 sf	10 sf	100 sf	500 sf	1000 sf	100 sf	100 sf
Negative Zone 1	-1.70	-1.29	-1.00	-1.00	-45.6	-35.6	-28.6	-28.6	-35.6	-35.6
Negative Zone 1'	-0.90	-0.90	-0.55	-0.40	-26.2	-26.2	-17.7	-16.0	-26.2	-26.2
Negative Zone 2	-2.30	-1.77	-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

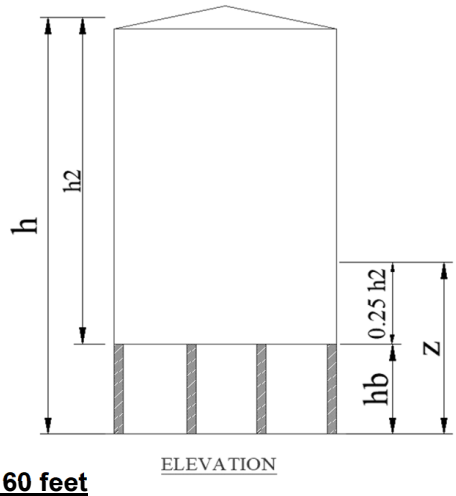
Negative pressures are downward





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





Building Bottom Plan: h> 60 feet

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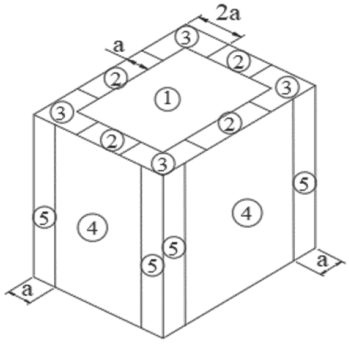
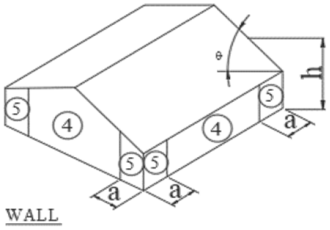
CHECKED BY		**DATE**	

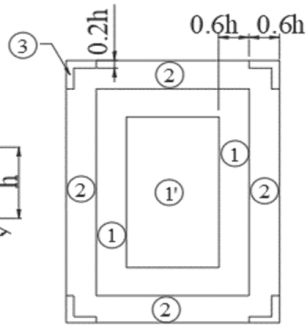
Location of C&C Wind Pressure Zones - ASCE 7-22



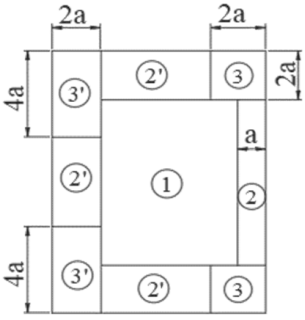
Roofs w/ θ ≤ 10°
and all walls
h > 60'



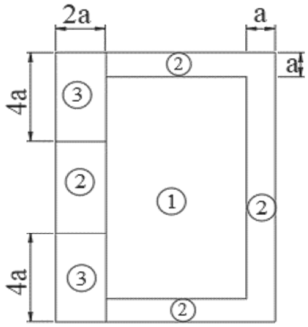
Walls h ≤ 60'
& alt design h<90'



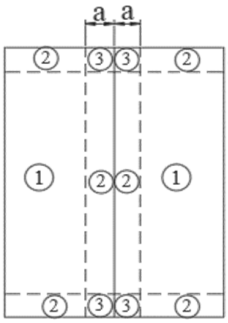
Multispan Gable & Sawtooth ≤ 10°
& Gable θ ≤ 7 degrees &
Monoslope ≤ 3 degrees
h ≤ 60' & alt design h<90'



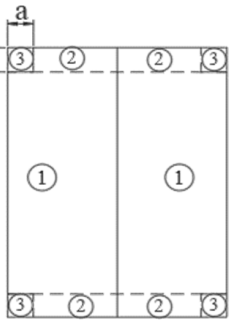
Monoslope roofs
3° < θ ≤ 10°
h ≤ 60' & alt design h<90'



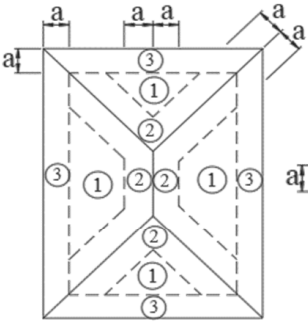
Monoslope roofs
10° < θ ≤ 30°
h ≤ 60' & alt design h<90'



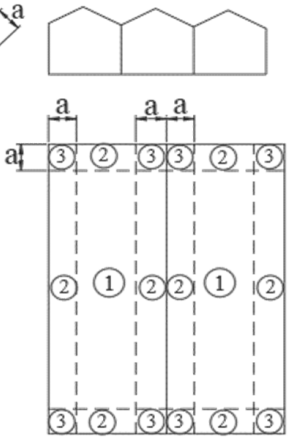
Gable 7° < θ ≤ 27°



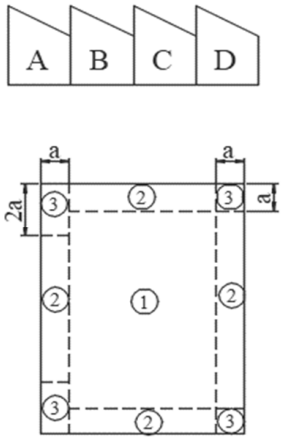
Gable 27° < θ ≤ 45°



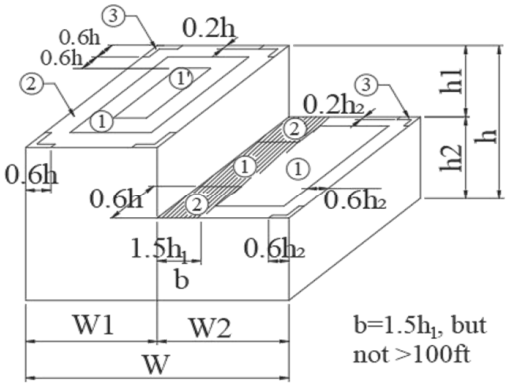
Hip 7° < θ ≤ 45°

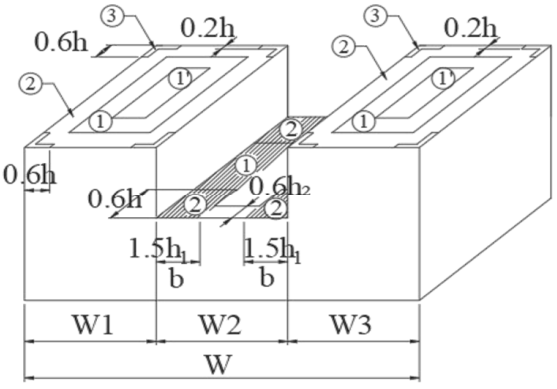


Multispan gable 10° < θ ≤ 45°
h ≤ 60' & alt design h<90'



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.

Michael Baker International
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JOB TITLE Creech 2 - Admin Criteria
JOB NO. **SHEET NO.**
CALCULATED BY ACV **DATE** 10/7/25
CHECKED BY **DATE**

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**

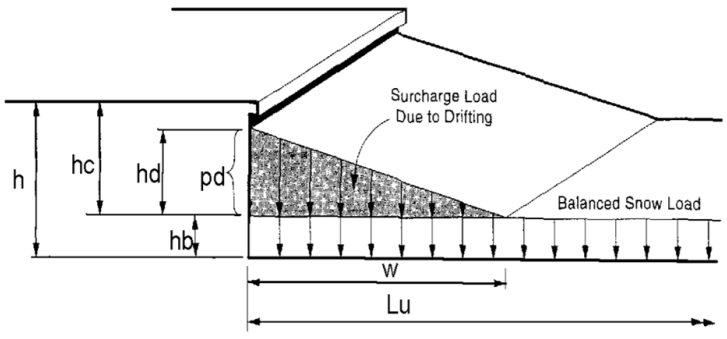
NOTE: Alternate spans of continuous beams shall be loaded with half the design roof snow load so as to produce the greatest possible effect - see code for loading diagrams and exceptions for gable roofs

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.

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JOB TITLE Creech 2 - Admin Criteria

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

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

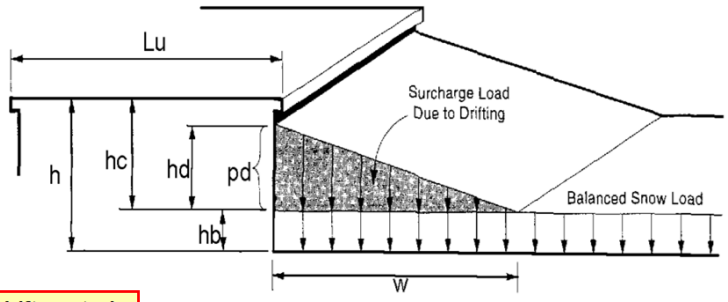
Winter Wind Parameter W2 = 0.35

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.

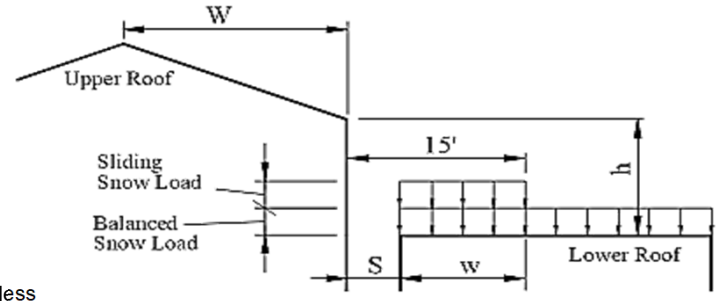
Leeward Snow Drifts - from adjacent higher roof

Upper roof length	lu =	123.0 ft
Snow density	γ =	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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JOB TITLE	Creech 2 - Admin Criteria		
JOB NO.		**SHEET NO.**	
CALCULATED BY	ACV	**DATE**	10/7/25
CHECKED BY		**DATE**	

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
Over-Strength Factor (Ωo) = 2.5
Deflection Amplification Factor (Cd) = 4

To = 0.2(Sd1/Sds) = 0.116
Ts = Sd1/Sds = 0.582
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
Special Seismic Load Effect (Em) = Emh +/- Ev = Ωo Q_E +/- 0.2S_{DS} D = 2.5Qe +/- 0.105D

Q_E = horizontal seismic force
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)

V = 214.9k

Level (x)	EL above Seismic Base hx (ft)	Level Weight Wx (kips)	Wx hx^k (ft-kips)	Cvx = Wx hx^k / Σ Wi hi^k	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

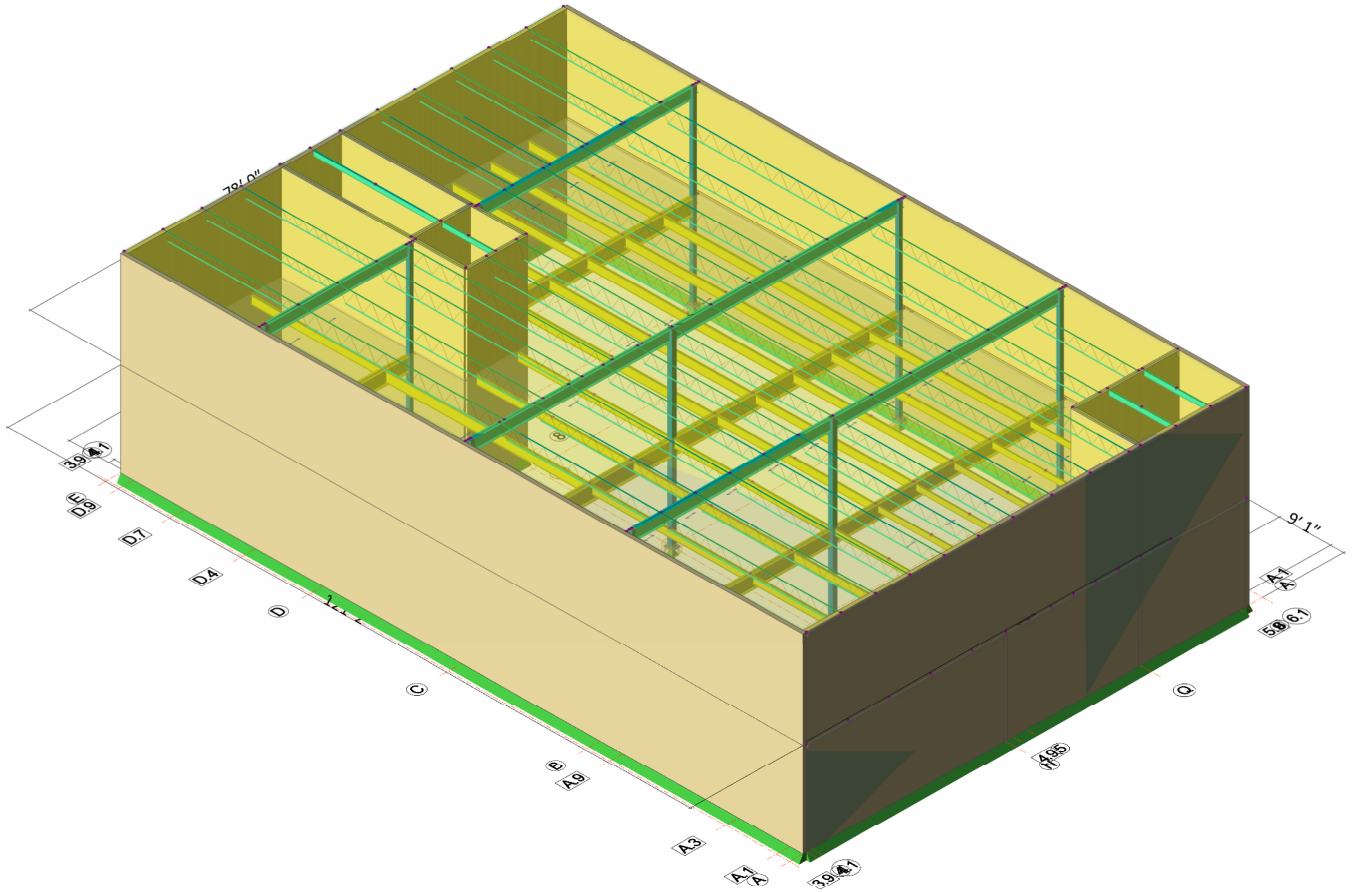
Cvx =	Fx=CvxV	Σ Fx (k)	Σ Wi (k)	Fpx	Design Fpx
0.357	57.11	57.1	288	57.1	57.1
0.643	102.76	159.9	1,336	125.4	137.9
0.000	0.00	0.0	0	0.0	0.0
1.000		159.9			

N/S Seismic Direction

Diaphragm Force Fpx Normal to Bldg Length V= 179k

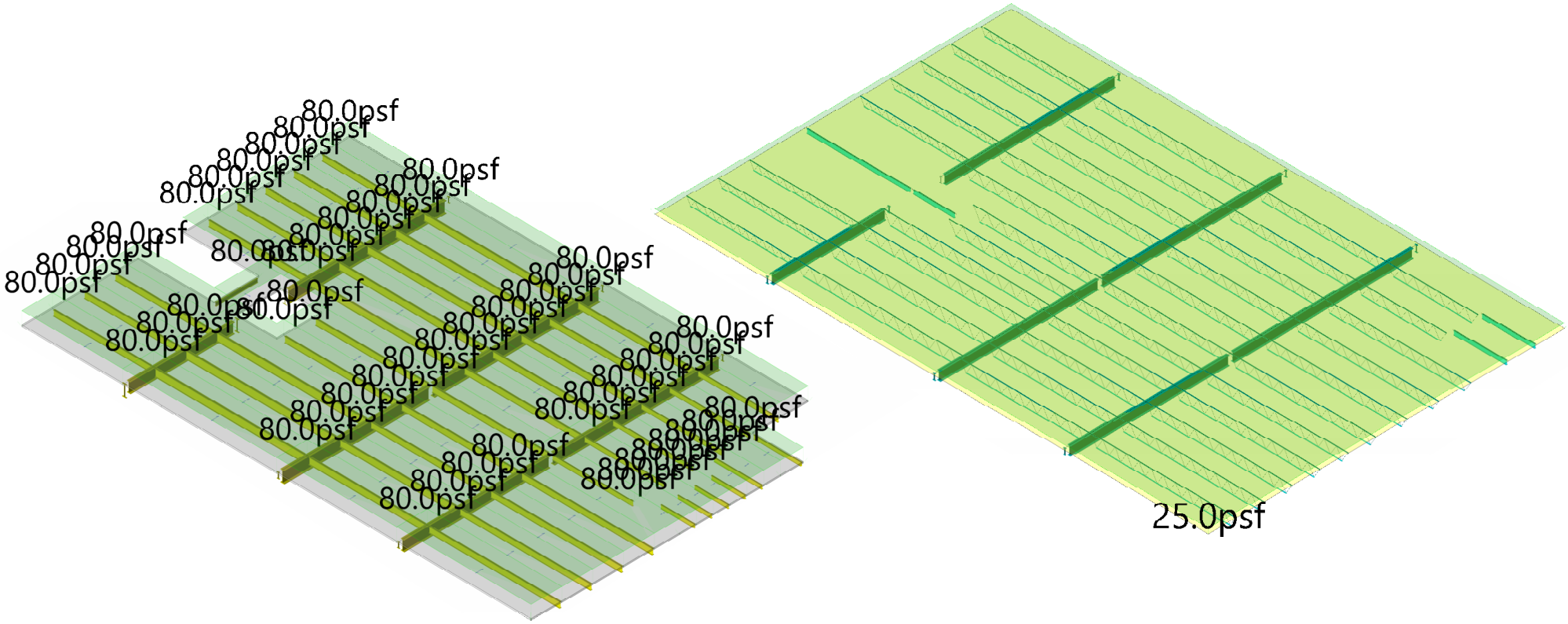
Level (x)	Cvx =	Fx=CvxV	Σ Fx (k)	Σ Wi (k)	Fpx	Design Fpx
Roof	0.375	67.3	67.3	343	67.3	67.3
2	0.625	112.2	179.5	1,499	138.4	152.2
1	0.000	0.0	0.0	0	0.0	0.0
Base	1.000		179.5			

E/W Seismic Direction



Structure

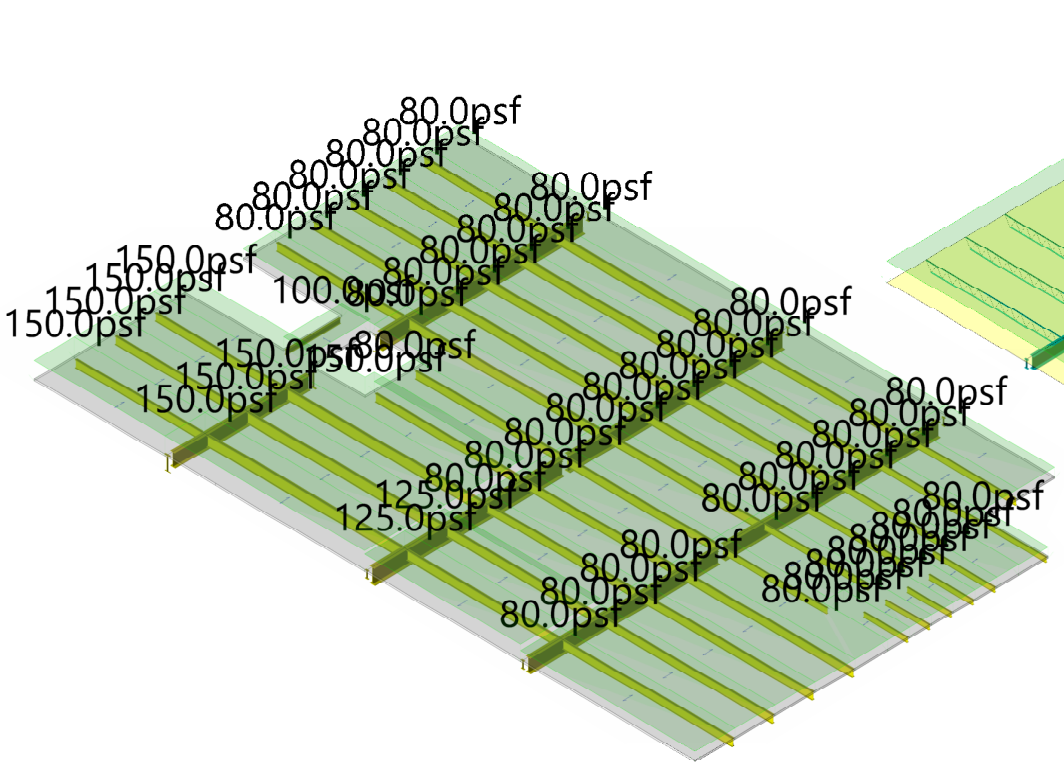
Tekla Structural Designer, version: 25.0.1.45



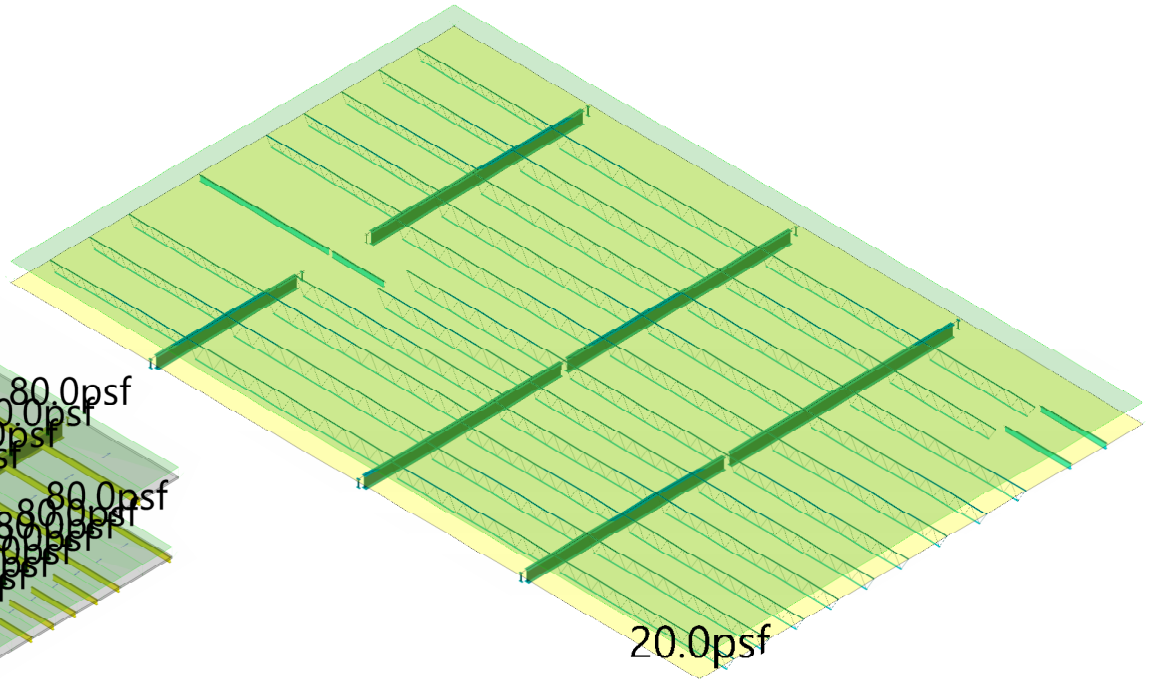
2nd Floor

Roof

Tekla Structural Designer: Dead Loading

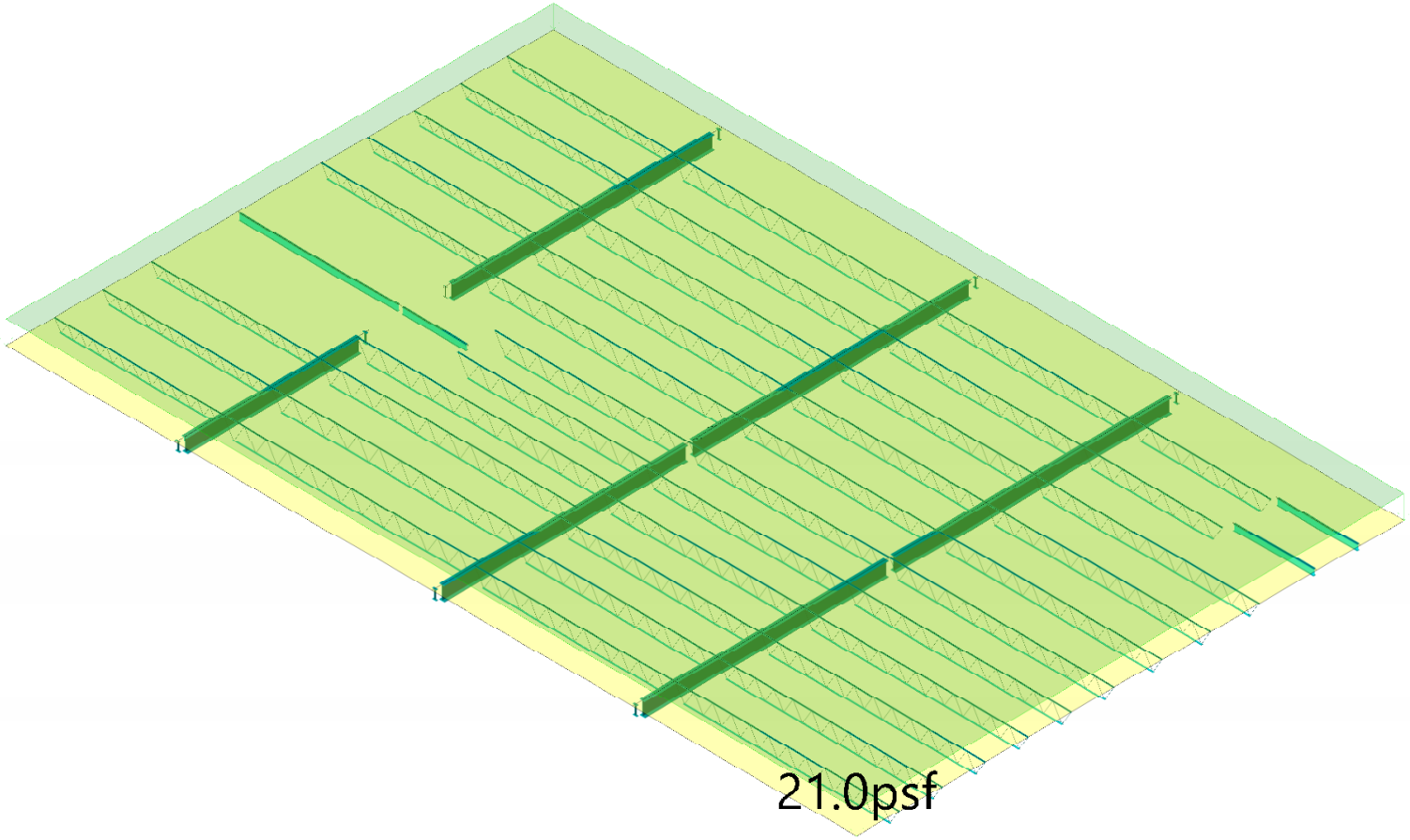


2nd Floor

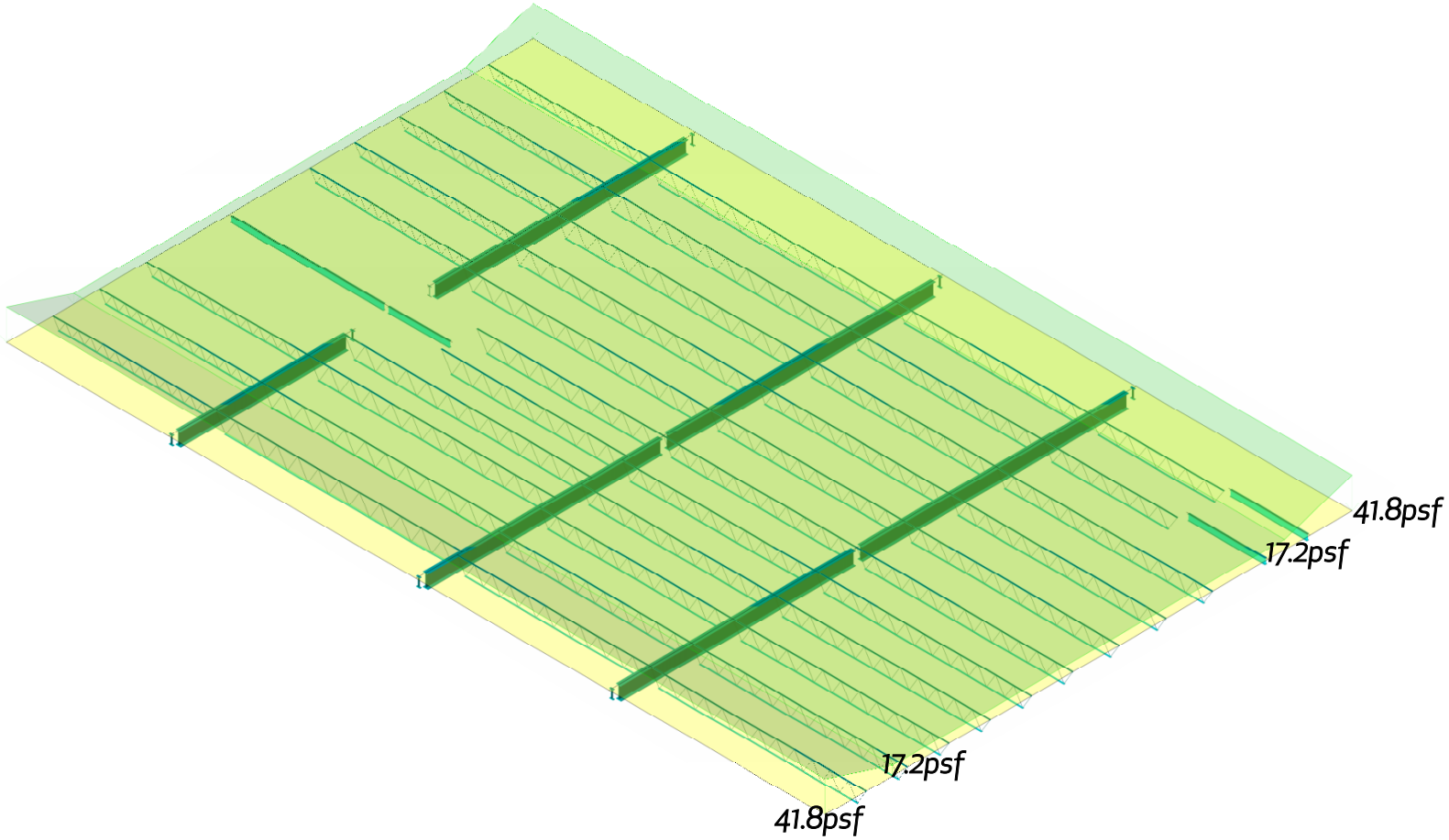


Roof

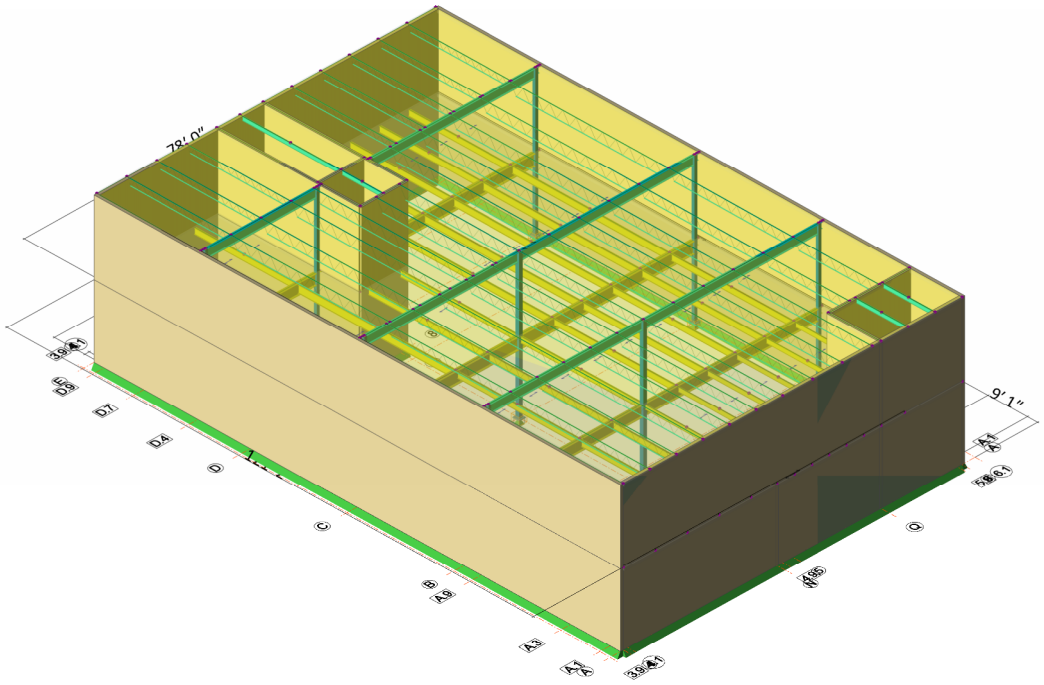
Tekla Structural Designer: Live/Roof Live Loading



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

Construction Levels

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 [ft²]	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 [ft²]	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 [ft²]	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

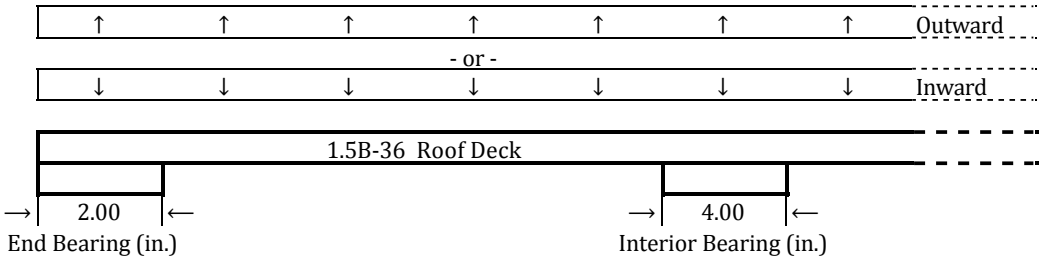
20 Gage 1.5B-36 Grade 50

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) = 62psf

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: $W \le \Phi Wn$

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

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Vulcraft Open Web Steel Joist Depth Selector

Parallel Chord Joist Only

Key: R4cnlF4d3gg

NUCOR® VULCRAFT

Project Information:

Project: CREECH DRP PH 2

Design Professional: ACV

Allowable Stress Design

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.

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Steel

Beam Design

Beam Design Summary

Static

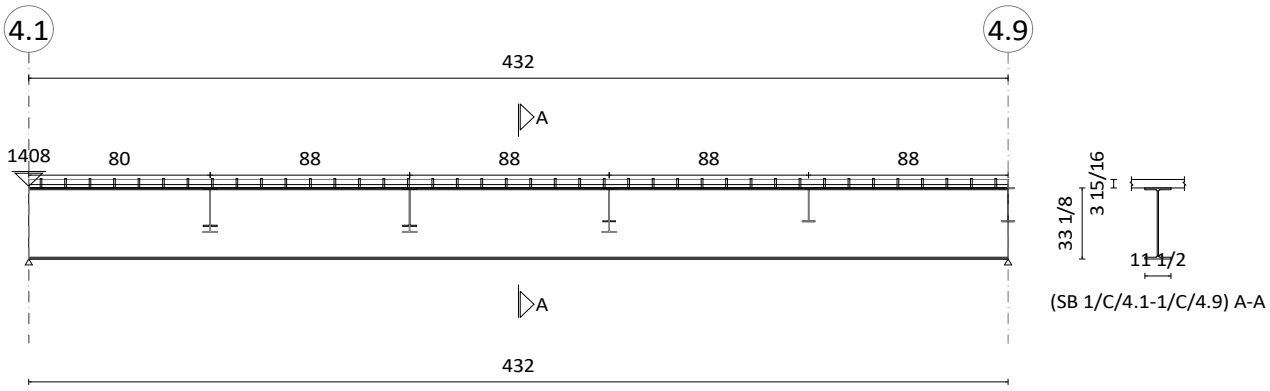
Member Reference	Group Ref.	Span	Section	Grade	Length [ft, in]	No. Connectors	Camber	Utilization	Status
SB 1/D.4/#443-1/E/#444	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.497	Pass
SB 1/D.4/#445-1/E/#446	SBRC11	1	W 16x26	A992-50	24′ 0″	21	0.00	0.467	Pass
SB 1/%69-1/A/#1043	SBRC18	1	W 8x10	A992-50	7′ 0″	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

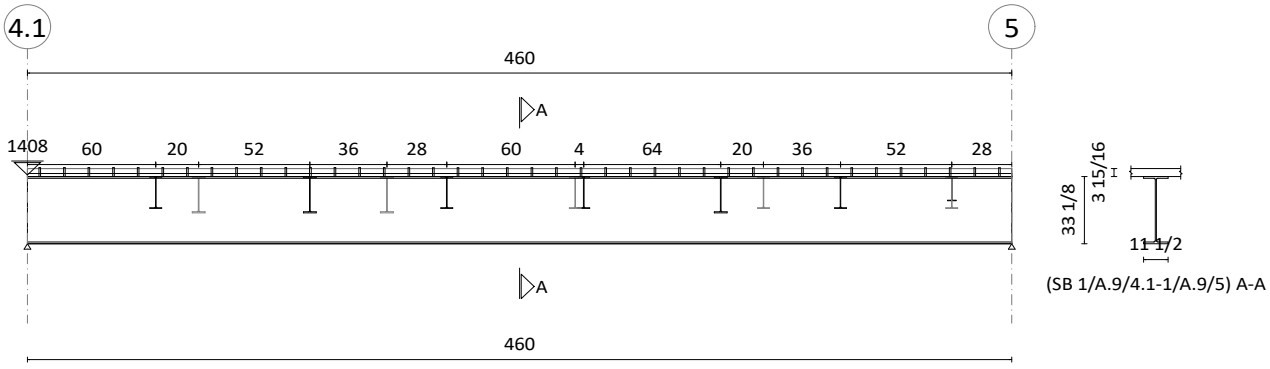
Maximum group spacing, s	=**10 51/64** in
Maximum permitted group spacing	=**31 1/2** in

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

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

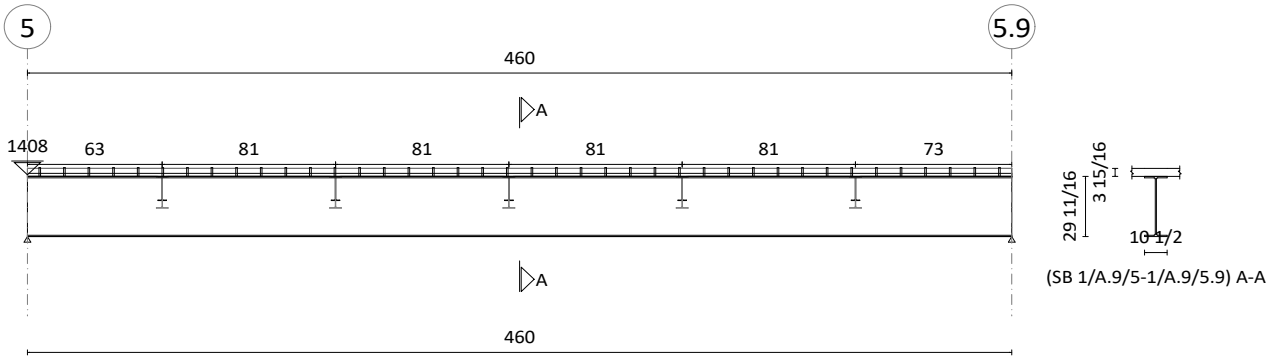
Span 1 W 33x130 A992-50 - Critical

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



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			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 (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

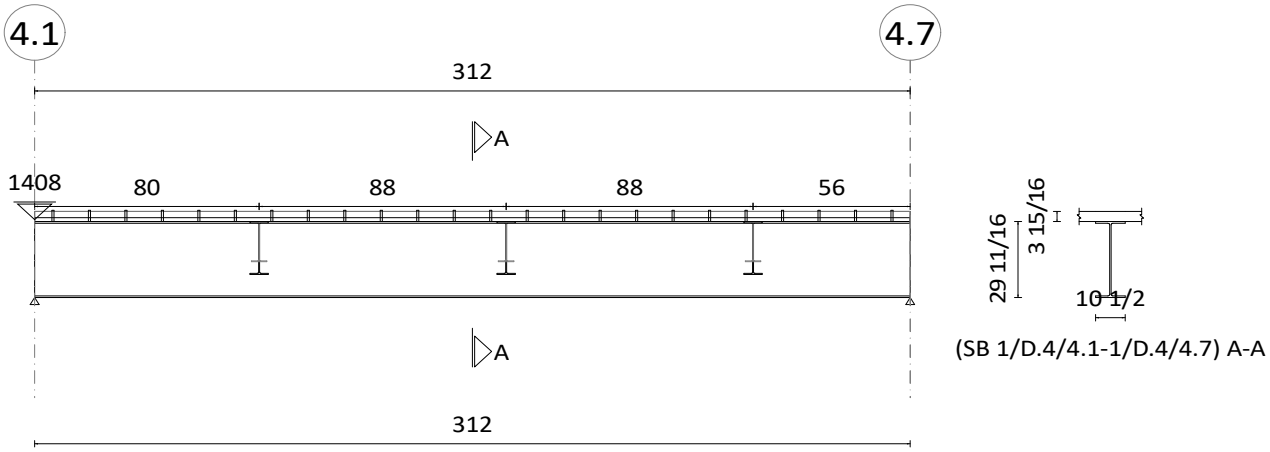
Span 1 W 30x99 A992-50 - Critical

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

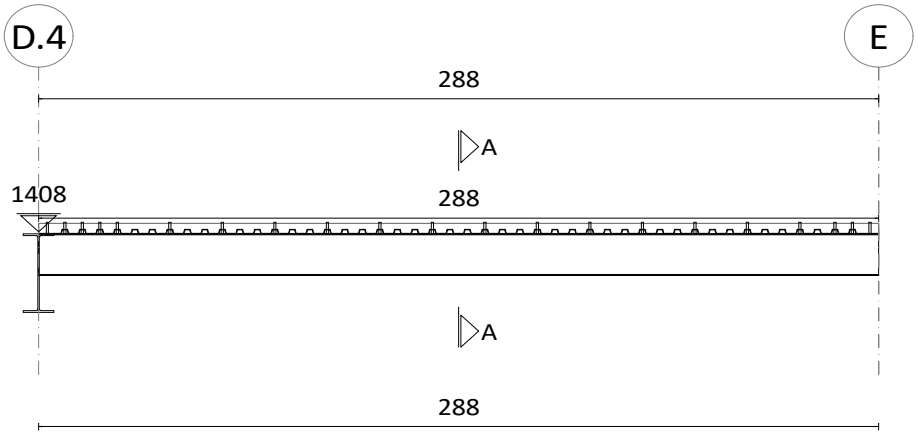
Maximum permitted group spacing = **31 1/2** in

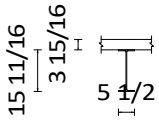
Number of shear connectors over full beam length	24
Number of shear connectors to critical point, N_a	11
Degree of shear connection	=**0.259**
Absolute minimum degree of shear connection	=**0.250**
Optimum amount of shear connection	=**0.500**

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

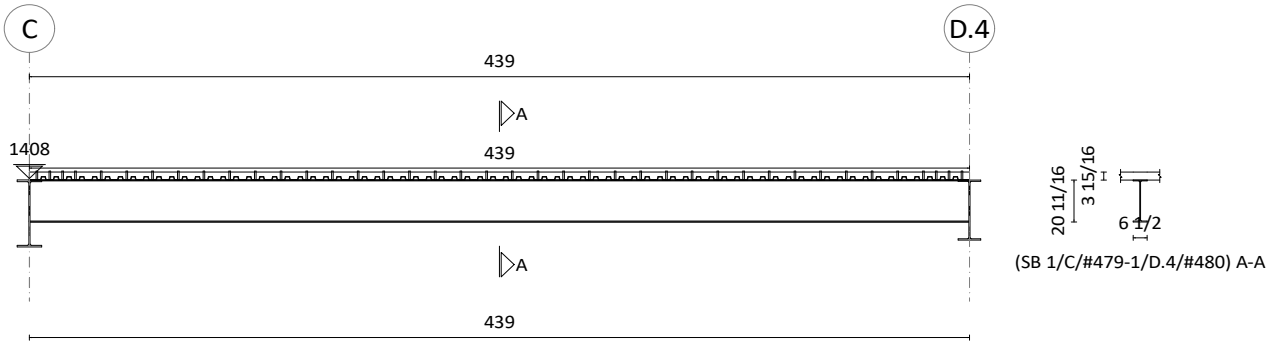
14 (Operating) LRFD$_{11.1}$-1.2D+L+0.15S+E - Critical

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



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

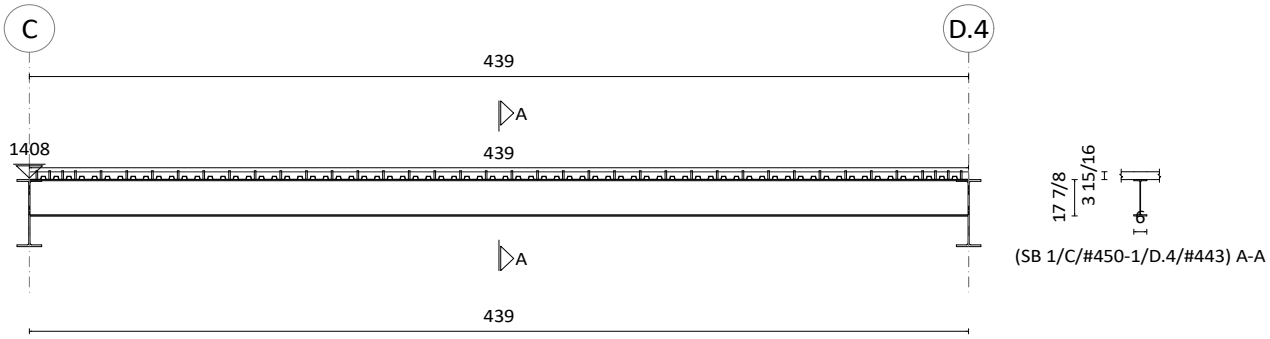
Span 1 W 21x44 A992-50 - Critical

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

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

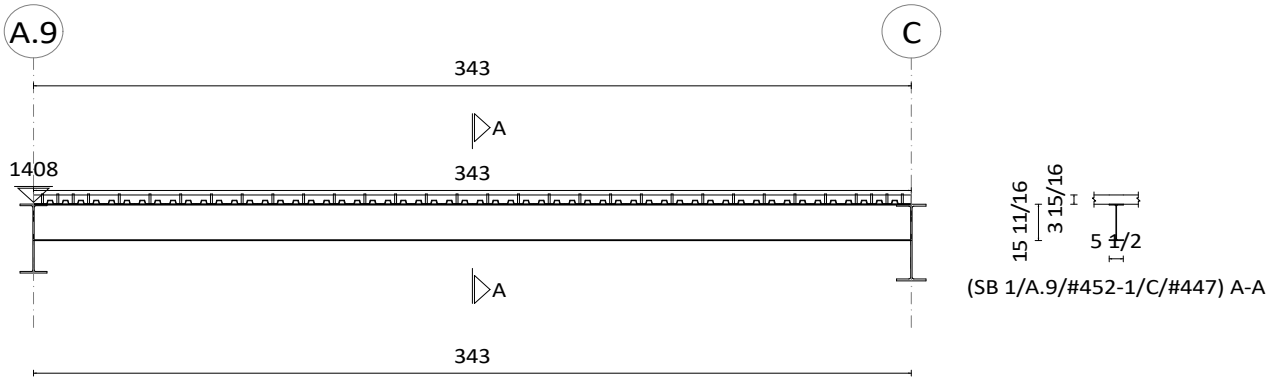
14 (Operating) LRFD$_{11.1}$-1.2D+L+0.15S+E - Critical

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



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

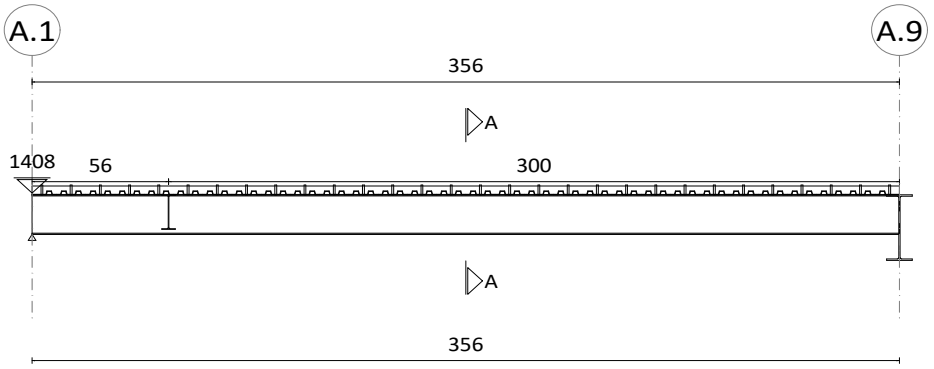
14 (Operating) LRFD$_{11.1}$-1.2D+L+0.15S+E - Critical

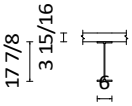
Span 1 W 16x26 A992-50 - Critical

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

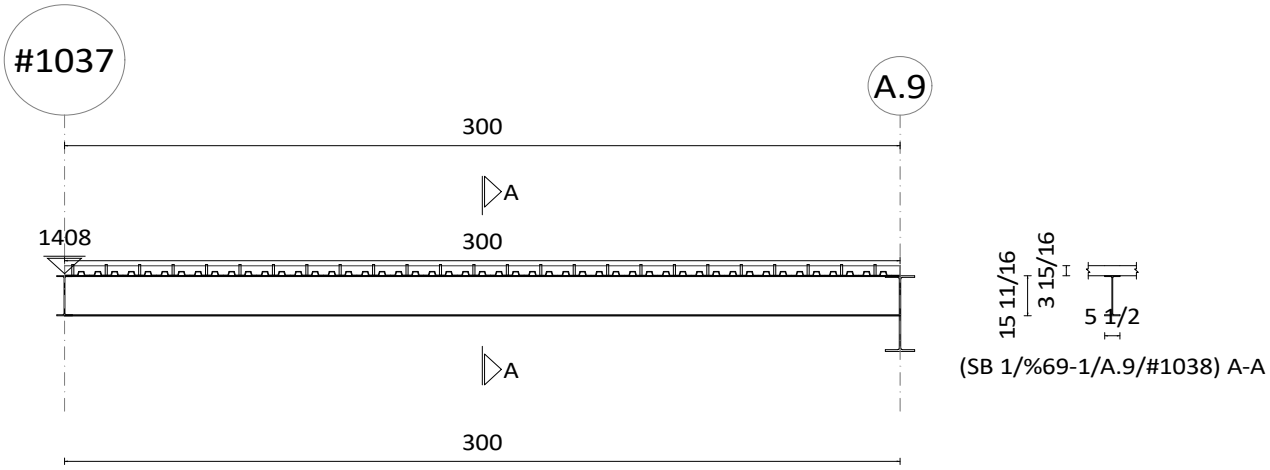
Span 1 W 18x40 A992-50 - Critical

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

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

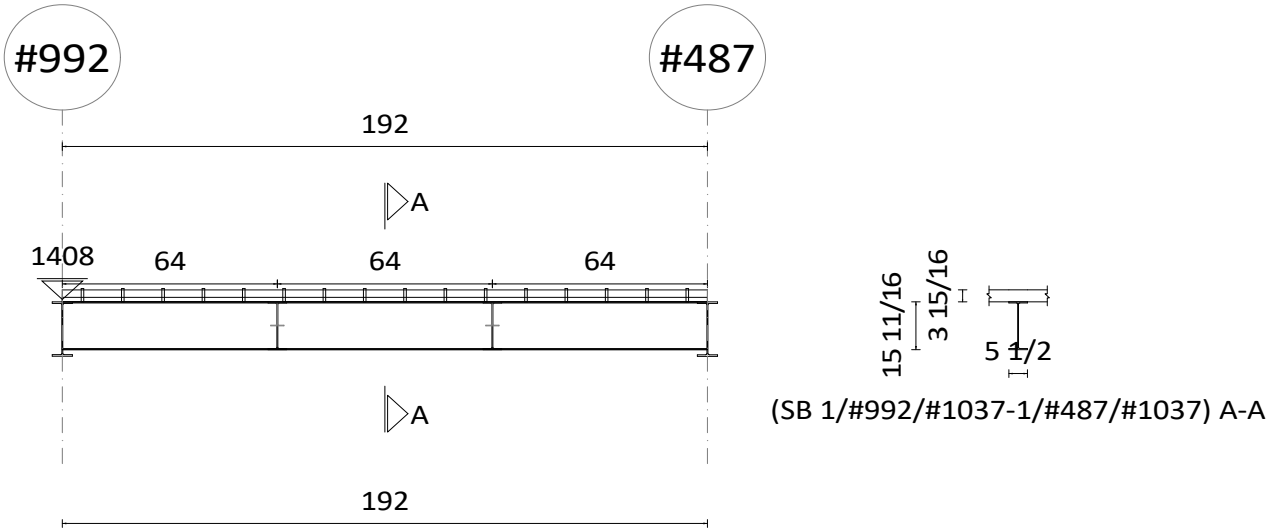
14 (Operating) LRFD$_{11.1}$-1.2D+L+0.15S+E - Critical

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

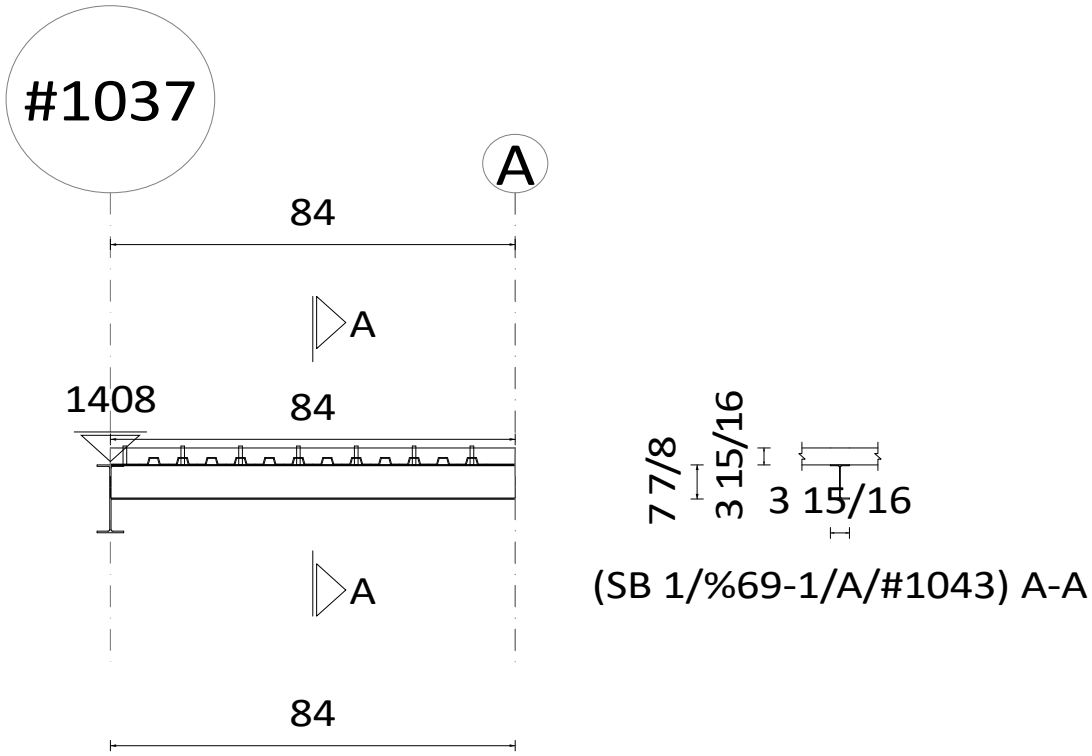
Span 1 W 16x26 A992-50 - Critical

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

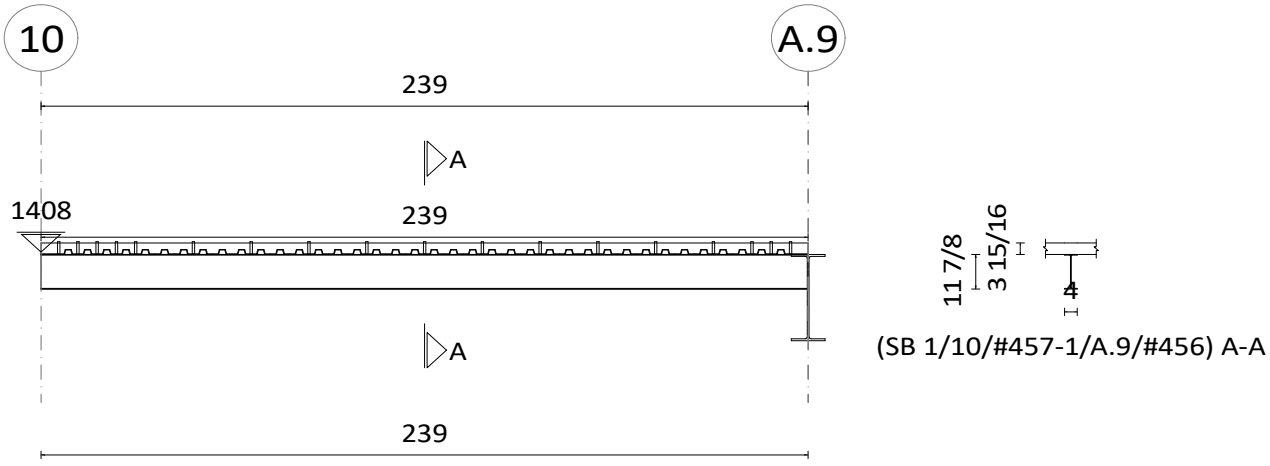
Span 1 W 8x10 A992-50 - Critical

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



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

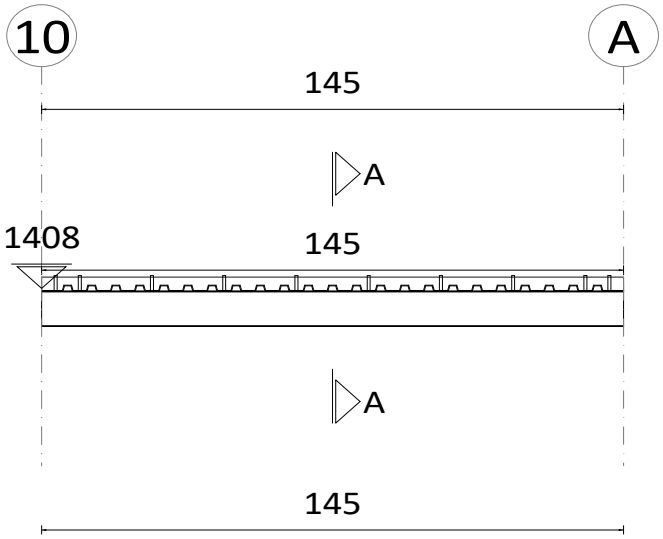
Maximum permitted group spacing =**31 1/2** in
Number of shear connectors over full beam length 18
Number of shear connectors to critical point, N_a 8
Degree of shear connection =**0.663**
Absolute minimum degree of shear connection =**0.250**
Optimum amount of shear connection =**0.500**
Partial shear connection
Note: Stud dia. > 2.5 x flange thickness
Warning

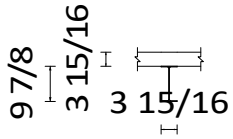
Deflection

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

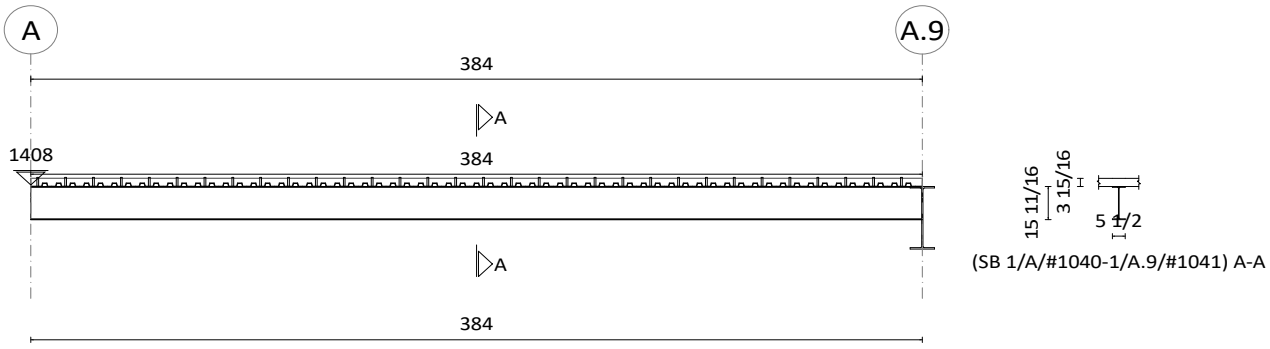
Span 1 W 10x12 A992-50 - Critical

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

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

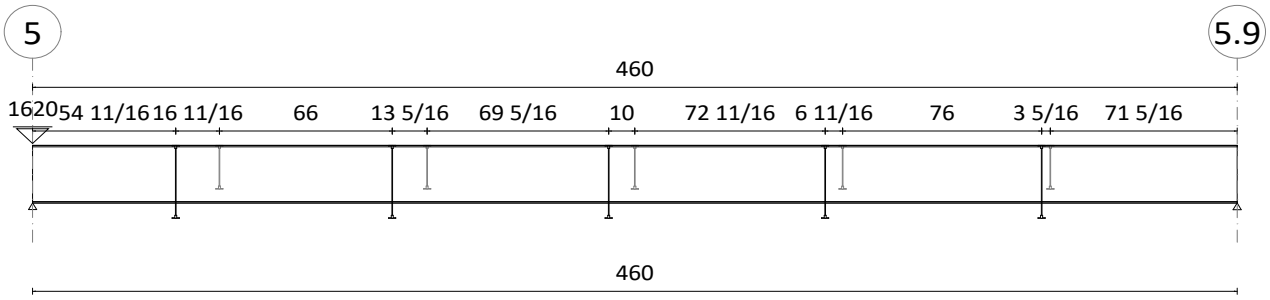
Short term inertia, $I_{\alpha S}$	=**955.7** in^4
Long term inertia, $I_{\alpha L}$	=**733.6** in^4
Short term inertia @ partial interaction, $I_{\alpha S}$	=**855.7** in^4
Long term inertia @ partial interaction, $I_{\alpha L}$	=**667.6** in^4
Position Total load deflection	=**16' 0"** ft, in
Deflection 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

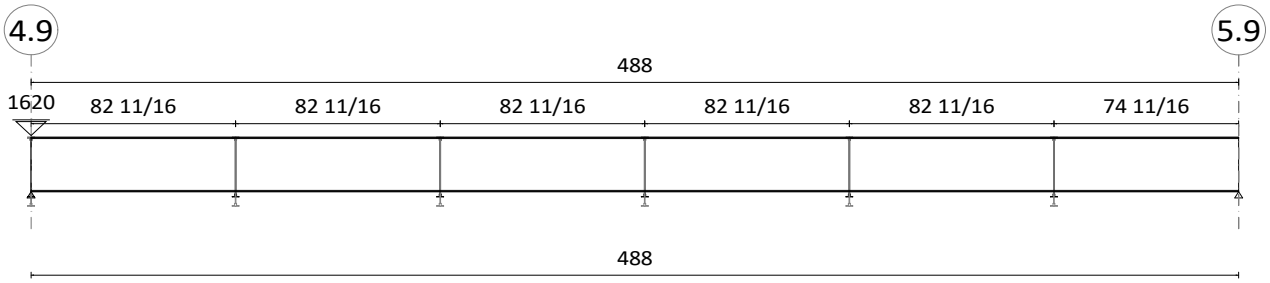
Summary W 24x68

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	Tributary Area [ft^2]	K_{LL} Factor
4 Live	No				
5 Roof Live	No				

Static Design Summary

Summary W 24x68

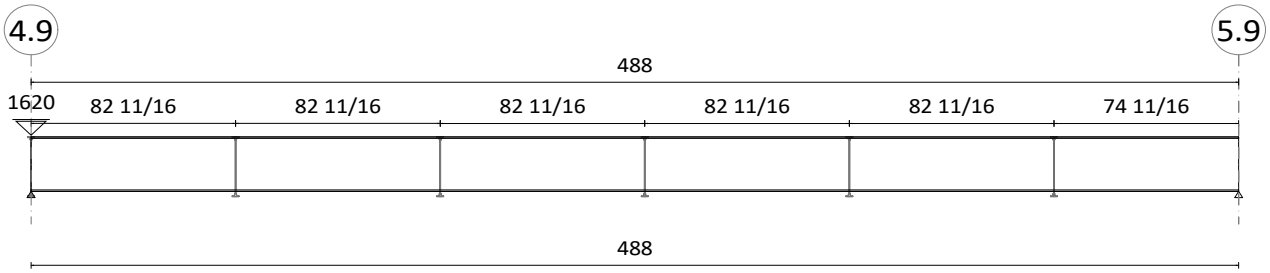
Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	14	Slender	-	-	-	✔ Pass
Flexural Classification	7	Compact	-	-	-	✔ Pass
Shear Major	11	-37.9	295.1	kip	0.129	✔ Pass
Shear Minor	17	3.0	283.4	kip	0.011	✔ Pass
Flexure Major	11	447.4	663.7	kip ft	0.674	✔ Pass
Flexure 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

Steel

Column Design

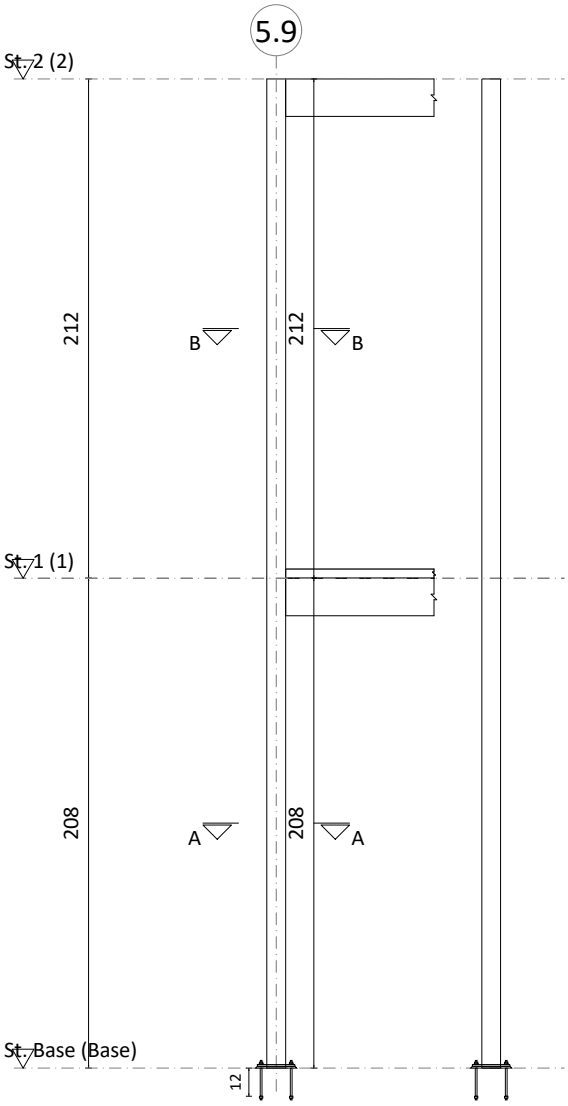
Column Design Summary

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.

SC D.4/5.9



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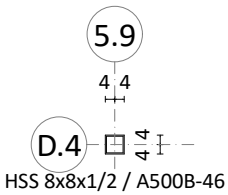

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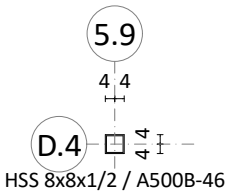

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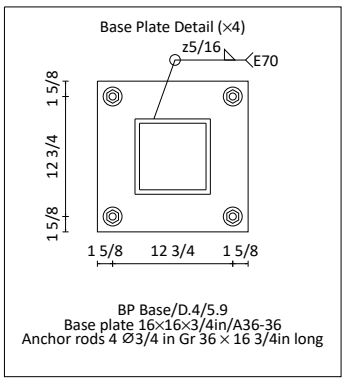


Loading

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

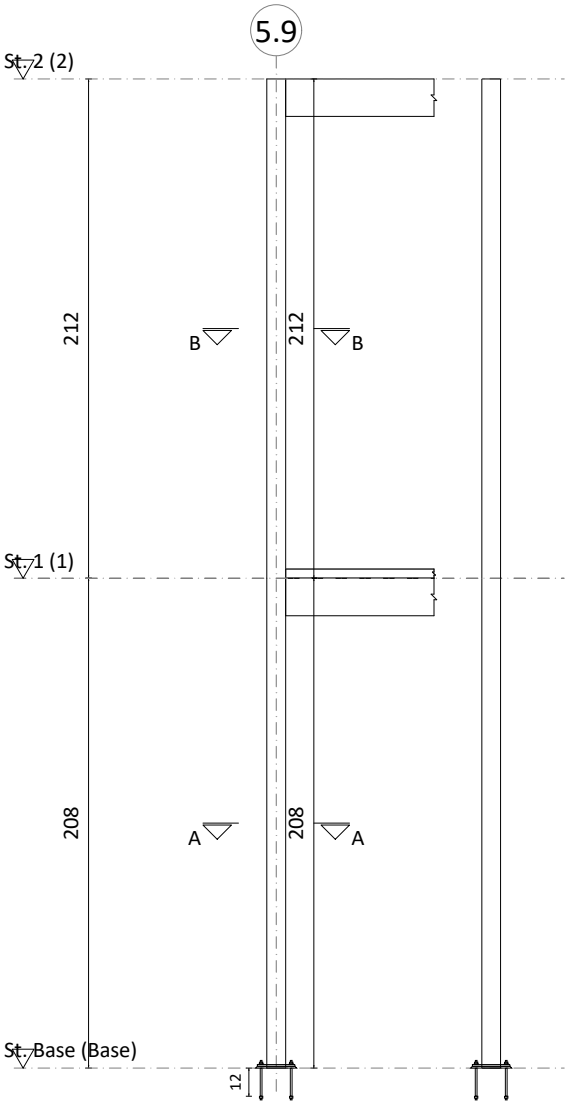
Level	Source	Distance / Length	Major restrained / Sub-stack	...laj... ...ct...	Minor restrained / Sub-stack	...in... ...ct...
3	floor	35′ 0″	No		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

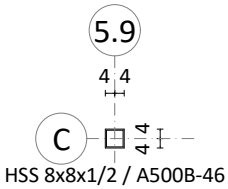
Summary HSS 8x8x1/2 (A500B-46)

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

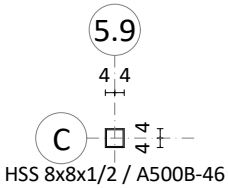
SC C/5.9



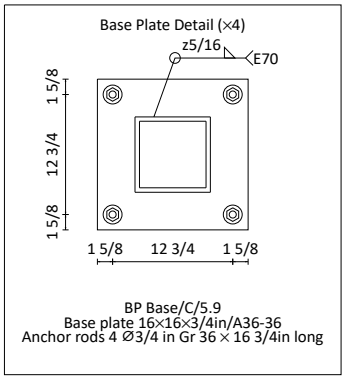
SC C/5.9 B-B



SC C/5.9 A-A



SC C/5.9



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

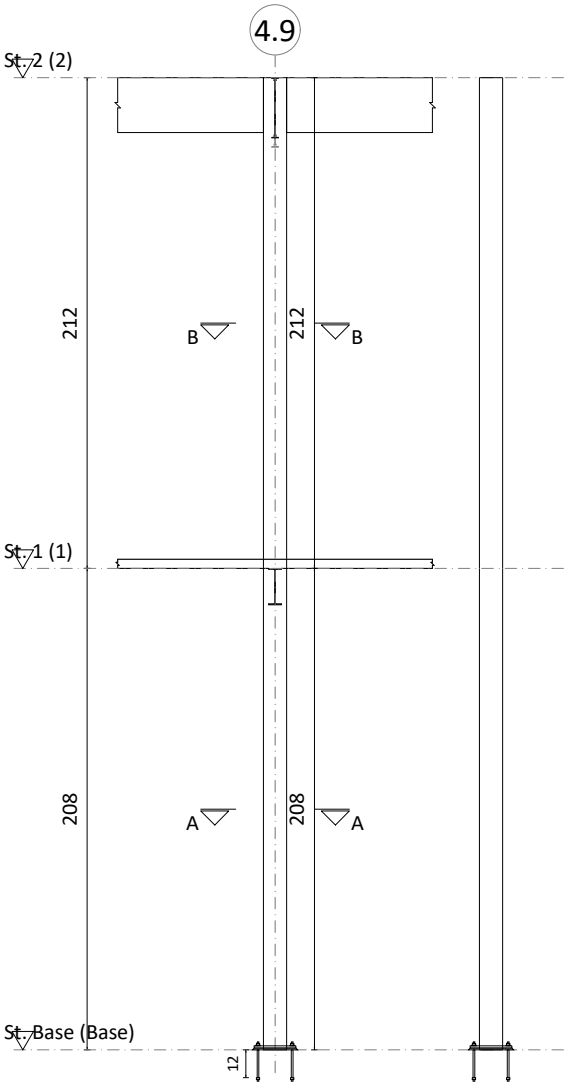
Level	Source	Distance / Length	Major restrained / Sub-stack	...laj... ...ct...	Minor restrained / Sub-stack	...in... ...ct...
3	floor	35′ 0″	No		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

Static Design Summary

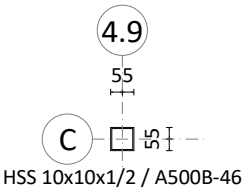
Summary HSS 8x8x1/2 (A500B-46)

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

SC C/4.9

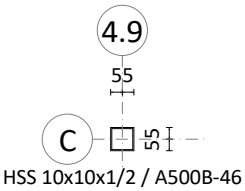


SC C/4.9 B-B



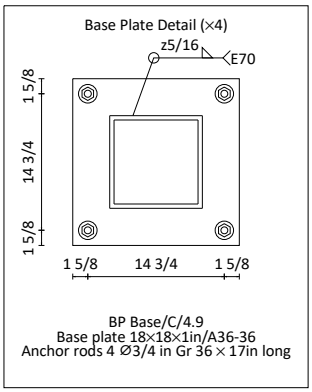
HSS 10x10x1/2 / A500B-46

SC C/4.9 A-A



HSS 10x10x1/2 / A500B-46

SC C/4.9



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

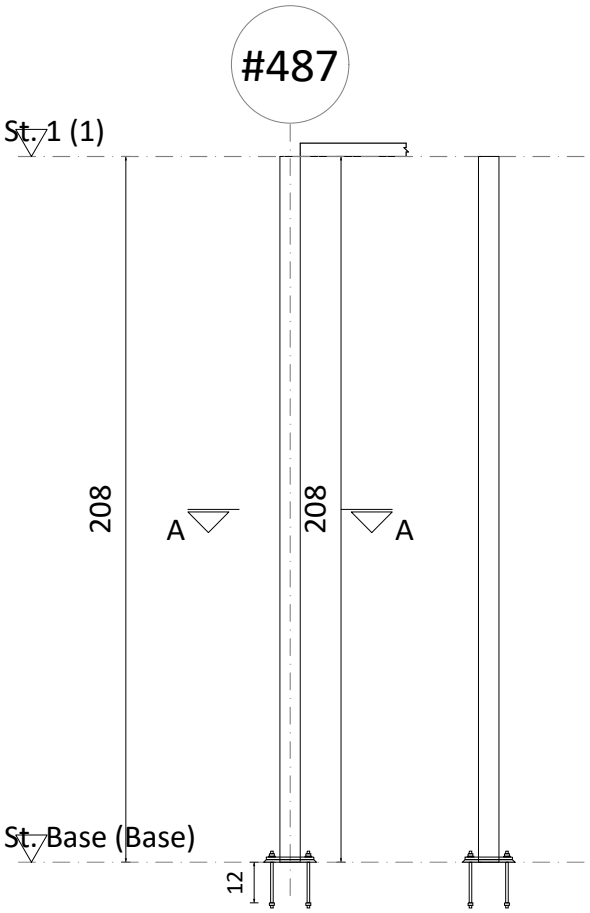
Level	Source	Distance / Length	Major restrained / Sub-stack	...laj... ...ct...	Minor restrained / Sub-stack	...in... ...ct...
3	floor	35′ 0″	Yes		Yes	
	sub-beam	17′ 8″	No	1.000	No	1.000
2	floor	17′ 4″	Yes		Yes	
	sub-beam	17′ 4″	No	1.000	No	1.000
1	floor	0″	Yes		Yes	

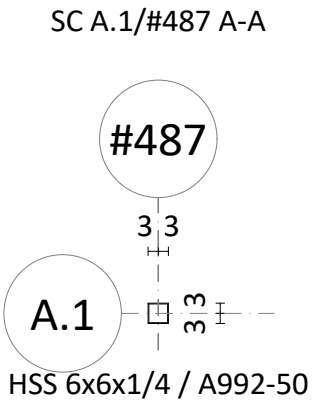
Static Design Summary

Summary HSS 10x10x1/2 (A500B-46)

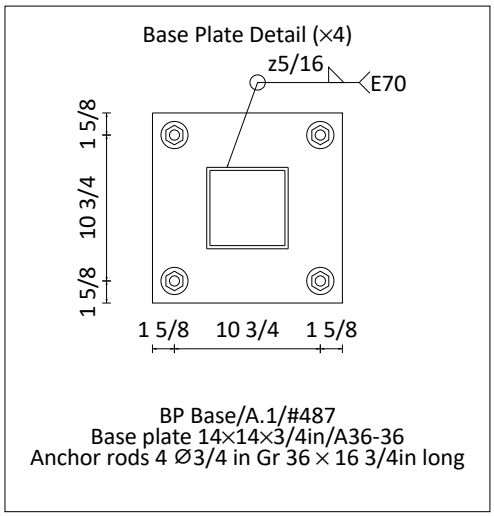
Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Axial Classification	7	Non-slender	-	-	-	Pass
Flexural Classification	7	Compact	-	-	-	Pass
Shear Major	17	0.7	198.8	kip	0.004	Pass
Shear Minor	15	3.2	198.8	kip	0.016	Pass
Flexure Major	17	13.0	209.4	kip ft	0.062	Pass
Flexure Minor	18	-57.8	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



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	...laj... ...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

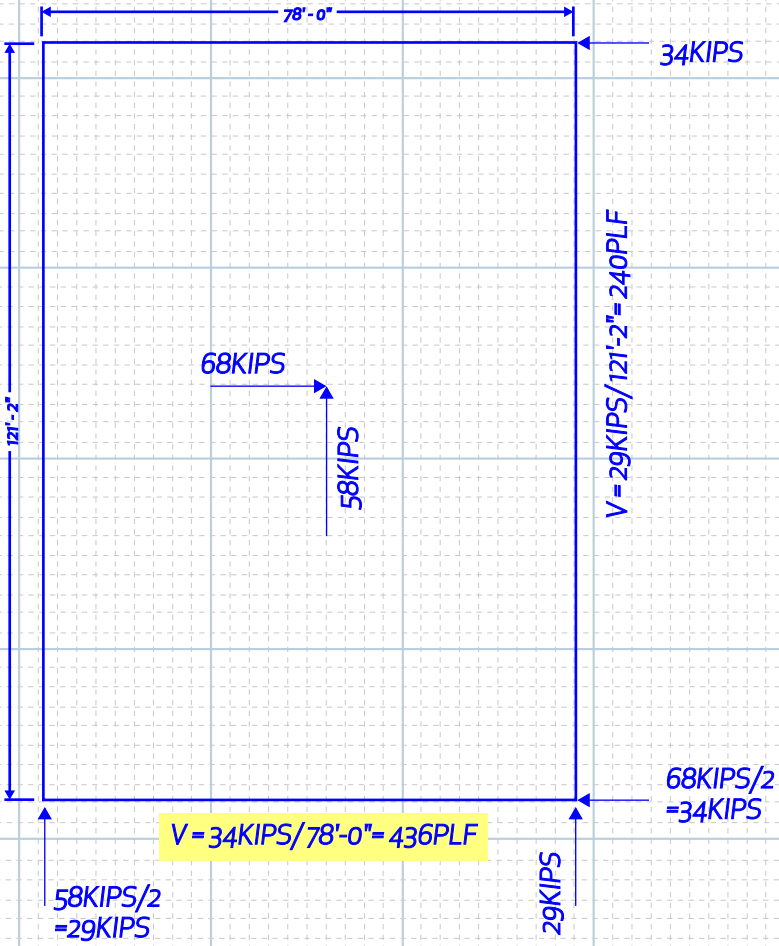
Drawing No.

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

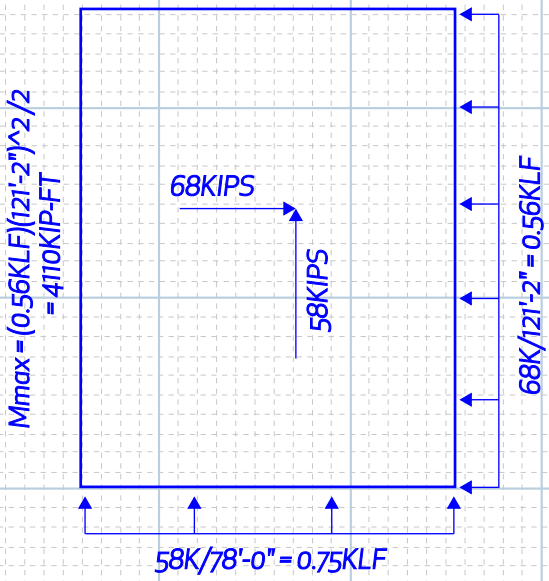
Michael Baker INTERNATIONAL

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.

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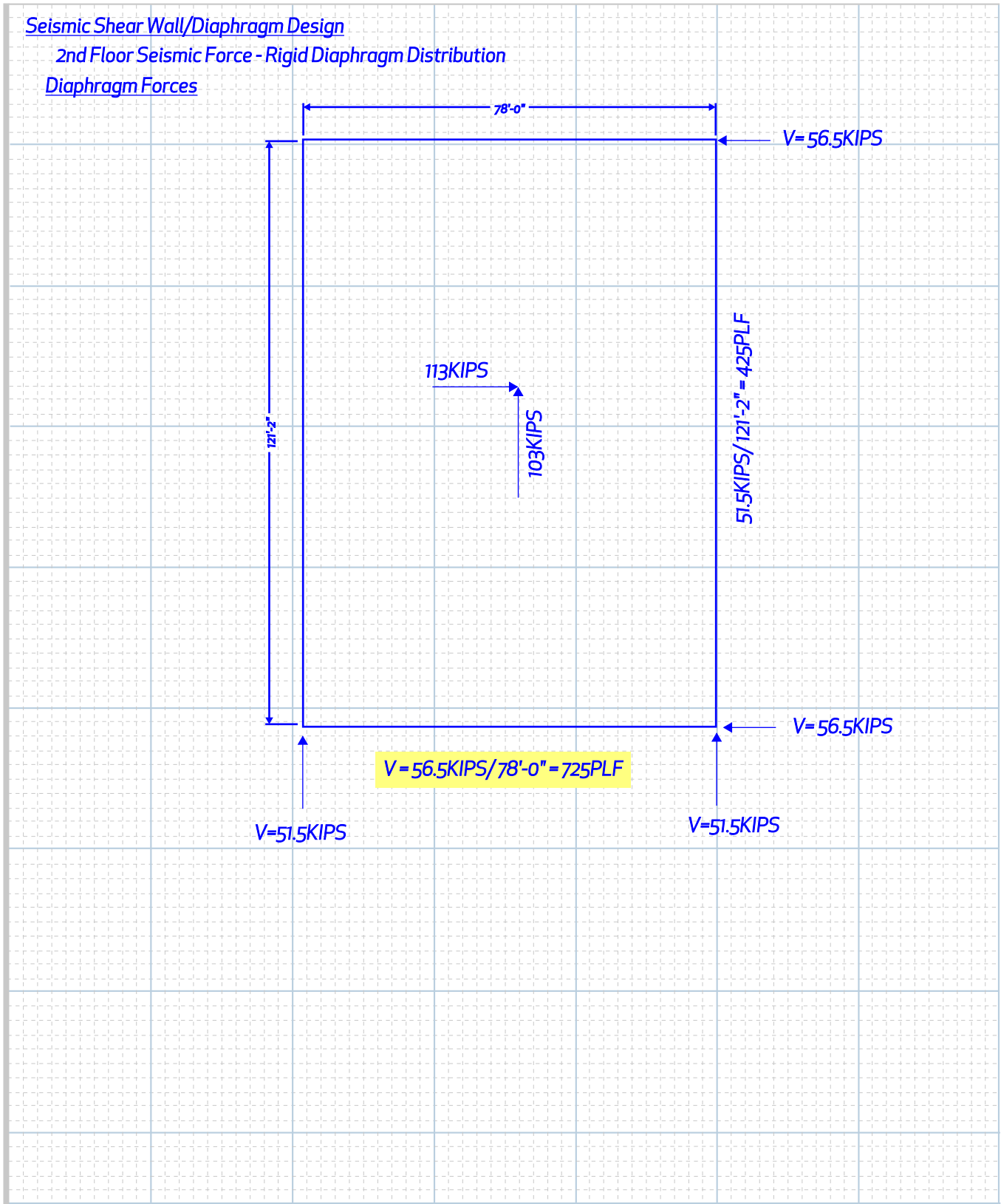
Michael Baker INTERNATIONAL

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

Sheet No. 1 of 1

Drawing No.

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

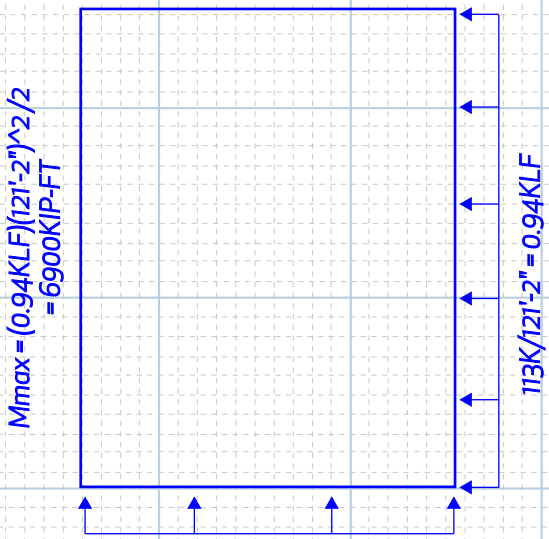
Michael Baker INTERNATIONAL

Seismic Shear Wall/Diaphragm Design

2nd Floor Seismic Force - Flexible Diaphragm Distribution

Chord Forces

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



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

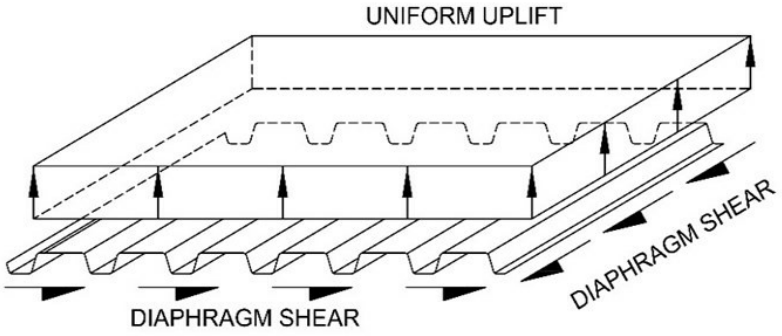
A572 GR50 Support Member or Equivalent
0.188 ≤ Support Thickness (in.) ≤ 0.375
2 in. Minimum Deck End Bearing Length

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

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

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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.

NUCOR® VULCRAFT

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

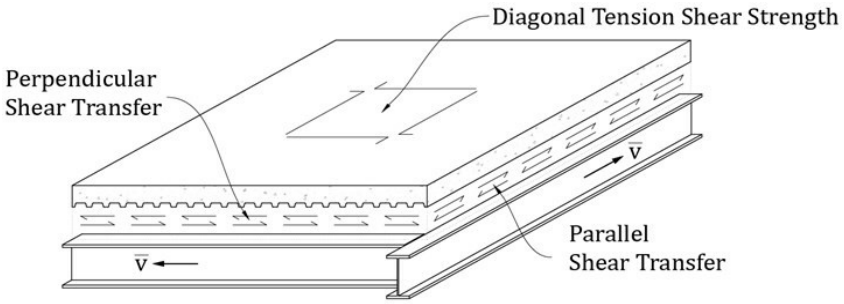
Available Diaphragm Design Strength Controlled by Connections to Chords and Collectors	**Va = (ΦPn)N =**	**1257**	**plf**
Deck-Slab Diaphragm Design Shear Stiffness	G' =	1657	kip/in

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

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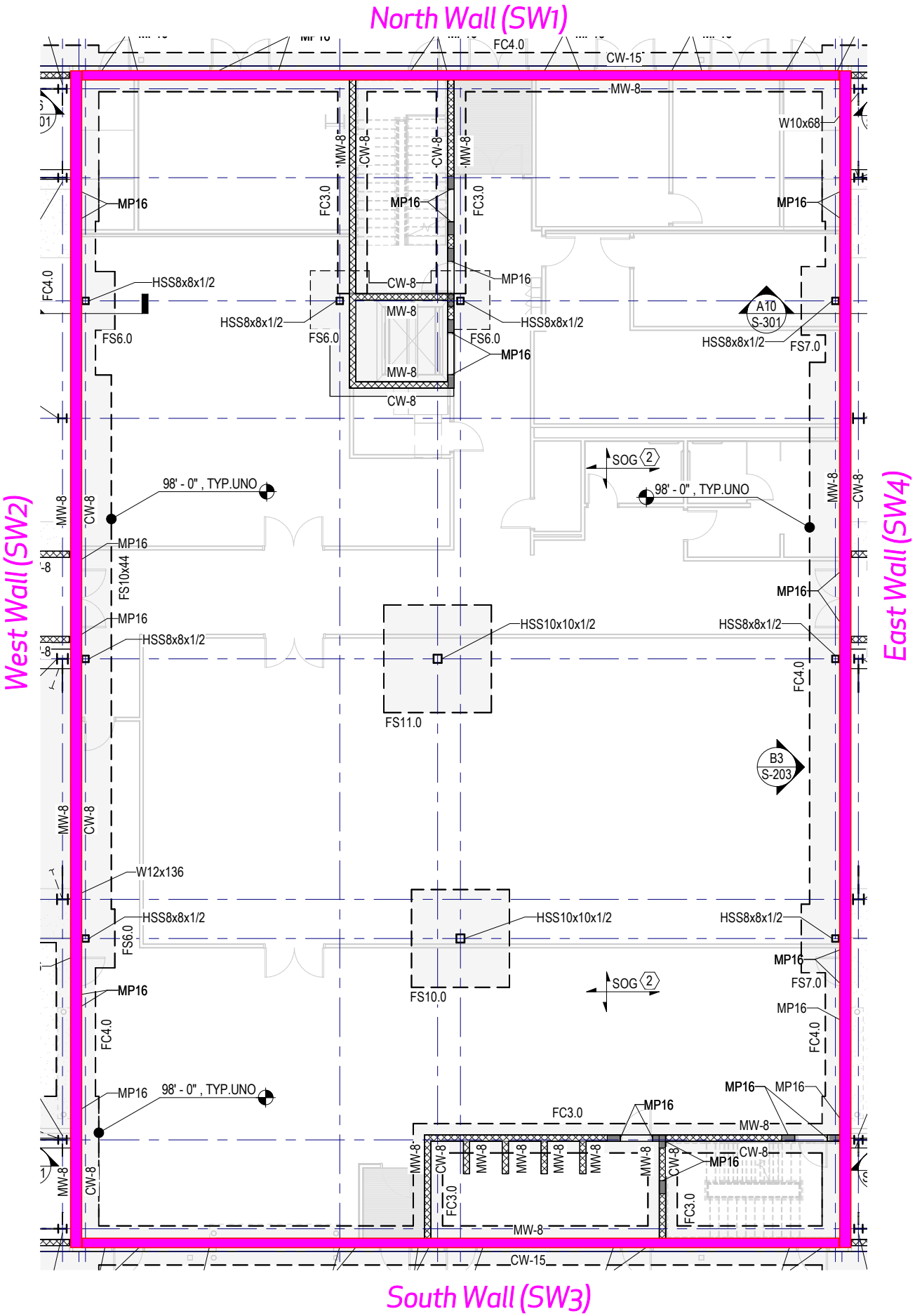
Shear Wall Plan/Labels



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		

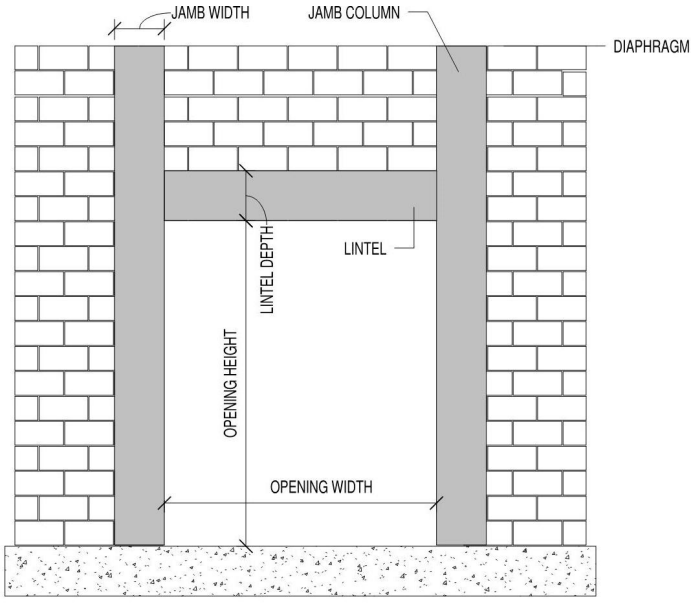
Analysis and Calculations

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:	17.6 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 - 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		

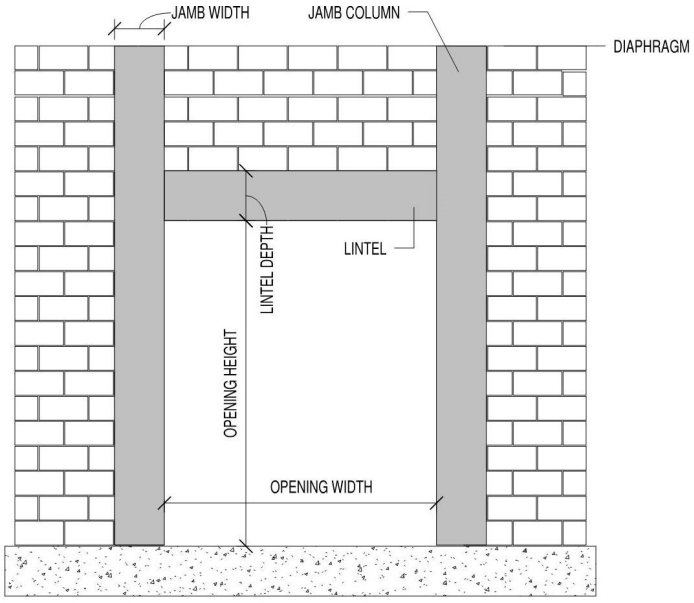
Analysis and Calculations

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:	I/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		

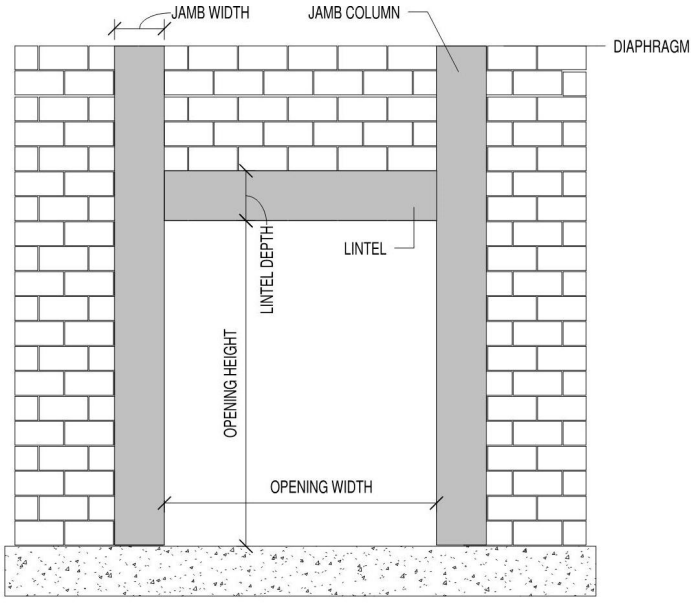
Analysis and Calculations

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		

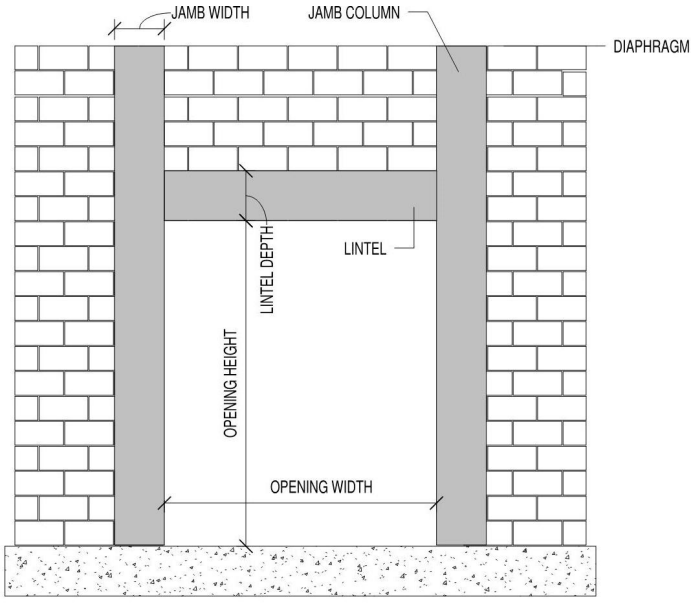
Analysis and Calculations

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		

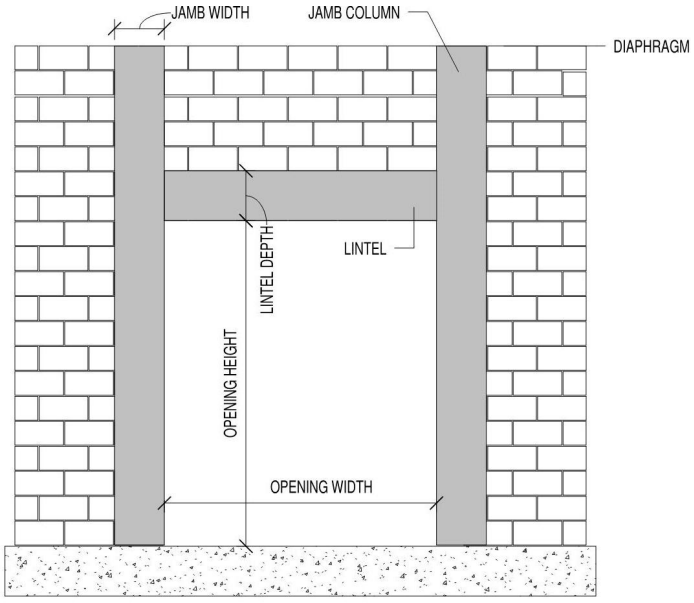
Analysis and Calculations

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		

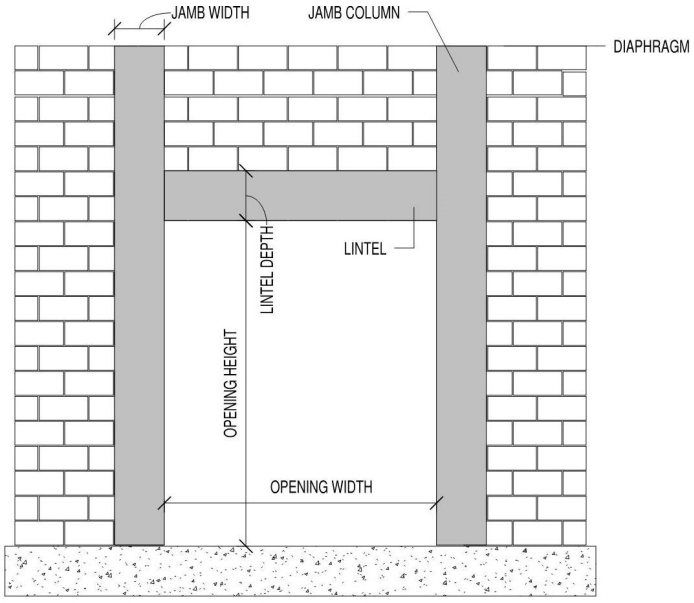
Analysis and Calculations

Masonry Lintel:

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



Masonry Column:

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

Masonry Lintel

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.

Masonry Column

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

Overall Status: **OK**

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

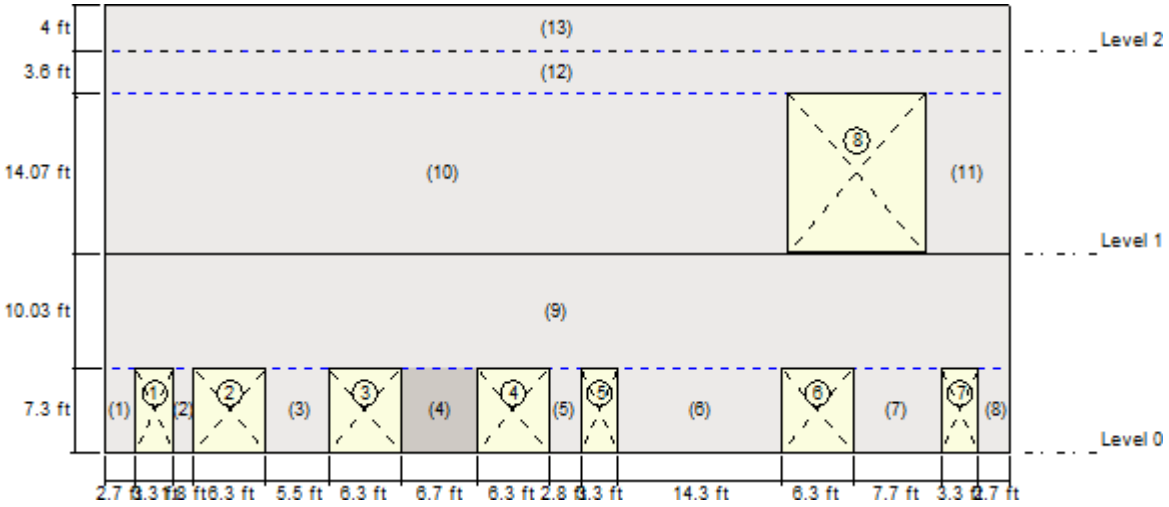
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.73	0.00
2	EQx	Horizontal	0.44	0.00

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
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	3-#5	24.00	34.07	2-#4	32.00	21.80
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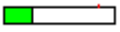
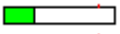
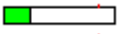
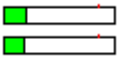
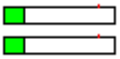
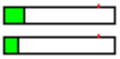
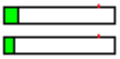
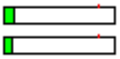
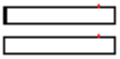
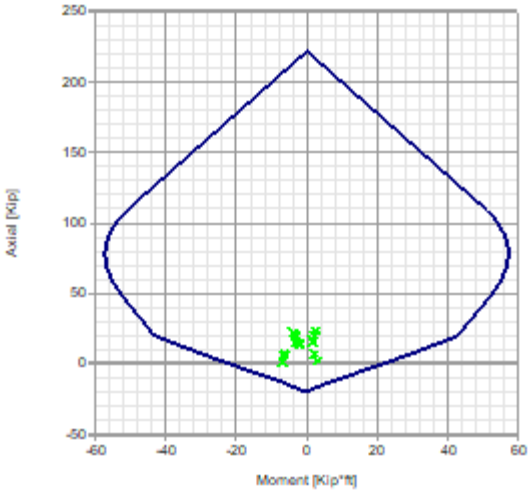
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	4-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	2-#5	24.00	34.07	2-#4	32.00	21.80
	7-#5	24.00	34.07	2-#4	32.00	21.80
	3-#5	24.00	34.07	2-#4	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

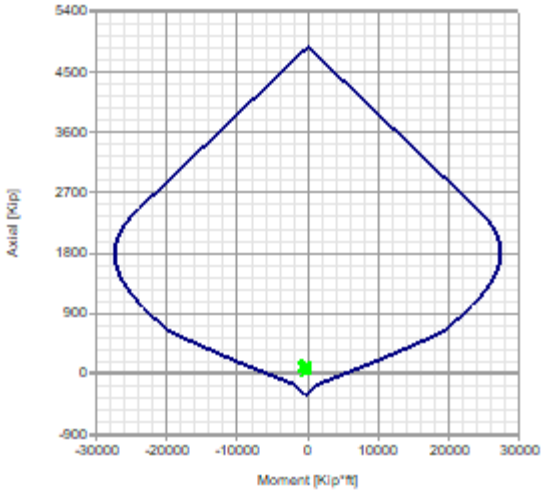
Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio	
1	D10(Max)	0.91	-6.92	22.99	0.30	
2	D10(Top)	4.53	2.41	4.70	0.51	
3	D10(Bottom)	19.94	-35.56	107.33	0.33	
4	D10(Bottom)	30.52	-48.86	169.15	0.29	
5	D10(Bottom)	13.54	-9.07	38.12	0.24	
6	D10(Bottom)	67.15	-158.93	715.97	0.22	
7	D10(Bottom)	41.15	-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

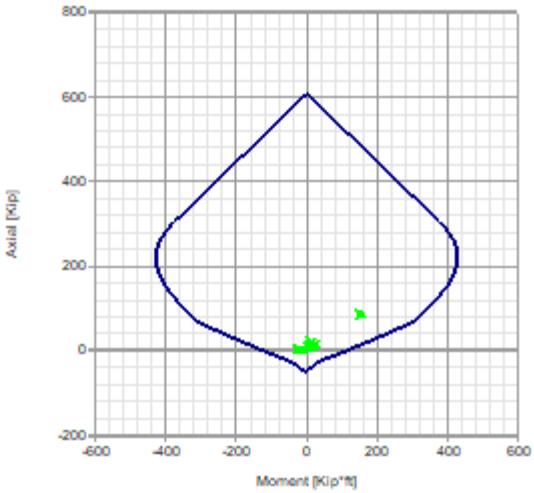
P vs. M (Segment 1)



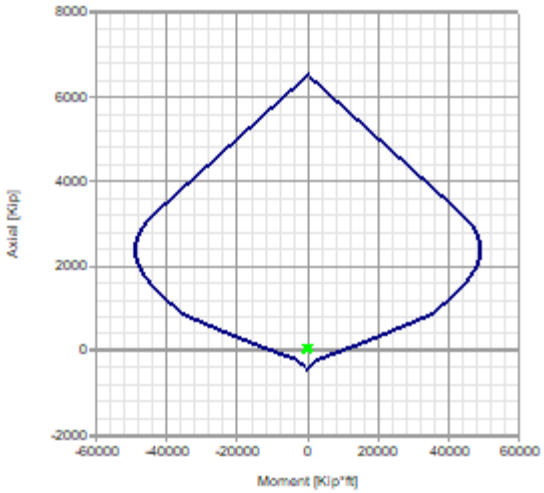
P vs. M (Segment 10)



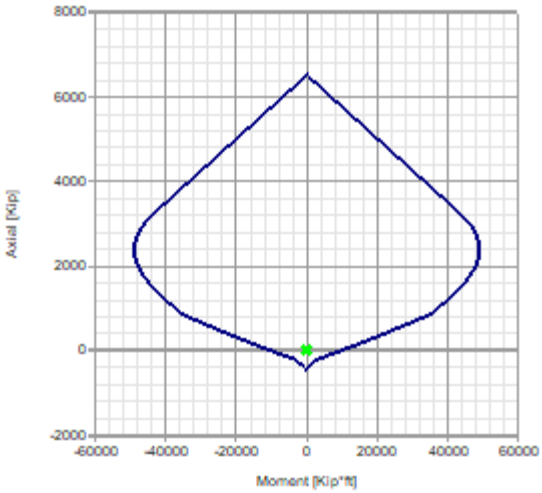
P vs. M (Segment 11)



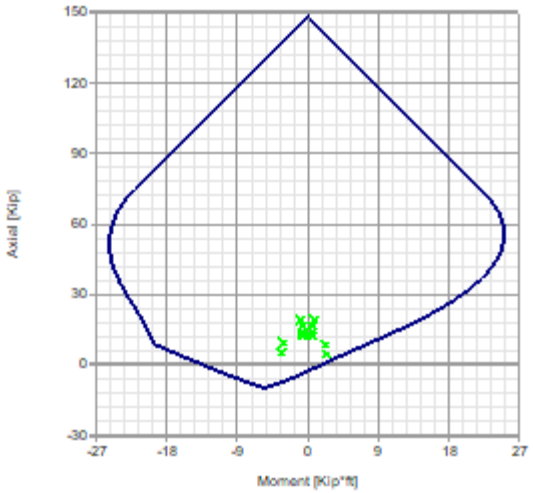
P vs. M (Segment 12)



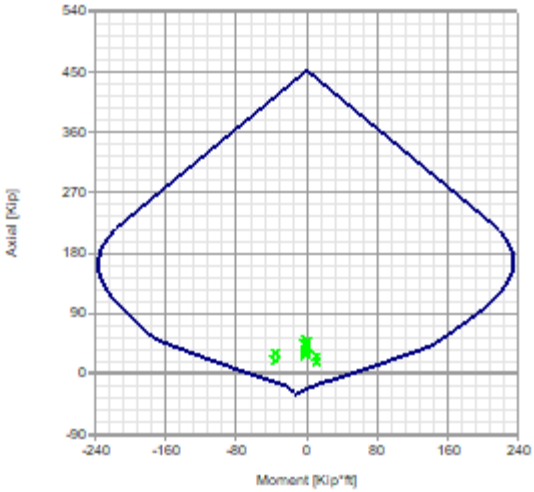
P vs. M (Segment 13)



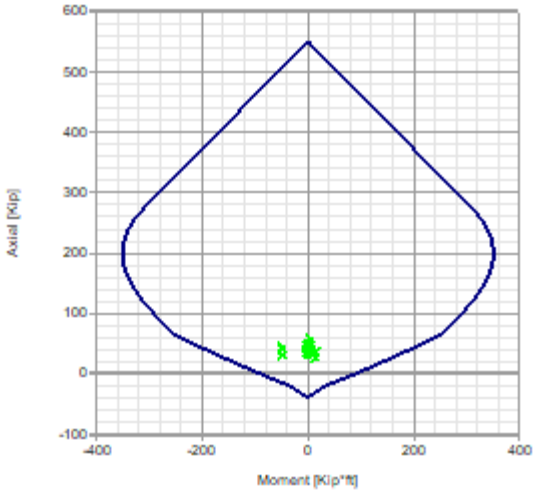
P vs. M (Segment 2)



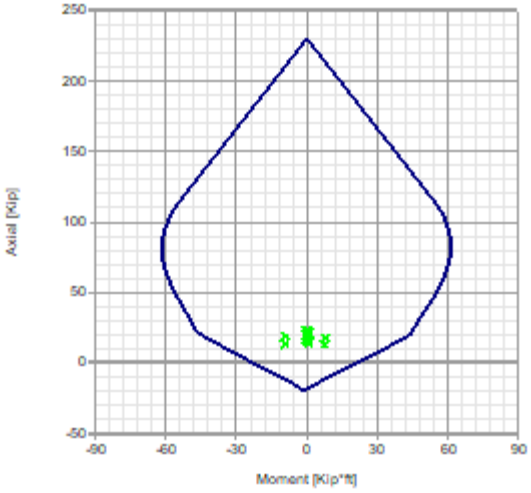
P vs. M (Segment 3)



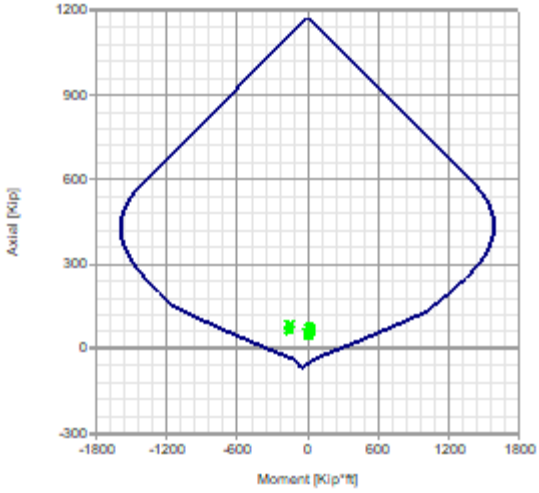
P vs. M (Segment 4)



P vs. M (Segment 5)

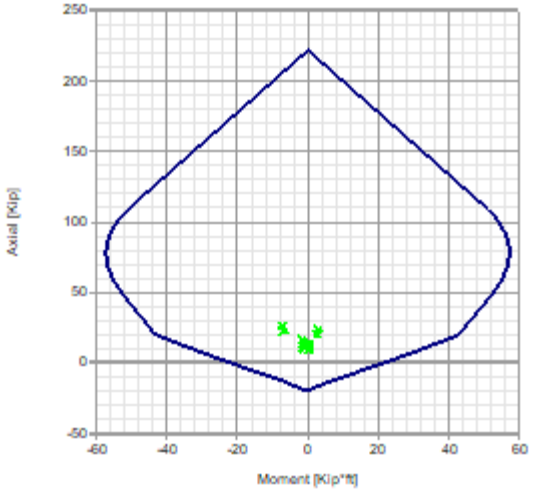


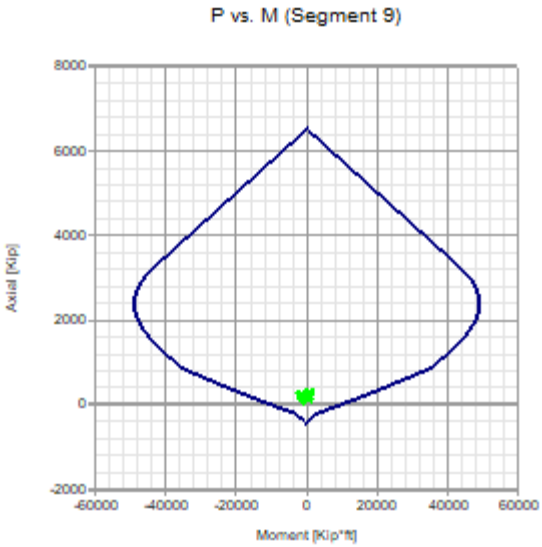
P vs. M (Segment 6)



P vs. M (Segment 7)

P vs. M (Segment 8)





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

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

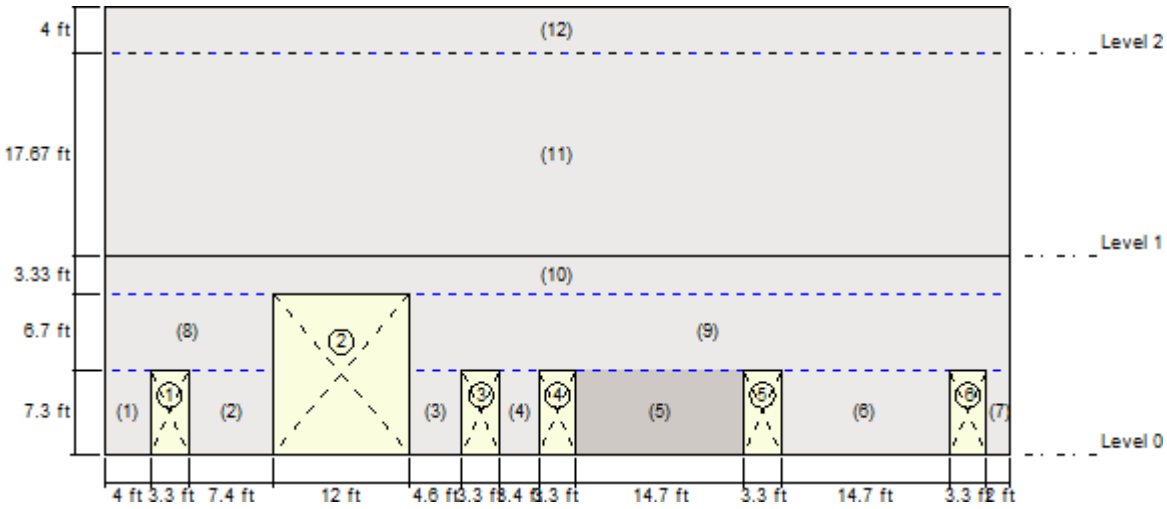
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.73	0.00
2	EQx	Horizontal	0.44	0.00

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

	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	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

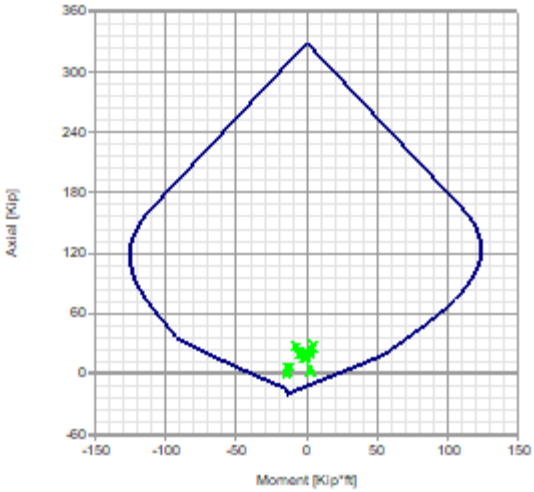
Combined axial flexure

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

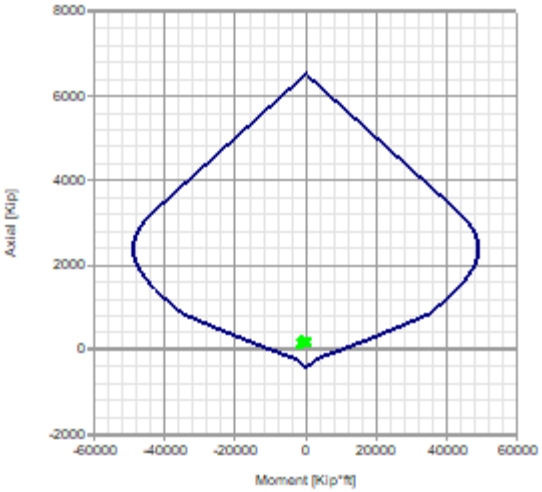
7	D8(Max)	21.89	2.99	18.01	0.17
8	D10(Bottom)	23.33	-66.90	506.50	0.13
9	D10(Bottom)	166.75	-951.23	8130.43	0.12
10	D10(Top)	142.36	-1064.39	14326.80	0.07
11	D10(Bottom)	119.41	-786.65	13618.07	0.06
12	D10(Bottom)	18.57	-14.85	10459.61	0.00

Interaction diagrams, P vs. M

P vs. M (Segment 1)

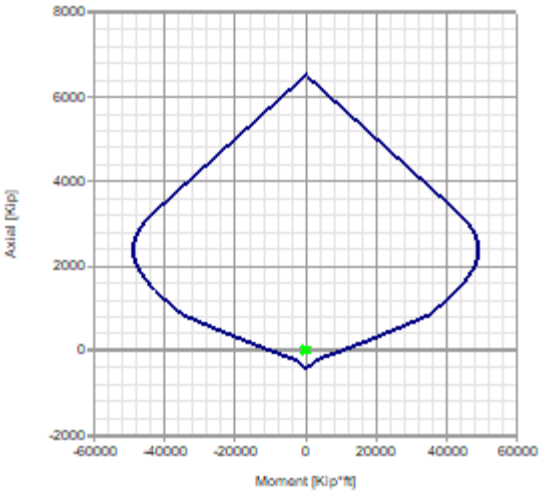


P vs. M (Segment 10)

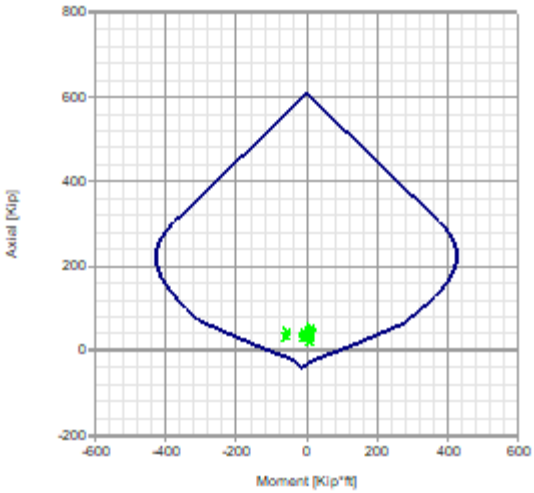


P vs. M (Segment 11)

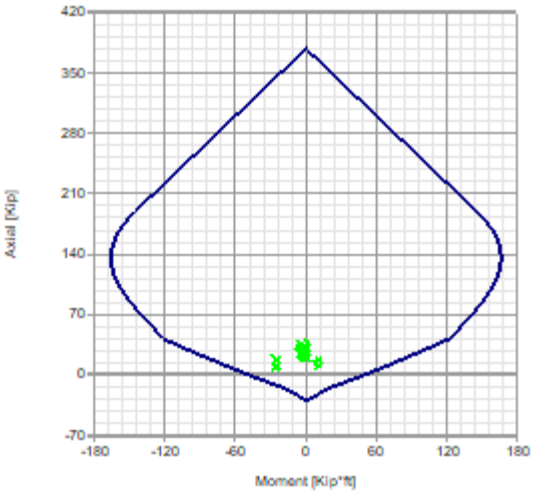
P vs. M (Segment 12)



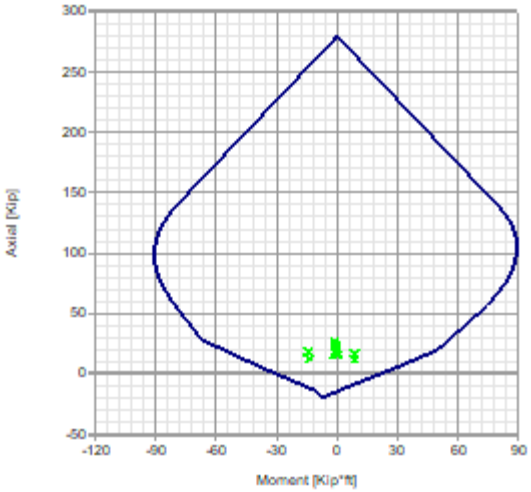
P vs. M (Segment 2)



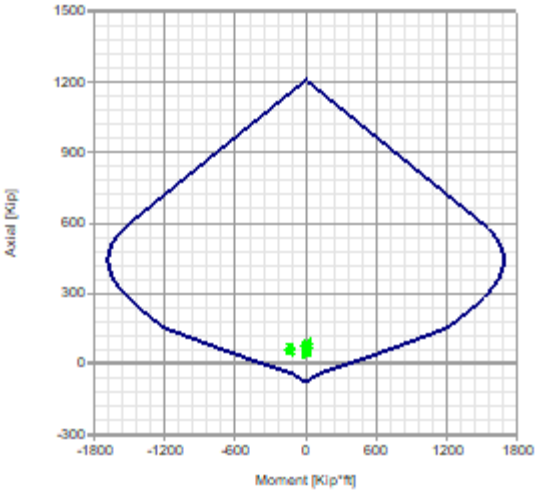
P vs. M (Segment 3)



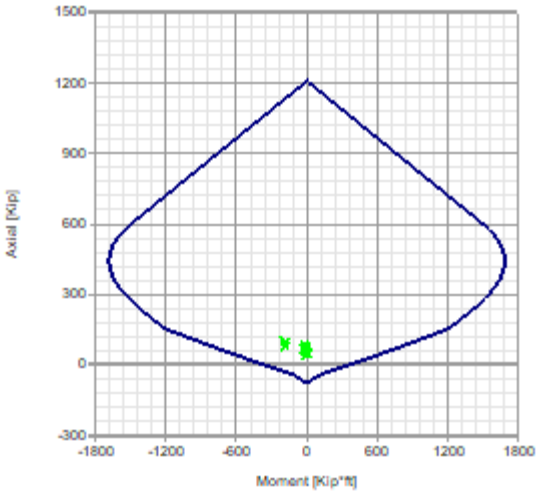
P vs. M (Segment 4)



P vs. M (Segment 5)

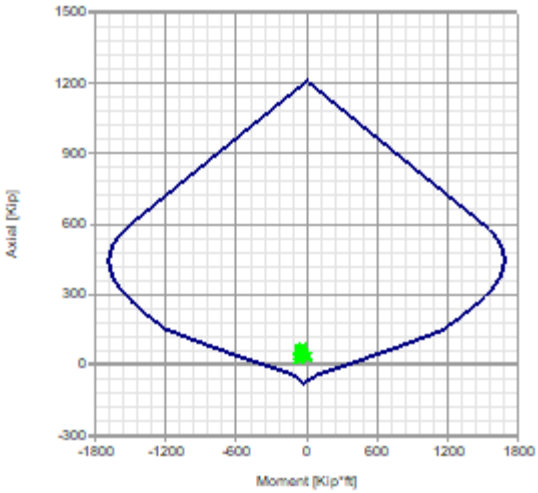


P vs. M (Segment 6)

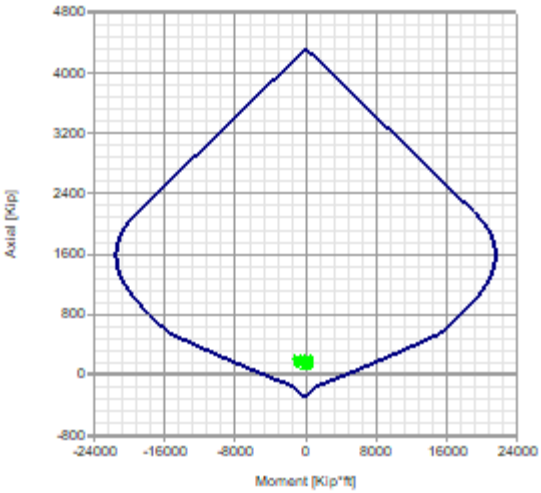


P vs. M (Segment 7)

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

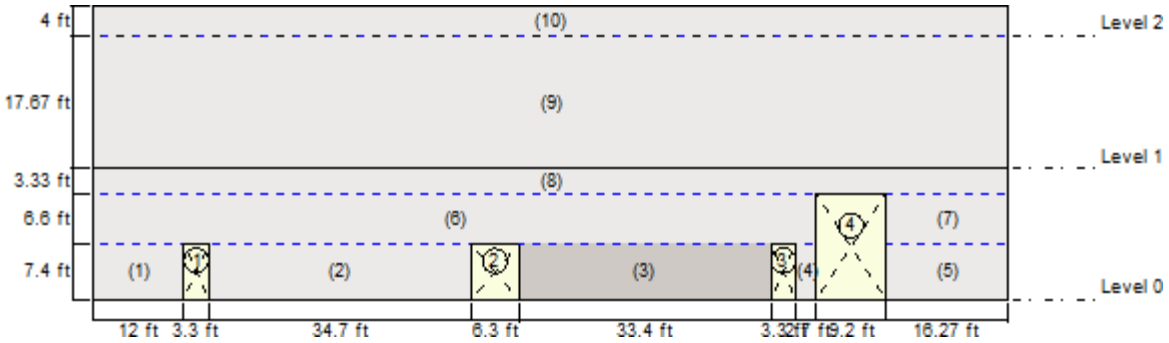
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.43	0.00
2	EQx	Horizontal	0.24	0.00

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

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	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

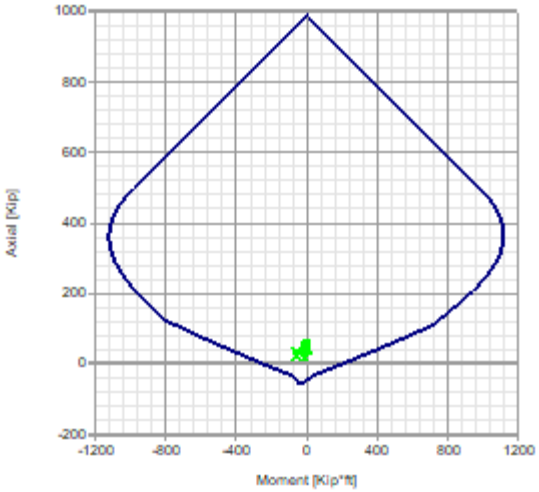
9	6-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	18-#5	24.00	34.07	7-#4	32.00	21.80
	3-#5	24.00	34.07	7-#4	32.00	21.80
	17-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	5-#5	24.00	34.07	7-#4	32.00	21.80
	8-#5	24.00	34.07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

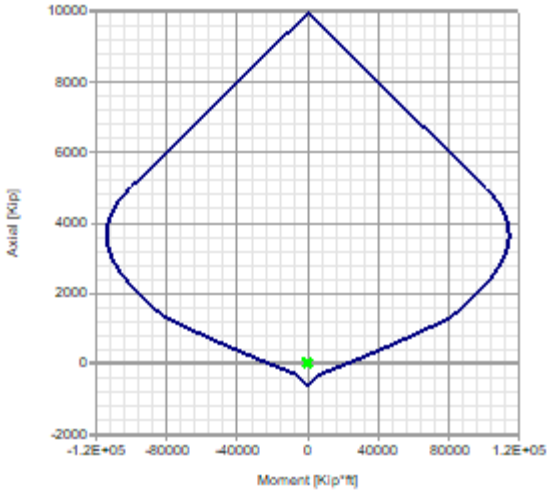
Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D10(Max)	21.19	-57.35	350.22	0.16
2	D10(Bottom)	106.39	-219.70	3287.47	0.07
3	D10(Bottom)	123.72	-163.40	3321.73	0.05
4	D10(Bottom)	15.05	-2.20	38.13	0.06
5	D10(Bottom)	70.41	-130.80	881.54	0.15
6	D10(Max)	216.54	-1385.34	21822.28	0.06
7	D8(Top)	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

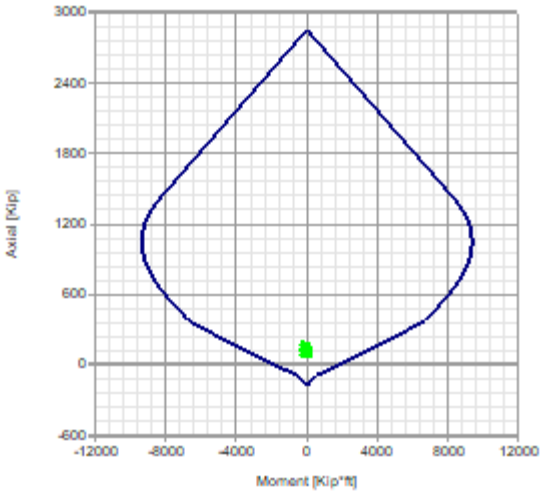
P vs. M (Segment 1)



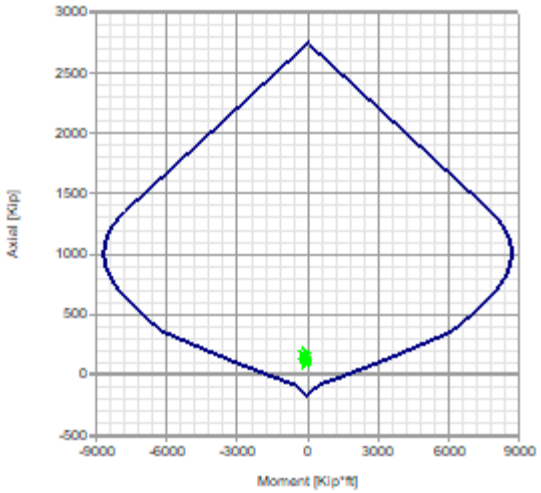
P vs. M (Segment 10)



P vs. M (Segment 2)

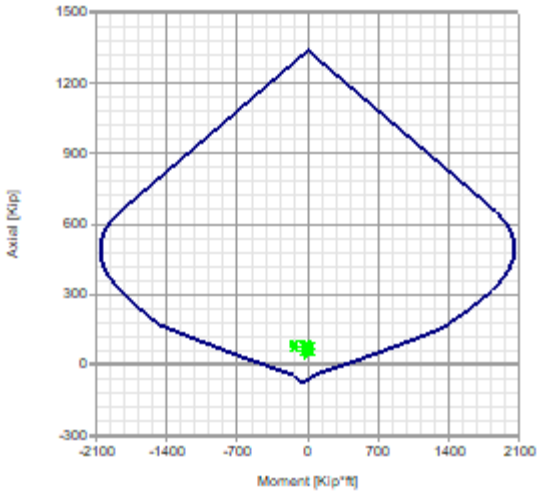


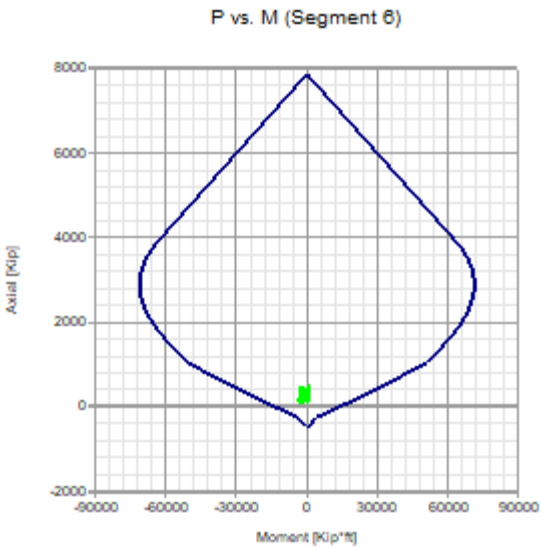
P vs. M (Segment 3)

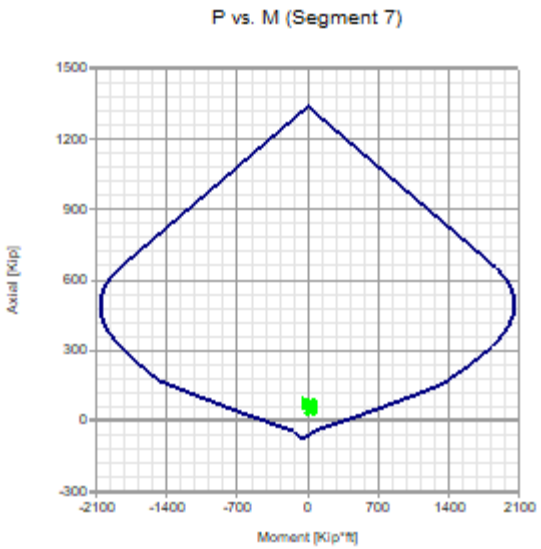


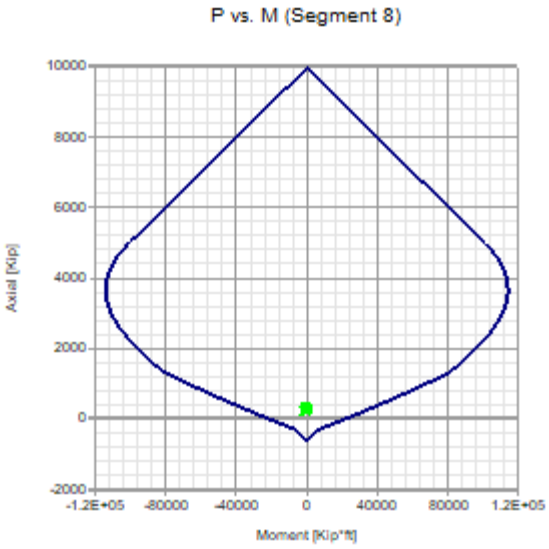
P vs. M (Segment 4)

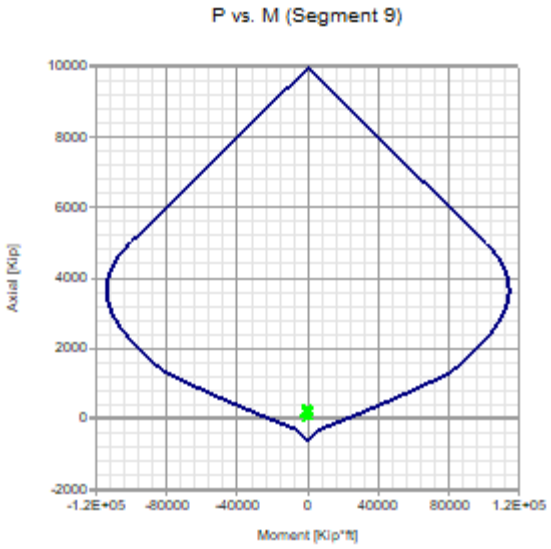
P vs. M (Segment 5)











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

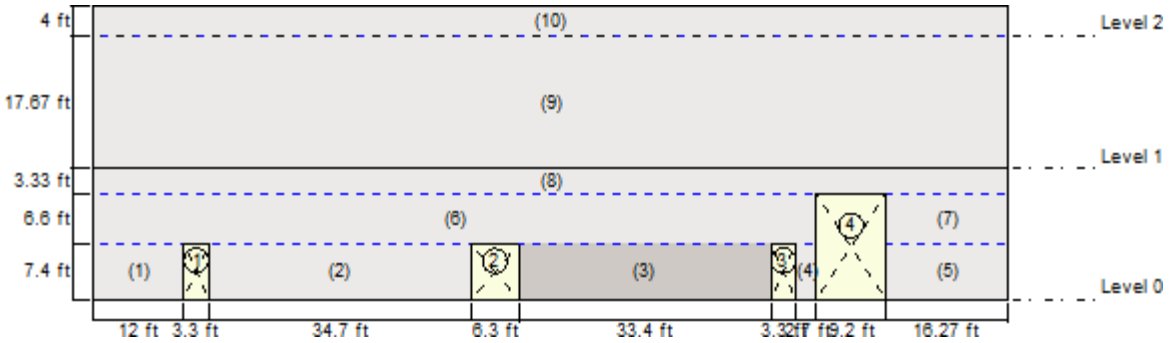
Story	Condition	Direction	Magnitude [Kip/ft]	Eccentricity [ft]
1	EQx	Horizontal	0.43	0.00
2	EQx	Horizontal	0.24	0.00

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

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	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

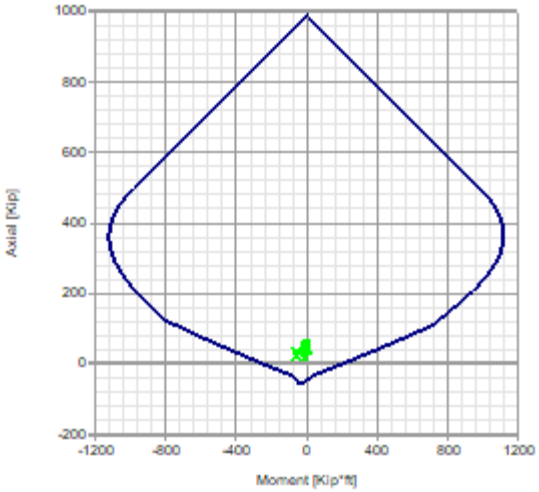
9	6-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	18-#5	24.00	34.07	7-#4	32.00	21.80
	3-#5	24.00	34.07	7-#4	32.00	21.80
	17-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	2-#5	24.00	34.07	7-#4	32.00	21.80
	5-#5	24.00	34.07	7-#4	32.00	21.80
	8-#5	24.00	34.07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

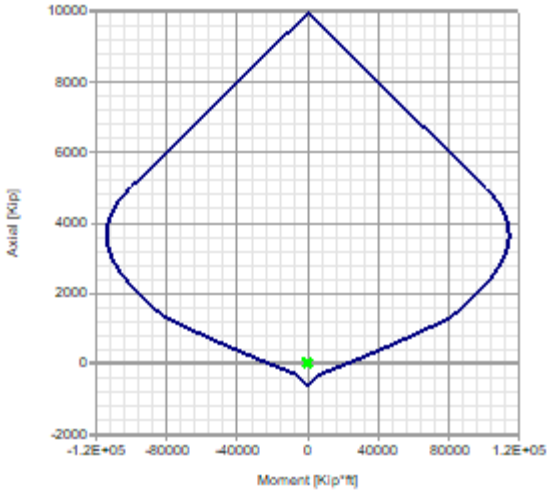
Segment	Condition	P [Kip]	M [Kip*ft]	Ma [Kip*ft]	Ratio
1	D10(Max)	21.19	-57.35	350.22	0.16
2	D10(Bottom)	106.39	-219.70	3287.47	0.07
3	D10(Bottom)	123.72	-163.40	3321.73	0.05
4	D10(Bottom)	15.05	-2.20	38.13	0.06
5	D10(Bottom)	70.41	-130.80	881.54	0.15
6	D10(Max)	216.54	-1385.34	21822.28	0.06
7	D8(Top)	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

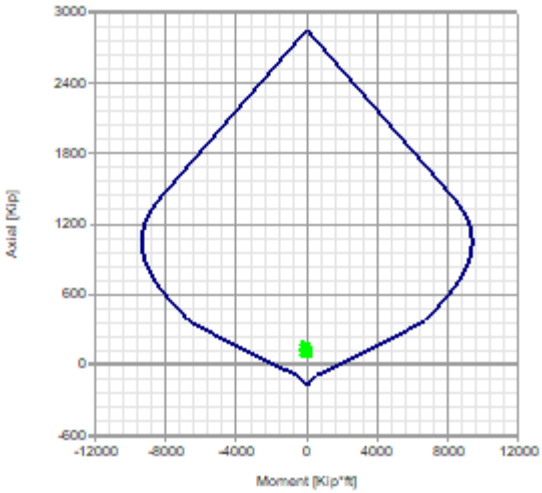
P vs. M (Segment 1)



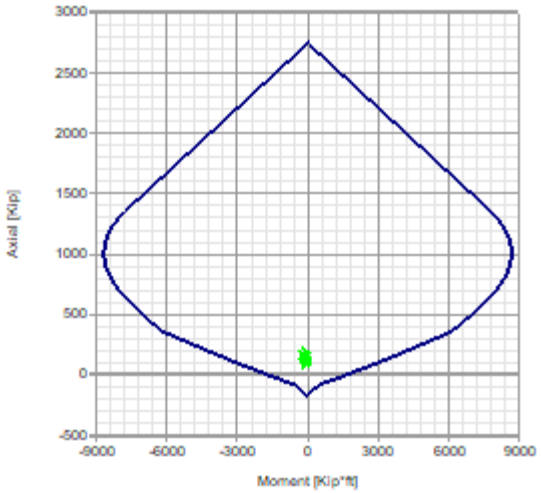
P vs. M (Segment 10)



P vs. M (Segment 2)

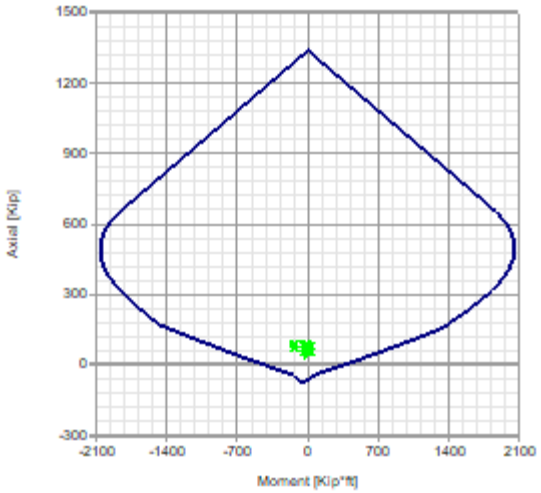


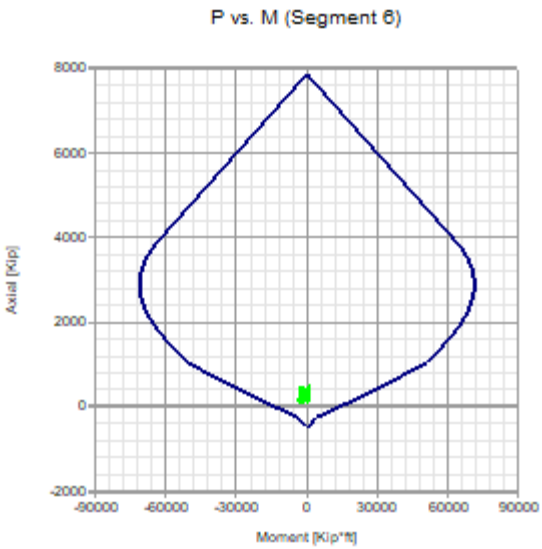
P vs. M (Segment 3)

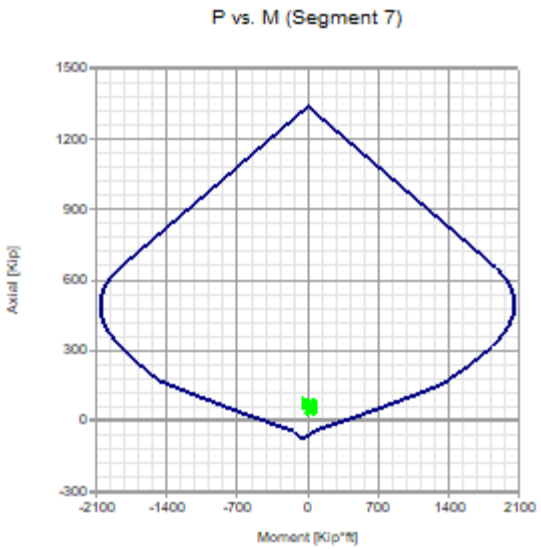


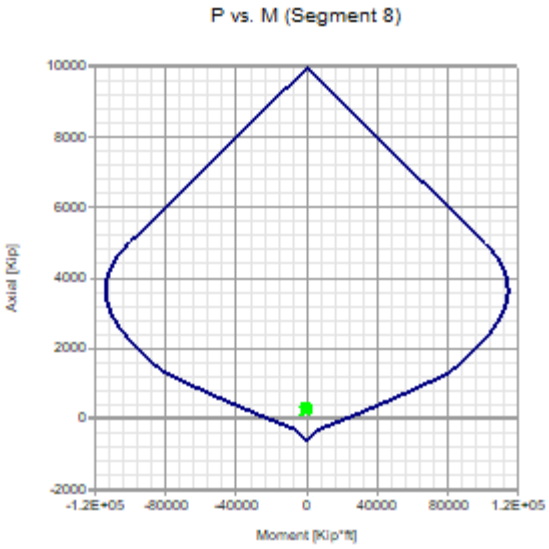
P vs. M (Segment 4)

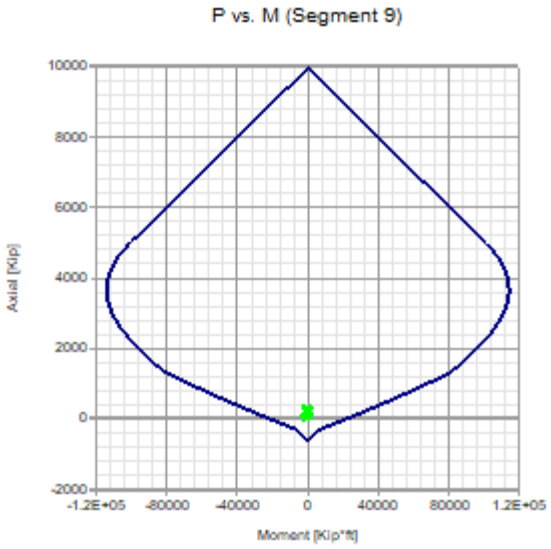
P vs. M (Segment 5)











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

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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)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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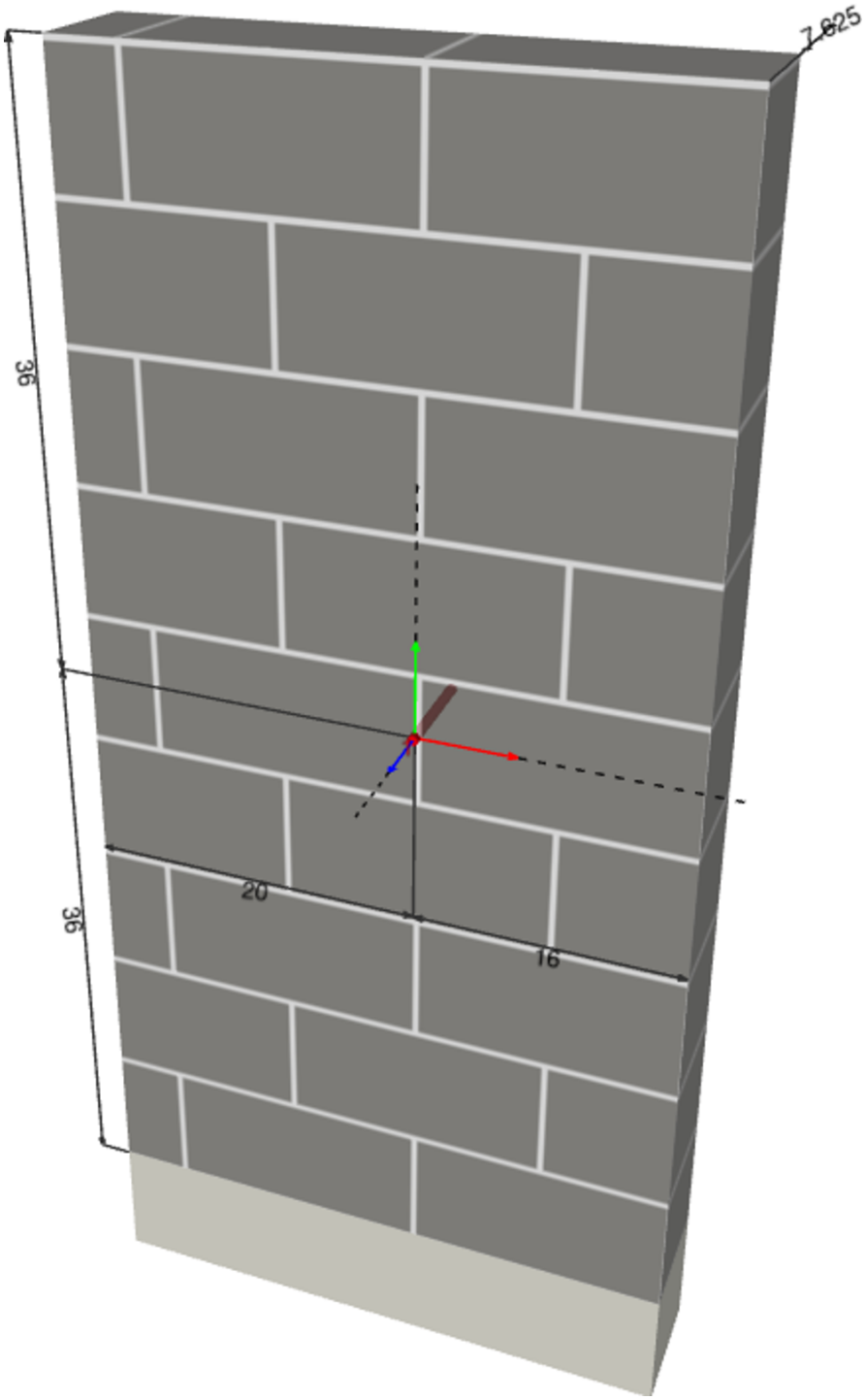
Geometry [in.]



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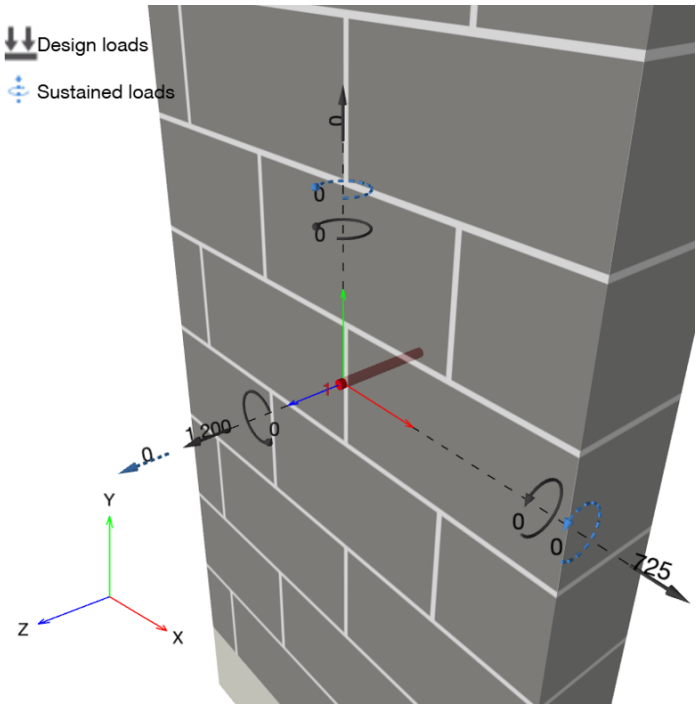
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 1,200; 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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4.3 Masonry crushing strength

$\phi V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot (f'_m \cdot A_{se,V})^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,V}$ [in.^{2}]
1,999	0.23

Results

V_{mc} [lb]	ϕ	ϕV_{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	ϕV_{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

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

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

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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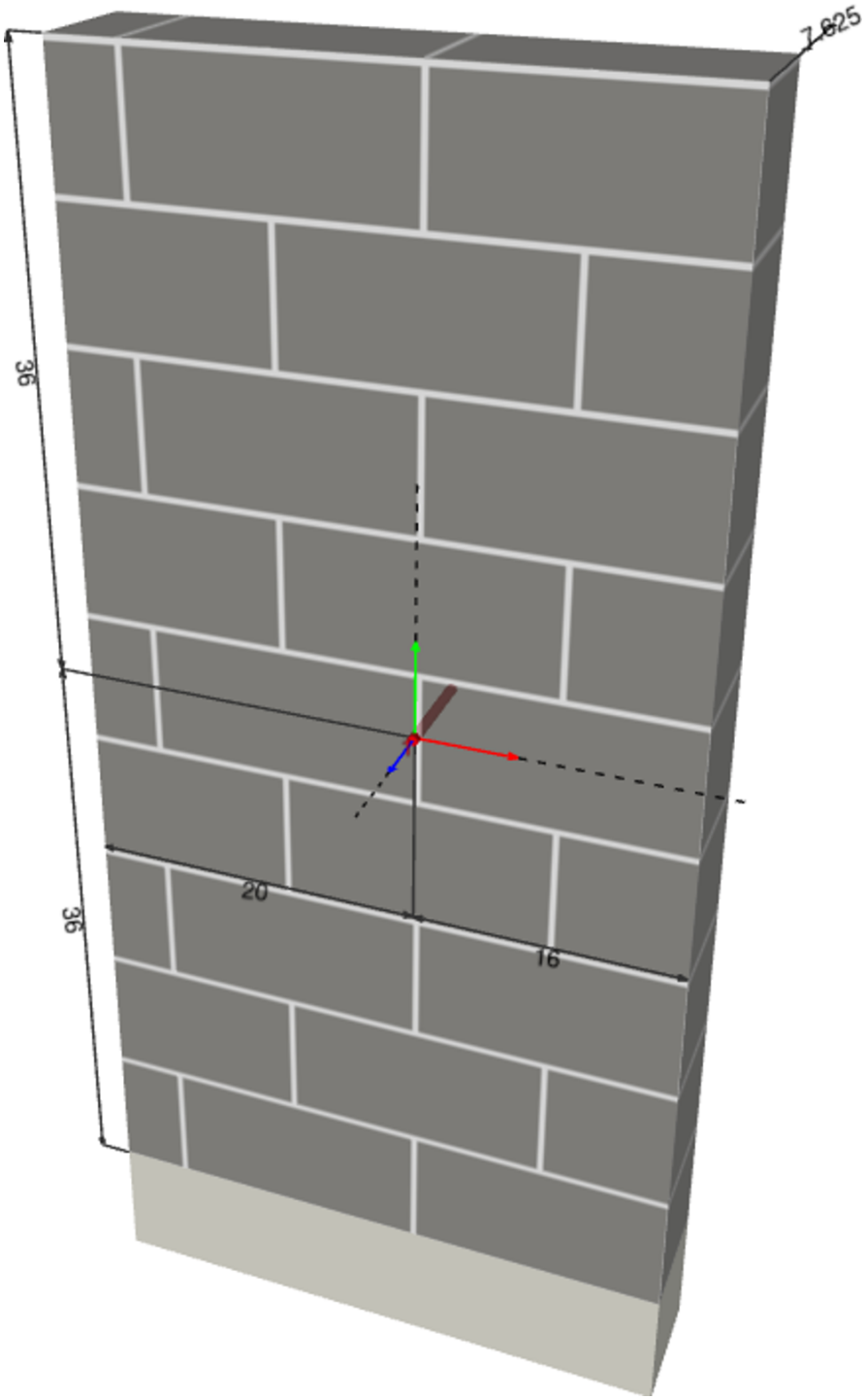
Design:	Creech DRP PH 2	Date:	10/16/2025
Fastening point:			

Geometry [in.]



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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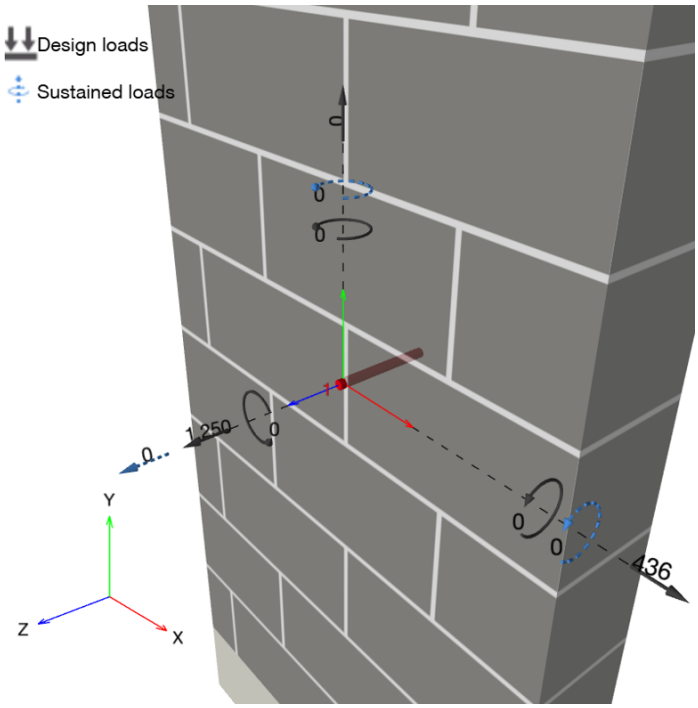
Geometry [in.] & Loading [lb, in.lb]



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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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4.3 Masonry crushing strength

$\phi V_{mc} \geq V_{ua}$ AC58 Table 3.2

$V_{mc} = 1750 \cdot (f'_m \cdot A_{se,V})^{\frac{1}{4}}$ AC58 Eq. 3-1

Variables

f'_m [psi]	$A_{se,V}$ [in.^{2}]
1,999	0.23

Results

V_{mc} [lb]	ϕ	ϕV_{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	ϕV_{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

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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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

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Michael Baker
INTERNATIONAL

Foundation Calculations

FoundationReactions

FoundationReactions, First-order linear, Local SupportAxis System

Supports

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.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]			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.4/4.1	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 2
Footing analysis 2
Footing design 5
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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

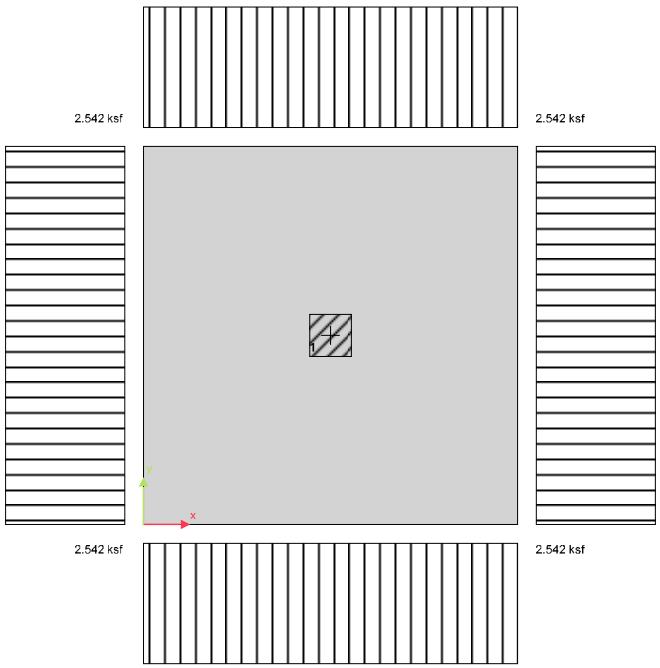
Width of footing; L_y = **6** ft

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



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; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **91.5** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **274.5** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.652** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.652** ksf

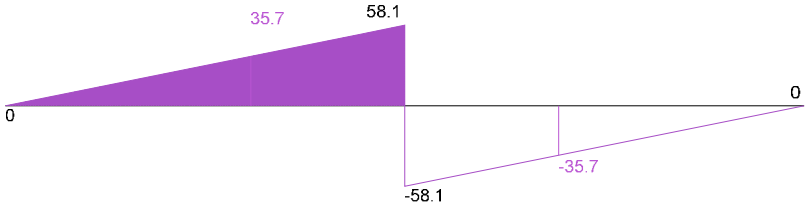
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.652** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.652** ksf

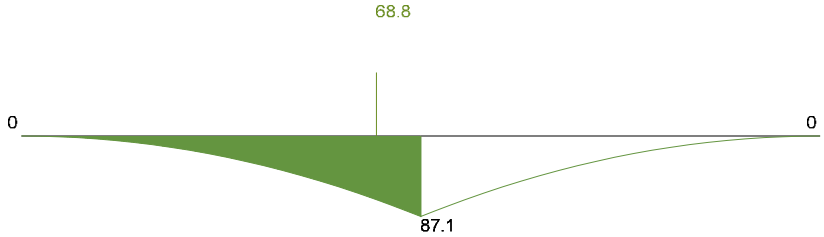
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.652** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.652** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} =$ **68.837** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} =$ **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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.813** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03619**

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

$M_{u.x.max} / \phi M_n =$ **0.483**

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**

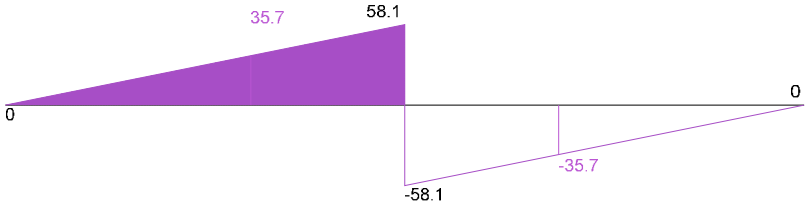
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **65.311** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **48.983** kips

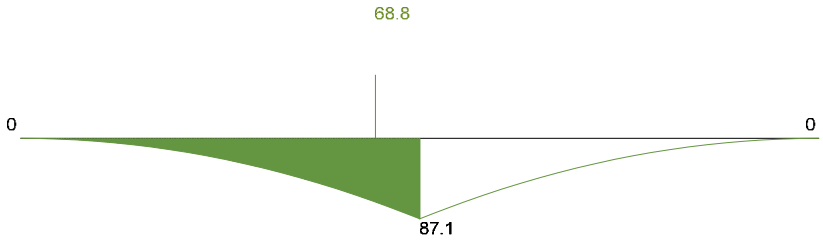
$V_{u.x} / \phi V_n$ = **0.729**

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

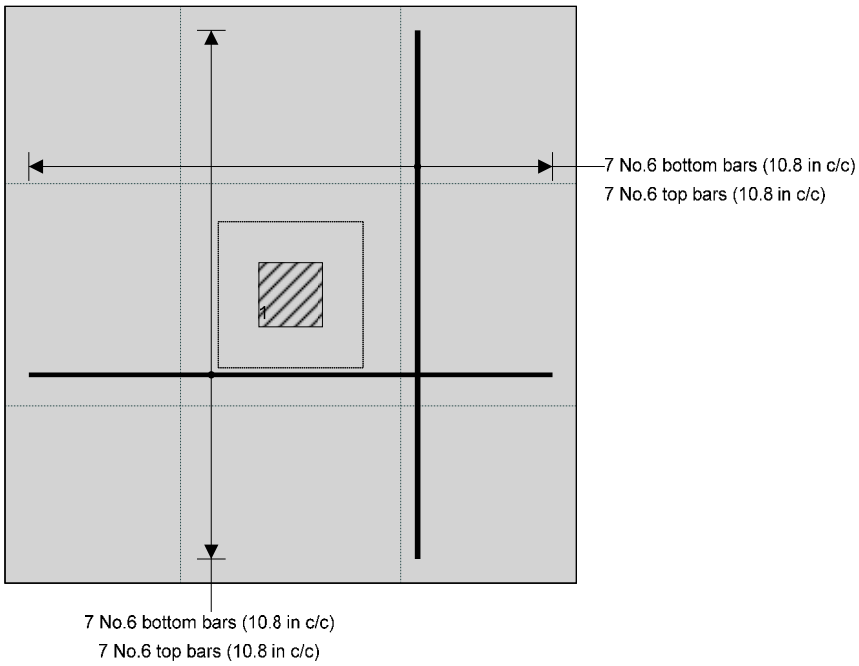
Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **146.908** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n$ = **132.217** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.521**
	PASS - Design moment capacity exceeds ultimate moment load

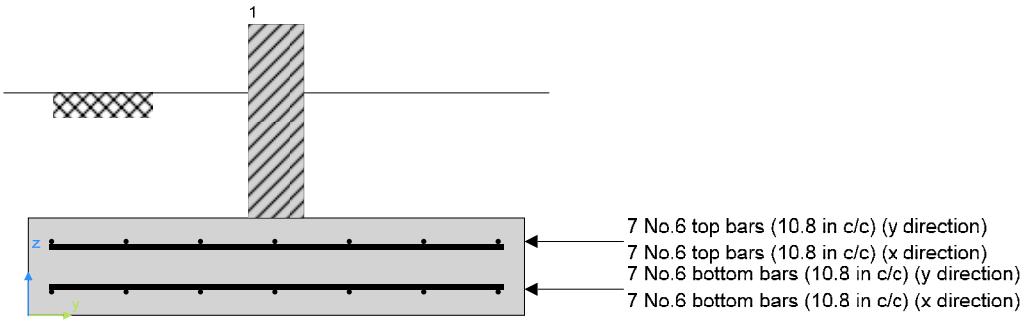
One-way shear design, y direction

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);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v)$ = **62.2** kips
Design shear capacity;	$\phi V_n = \phi_v \times V_n$ = **46.65** kips
	$V_{u.y} / \phi V_n$ = **0.765**
	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;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi})$ = **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);	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **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;	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)
1
7 No.6 bottom bars (10.8 in c/c)
7 No.6 top bars (10.8 in c/c)


1
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)
z
y

;

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

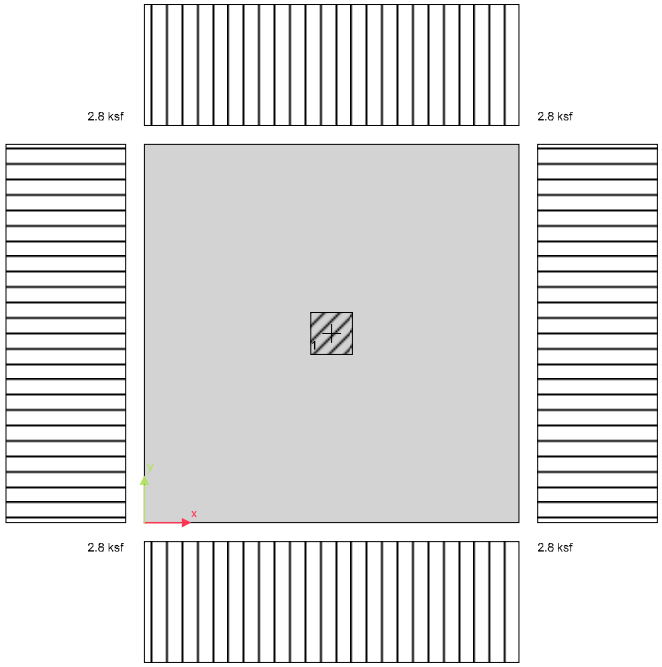
Width of footing; L_y = **6** ft

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

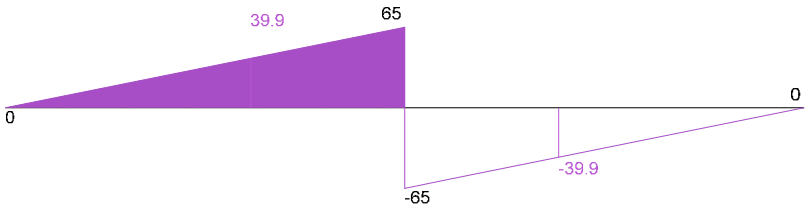
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **4.036** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **4.036** ksf

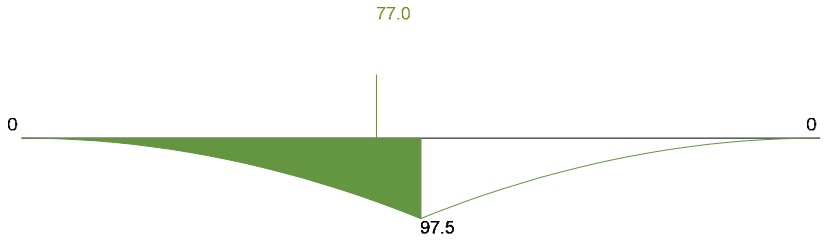
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **4.036** ksf

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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.813** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03619**

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

$M_{u.x.max} / \phi M_n =$ **0.540**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x} =$ **39.941** kips

Depth to reinforcement; $d_v = \min(h - c_{nom_b} - \phi_{x.bot} / 2, h - c_{nom_t} - \phi_{x.top} / 2) =$ **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**

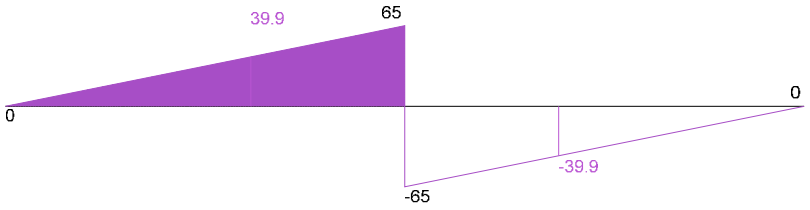
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v) =$ **65.311** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **48.983** kips

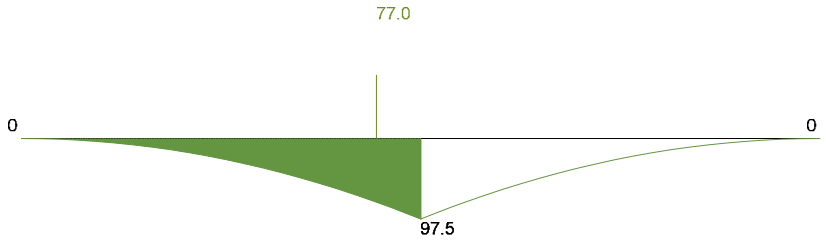
$V_{u.x} / \phi V_n =$ **0.815**

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; $M_{u.y.max} =$ **77.029** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (10.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} =$ **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

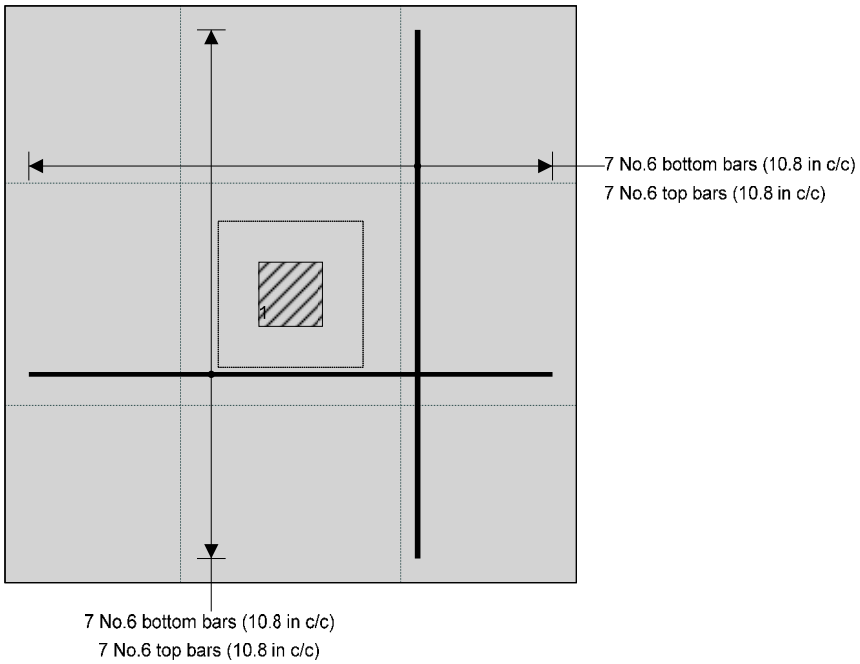
Depth to tension reinforcement;	$d = h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **9.875** in
Depth of compression block;	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.671** in
Neutral axis factor;	β_1 = **0.83**
Depth to neutral axis;	$c = a / \beta_1$ = **0.813** in
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
	$M_{u.y.max} / \phi M_n$ = **0.583**
	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);	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
	$V_{u.y} / \phi V_n$ = **0.856**
	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;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **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);	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**

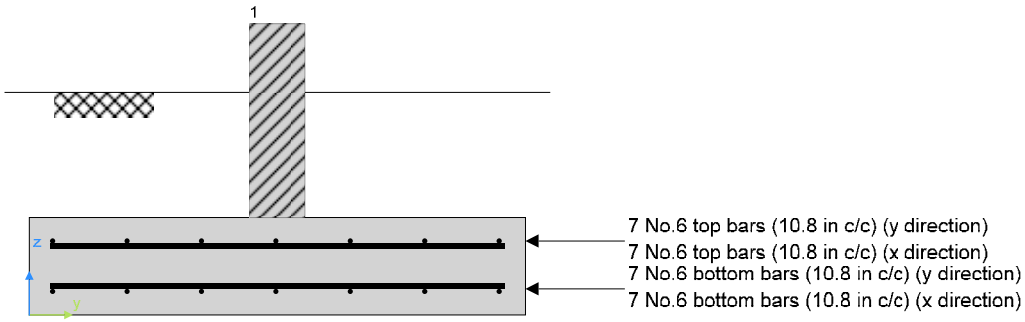
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **268.328** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.807**

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

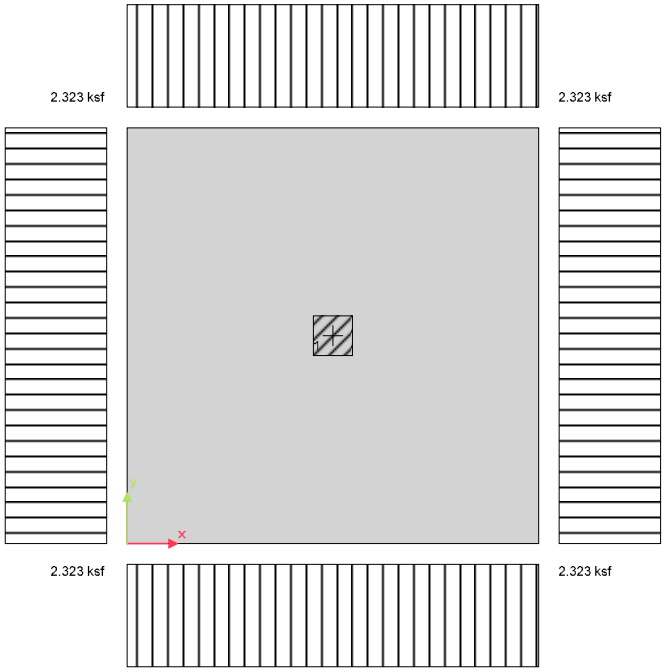
Width of footing; L_y = **7** ft

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; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) =$ **553.8** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) =$ **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

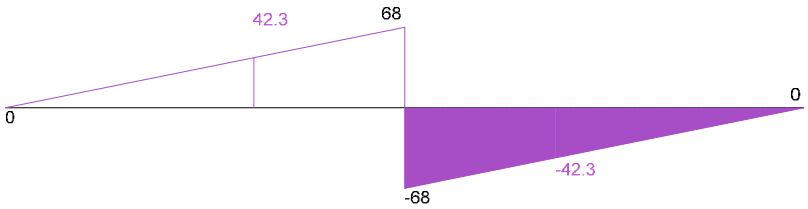
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.229** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.229** ksf

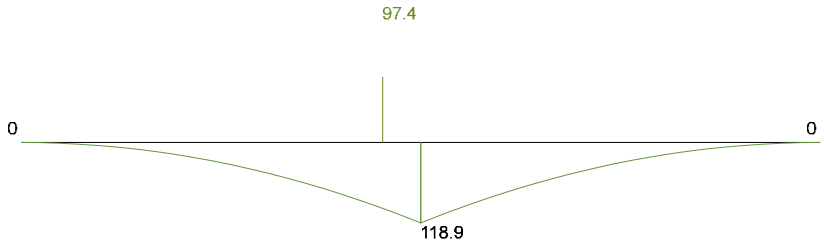
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.229** ksf

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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.05133**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **189.996** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **170.997** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.569**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x} =$ **42.271** kips

Depth to reinforcement; $d_v = \min(h - c_{nom_b} - \phi_{x.bot} / 2, h - c_{nom_t} - \phi_{x.top} / 2) =$ **12.625** in

Size effect factor (22.5.5.1.3); $\lambda_s =$ **1**

Ratio of longitudinal reinforcement; $\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v) =$ **0.0029**

Shear strength reduction factor; $\phi_v =$ **0.75**

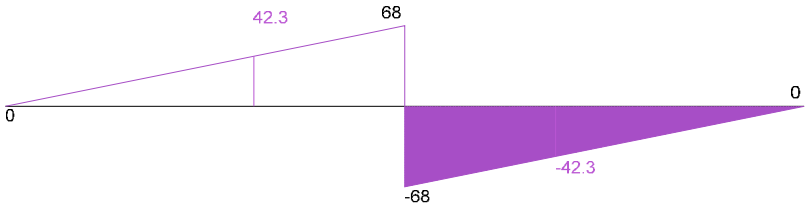
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v) =$ **81.2** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **60.9** kips

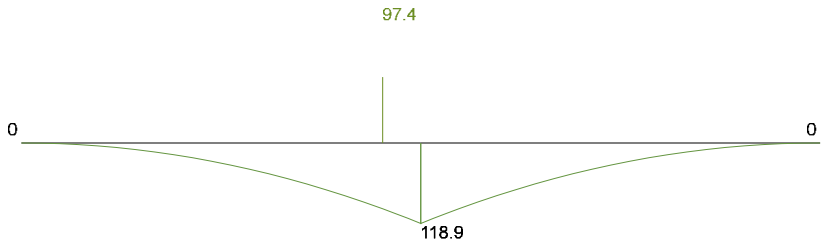
$V_{u.x} / \phi V_n =$ **0.694**

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; $M_{u.y.max} =$ **97.357** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} =$ **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

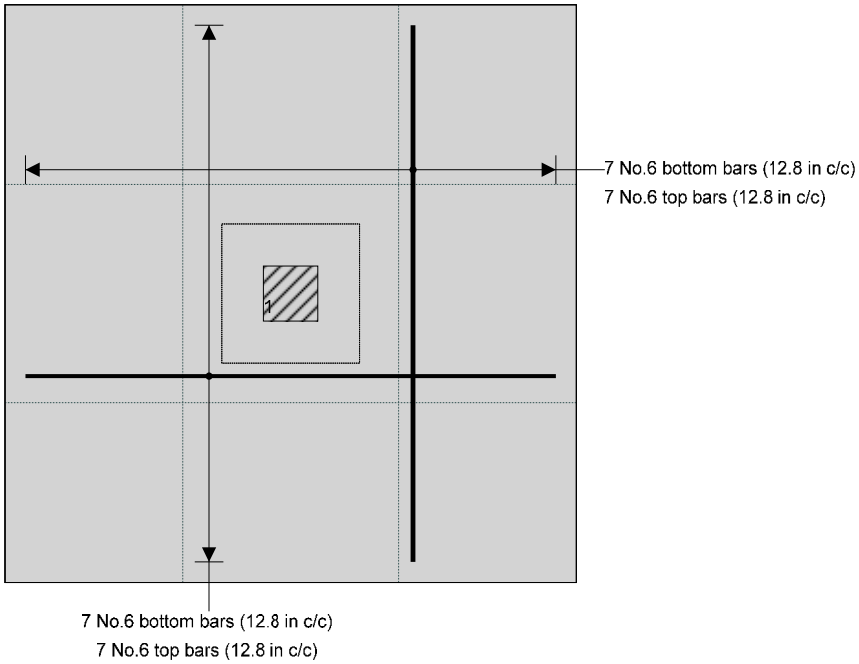
Depth to tension reinforcement;	$d = h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **11.875** in
Depth of compression block;	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.575** in
Neutral axis factor;	β_1 = **0.83**
Depth to neutral axis;	$c = a / \beta_1$ = **0.697** in
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
	$M_{u.y.max} / \phi M_n$ = **0.606**
	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);	λ_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;	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v)$ = **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);	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
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **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;	ϕ_v = **0.75**

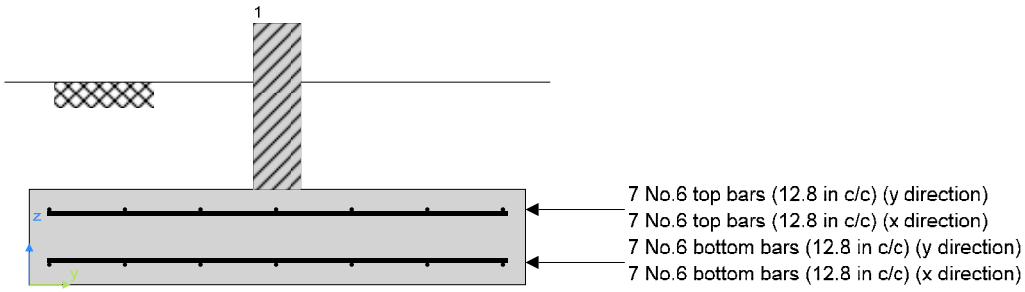
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **268.328** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **201.246** psi

$v_{ug} / \phi v_n =$ **0.641**

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

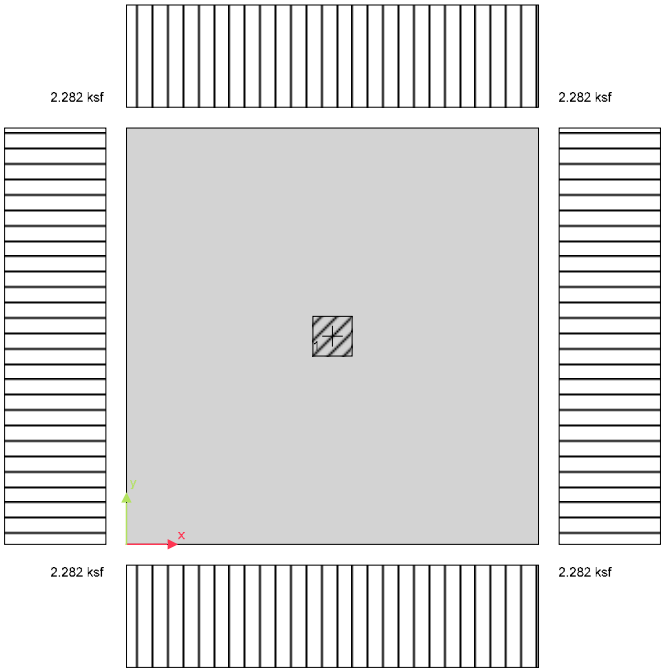
Width of footing; L_y = **7** ft

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; $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.136** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.136** ksf

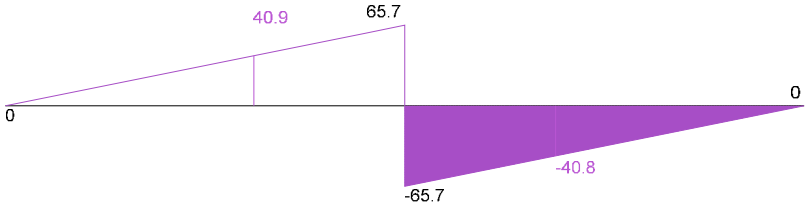
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.136** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.136** ksf

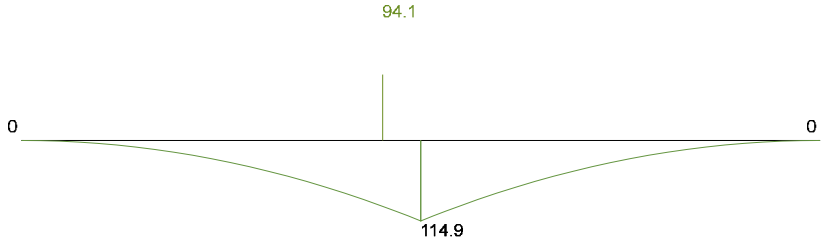
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.136** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.136** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} =$ **94.097** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} =$ **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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.05133**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **189.996** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **170.997** kip_ft

$M_{u.x.max} / \phi M_n =$ **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**

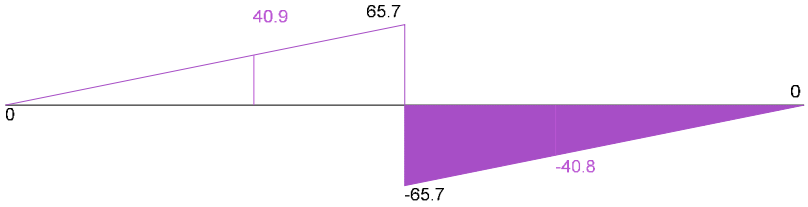
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **81.2** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **60.9** kips

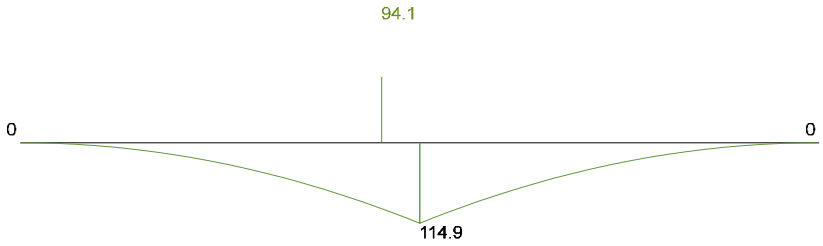
$V_{u.x} / \phi V_n$ = **0.671**

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); $s_{max} = \min(2 \times h, 18\ in)$ = **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

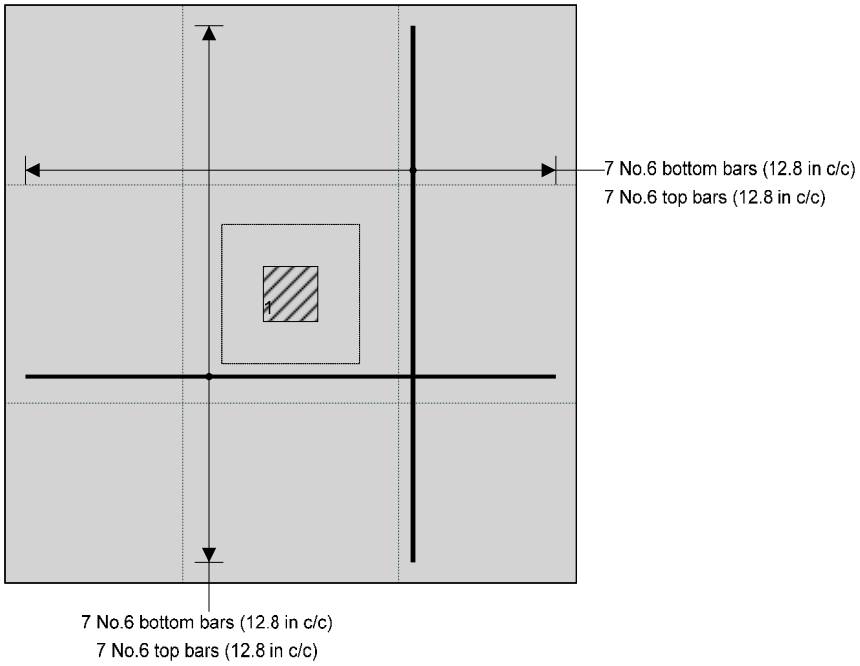
Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **178.446** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n$ = **160.602** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.586**
	PASS - Design moment capacity exceeds ultimate moment load

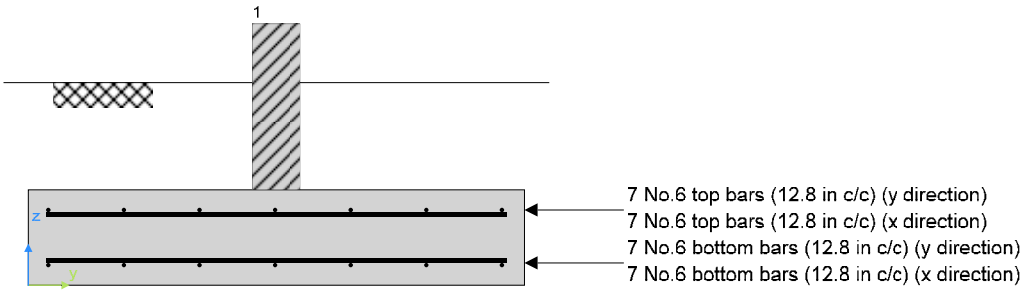
One-way shear design, y direction

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);	λ_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;	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v)$ = **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;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **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);	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
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **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;	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
	$v_{ug} / \phi v_n$ = **0.619**
	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)
1
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)
z
y

;

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

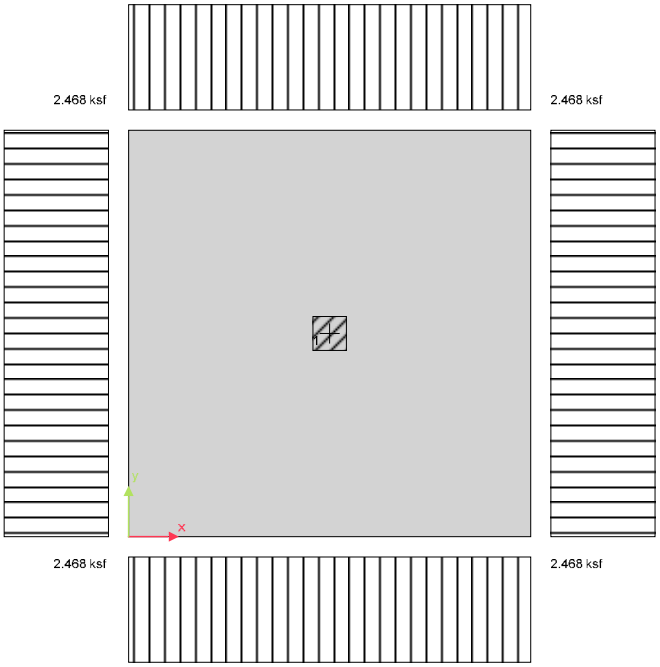
Width of footing; L_y = **8** ft

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; $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.349** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.349** ksf

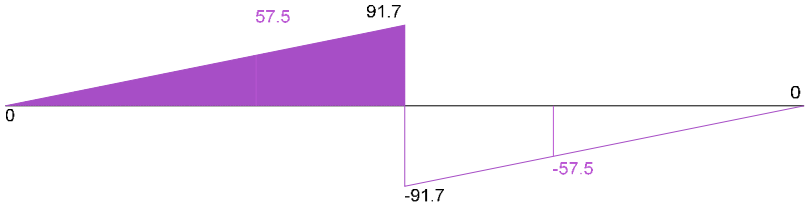
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.349** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.349** ksf

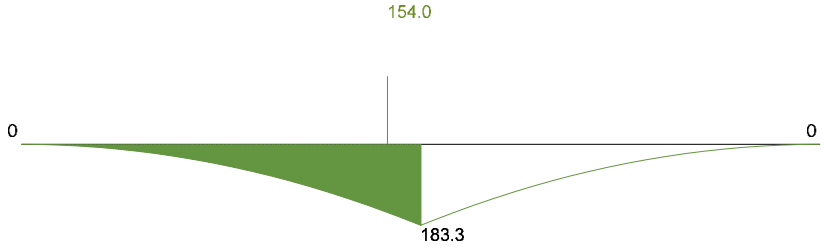
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.349** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.349** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} =$ **154.02** kip_ft

Tension reinforcement provided; 9 No.6 bottom bars (11.1 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} =$ **3.96** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h =$ **3.11** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom_b} - \phi_{x.bot} / 2 =$ **14.625** in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.647** in

Neutral axis factor; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.784** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.05294**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **283.169** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **254.852** kip_ft

$M_{u.x.max} / \phi M_n =$ **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**

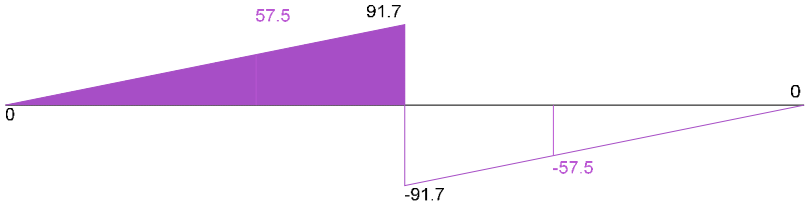
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **106.457** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **79.842** kips

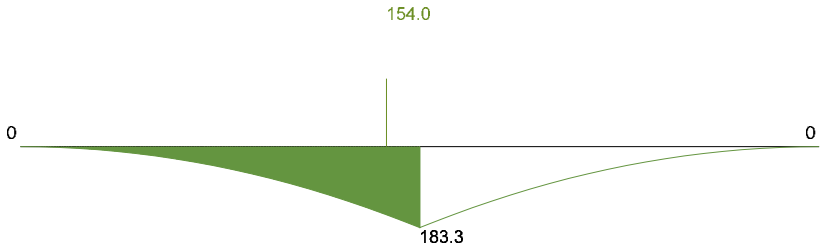
$V_{u.x} / \phi V_n$ = **0.720**

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

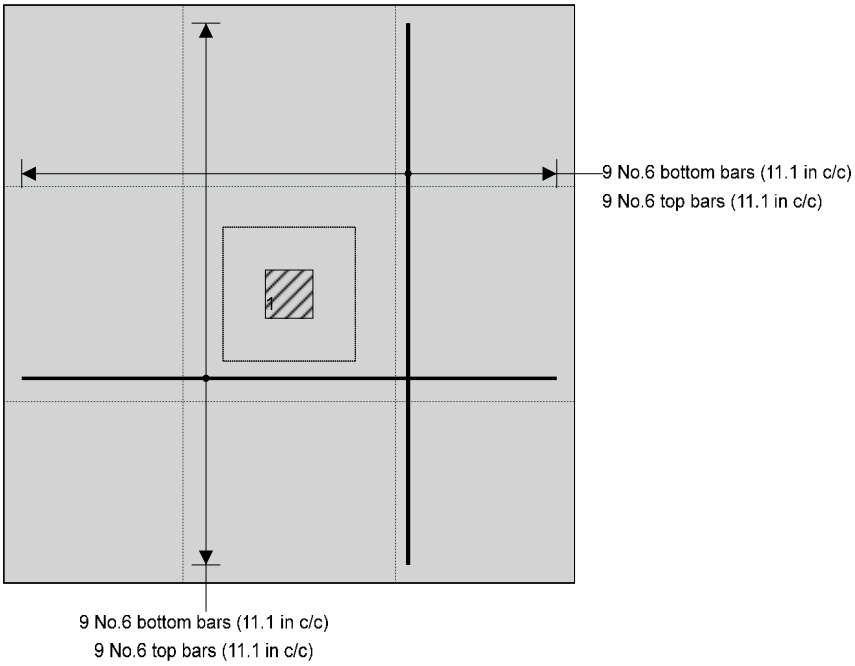
Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) =$ **268.319** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n =$ **241.487** kip_ft
	$M_{u.y.max} / \phi M_n =$ **0.638**
	PASS - Design moment capacity exceeds ultimate moment load

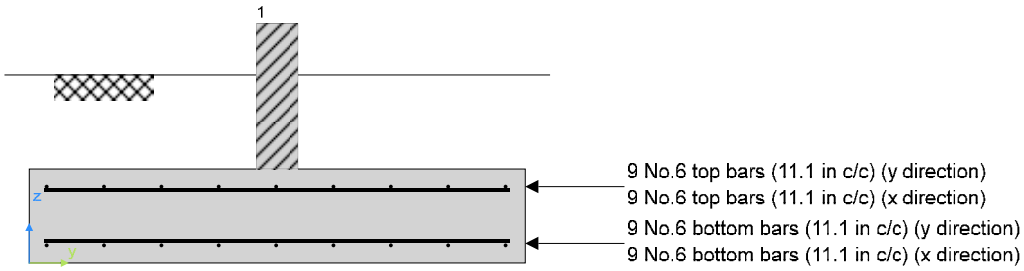
One-way shear design, y direction

Ultimate shear force;	$V_{u.y} =$ **57.516** kips
Depth to reinforcement;	$d_v = \min(h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom_t} - \phi_{y.top} / 2) =$ **13.875** in
Size effect factor (22.5.5.1.3);	$\lambda_s =$ **1**
Ratio of longitudinal reinforcement;	$\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00297**
Shear strength reduction factor;	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v) =$ **102.785** kips
Design shear capacity;	$\phi V_n = \phi_v \times V_n =$ **77.089** kips
	$V_{u.y} / \phi V_n =$ **0.746**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;	$d_{v2} =$ **14.25** in
Shear perimeter length (22.6.4);	$l_{xp} =$ **22.250** in
Shear perimeter width (22.6.4);	$l_{yp} =$ **22.250** in
Shear perimeter (22.6.4);	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **89.000** in
Shear area;	$A_p = l_{x,perim} \times l_{y,perim} =$ **495.062** in^2
Surcharge loaded area;	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **431.062** in^2
Ultimate bearing pressure at center of shear area;	$q_{up.avg} =$ **3.349** ksf
Ultimate shear load;	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **173.323** kips
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi) =$ **136.663** 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
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **563.791** 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
	$v_{ug} / \phi v_n =$ **0.679**
	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)
1
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)
z
y

;

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

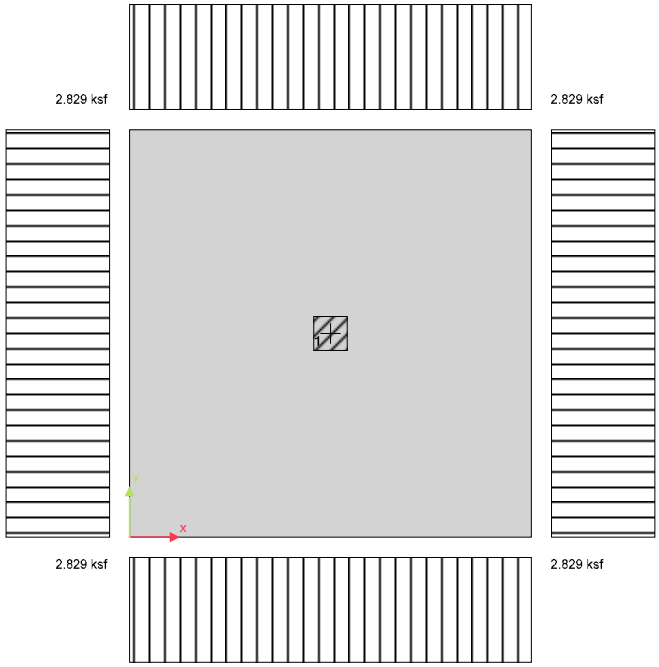
Width of footing; L_y = **10** ft

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

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.91** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.91** ksf

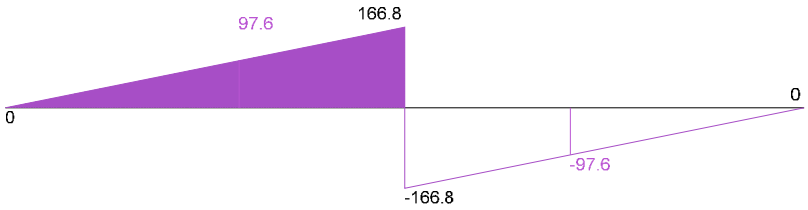
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.91** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.91** ksf

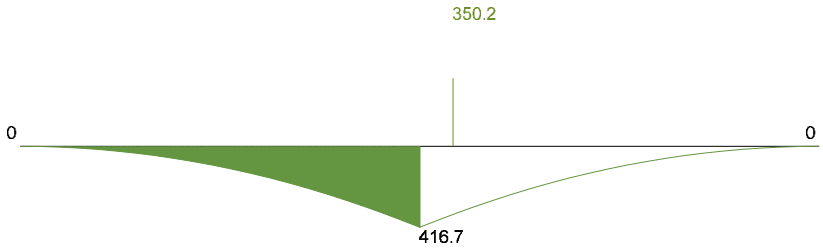
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.91** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.91** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} =$ **350.165** kip_ft

Tension reinforcement provided; 12 No.6 bottom bars (10.2 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} =$ **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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.837** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.07096**

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

$M_{u.x.max} / \phi M_n =$ **0.727**

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**

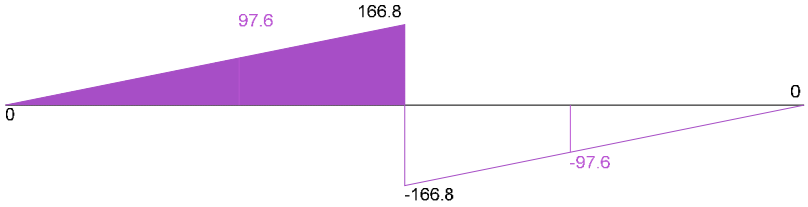
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **170.985** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **128.239** kips

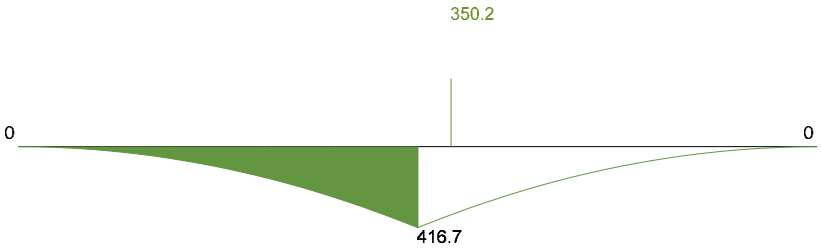
$V_{u.x} / \phi V_n$ = **0.761**

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

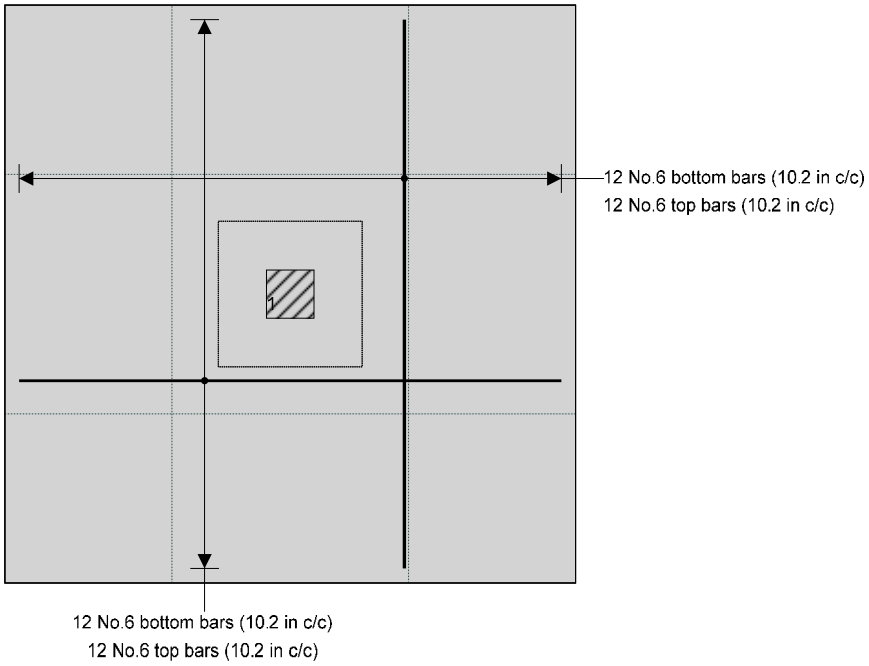
One-way shear design, y direction

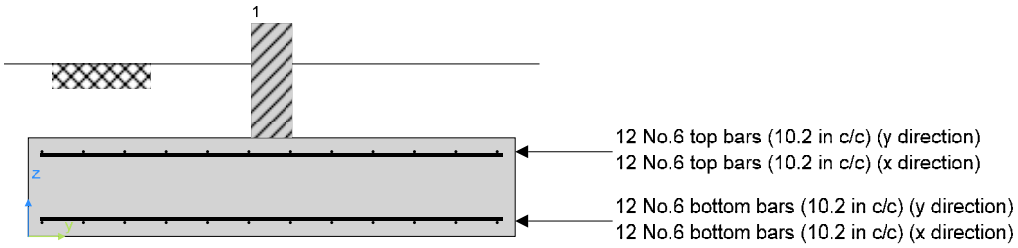
Ultimate shear force;	$V_{u.y}$ = **97.583** kips
Depth to reinforcement;	$d_v = \min(h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom_t} - \phi_{y.top} / 2)$ = **19.875** in
Size effect factor (22.5.5.1.3);	λ_s = **1**
Ratio of longitudinal reinforcement;	$\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v)$ = **0.00221**
Shear strength reduction factor;	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v)$ = **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;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **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);	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
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **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
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
	$v_{ug} / \phi v_n$ = **0.633**
	PASS - Design shear stress capacity exceeds ultimate shear stress load

Tekla Tedds	Project				Job Ref.	
Michael Baker Internation 7090 S Union Park Ave Midvale, UT 84107	Creech DRP PH 2					
	Section				Sheet no./rev.	
	Column C-4.9				49	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	ACV	10/16/2025				





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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

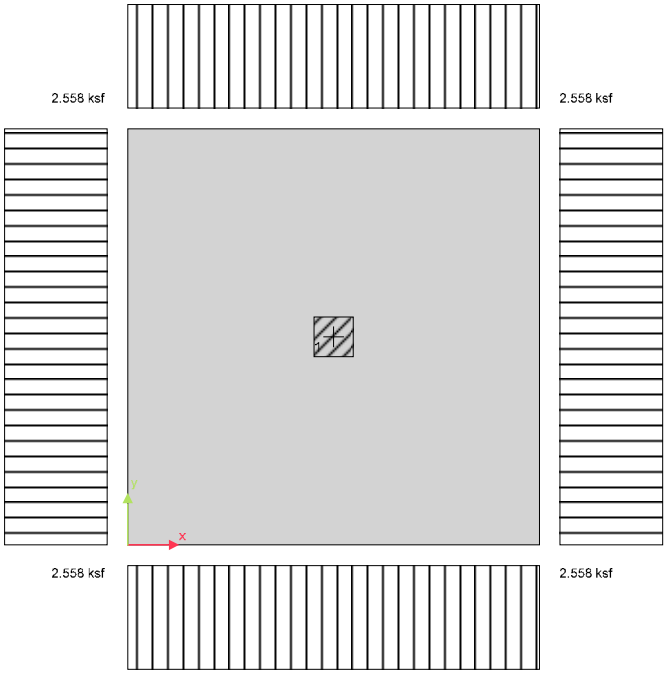
Width of footing; L_y = **7** ft

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)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (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)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.812)
0.6D + 0.6W (0.268)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (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; $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.524** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.524** ksf

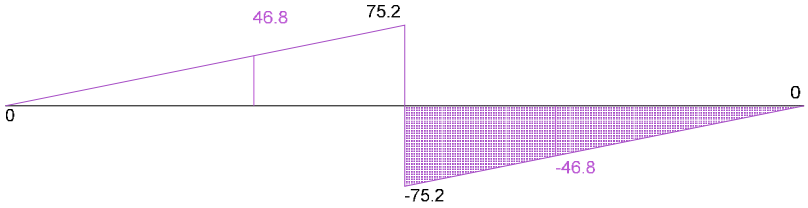
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.524** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.524** ksf

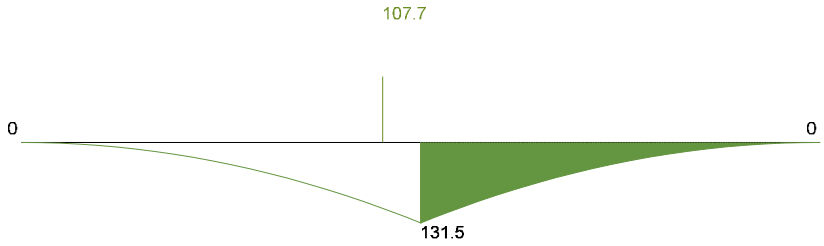
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.524** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.524** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} =$ **107.695** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} =$ **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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.05133**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **189.996** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **170.997** kip_ft

$M_{u.x.max} / \phi M_n =$ **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**

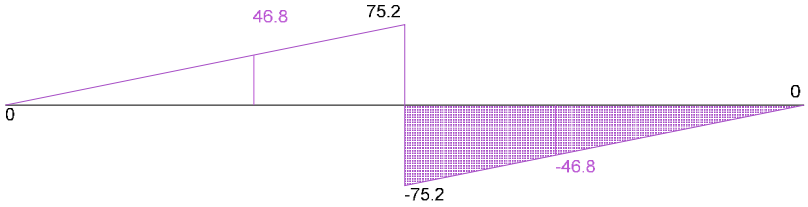
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **81.2** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **60.9** kips

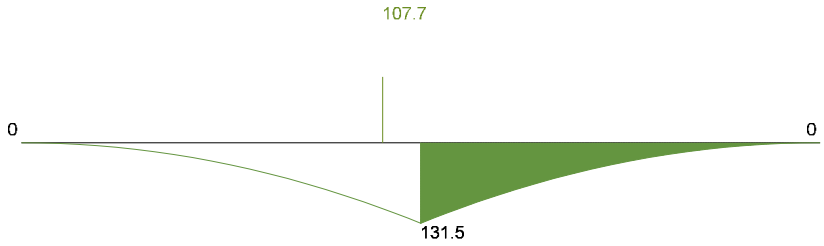
$V_{u.x} / \phi V_n$ = **0.768**

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

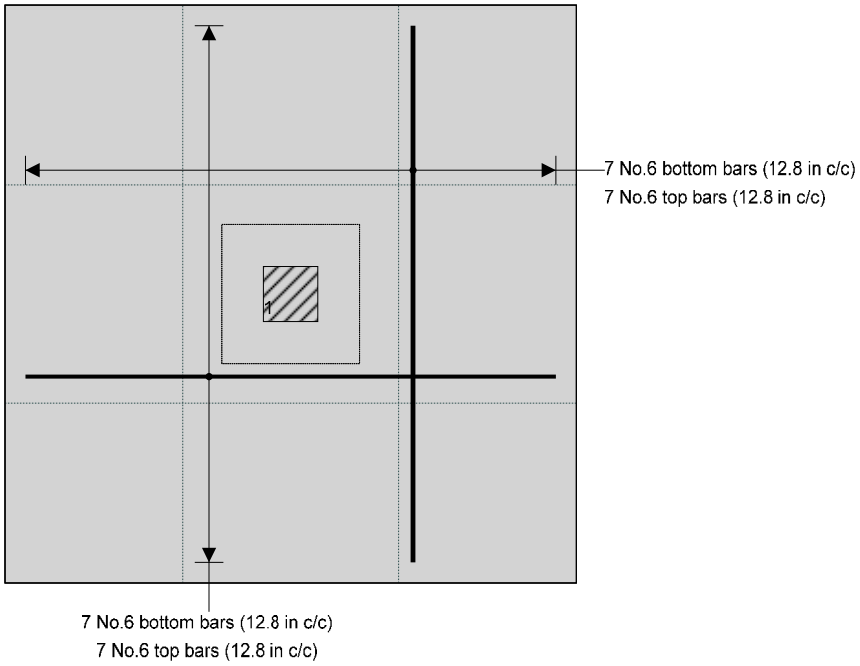
Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **178.446** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n$ = **160.602** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.671**
	PASS - Design moment capacity exceeds ultimate moment load

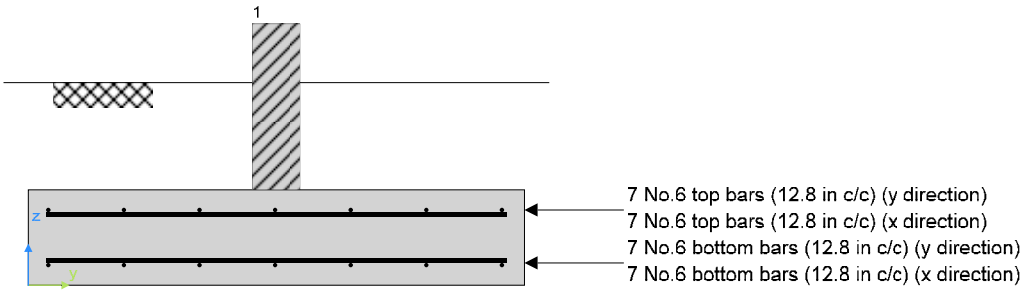
One-way shear design, y direction

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);	λ_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;	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v)$ = **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;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **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);	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
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **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;	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
	$v_{ug} / \phi v_n$ = **0.709**
	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)
1
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)
z
y

;

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

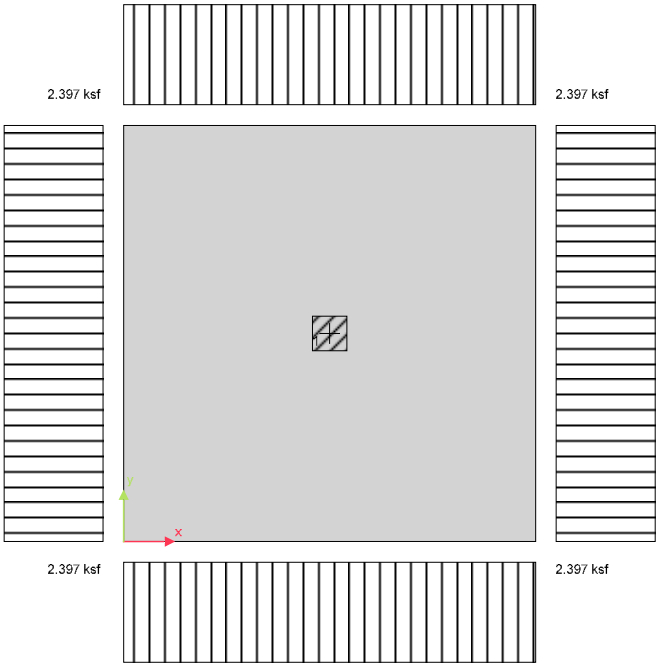
Width of footing; L_y = **10** ft

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



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)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (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)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.767)
0.6D + 0.6W (0.261)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (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

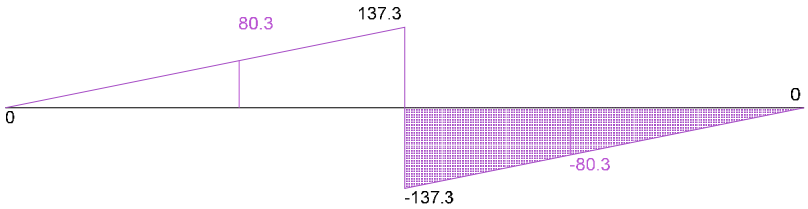
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.32** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.32** ksf

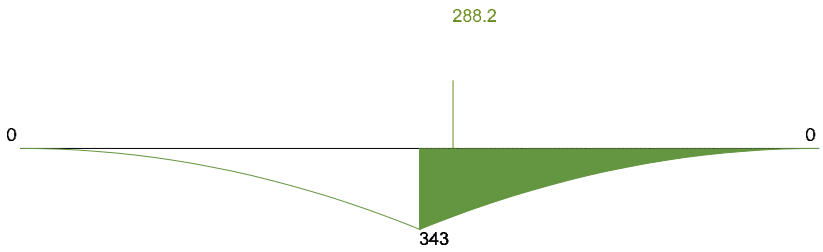
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.32** ksf

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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1$ = **0.837** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.07096**

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**

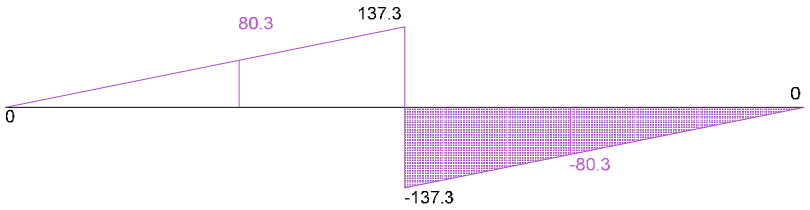
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **170.985** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **128.239** kips

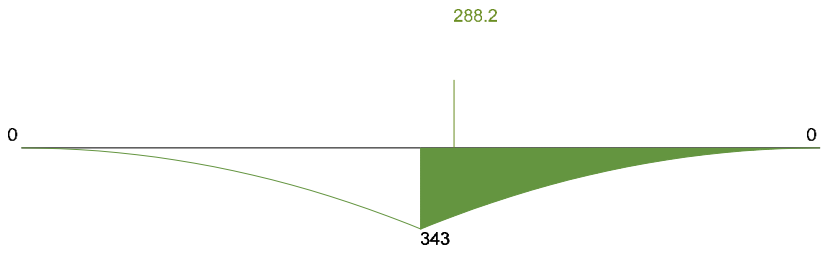
$V_{u.x} / \phi V_n$ = **0.626**

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); $s_{max} = \min(2 \times h, 18\ in)$ = **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

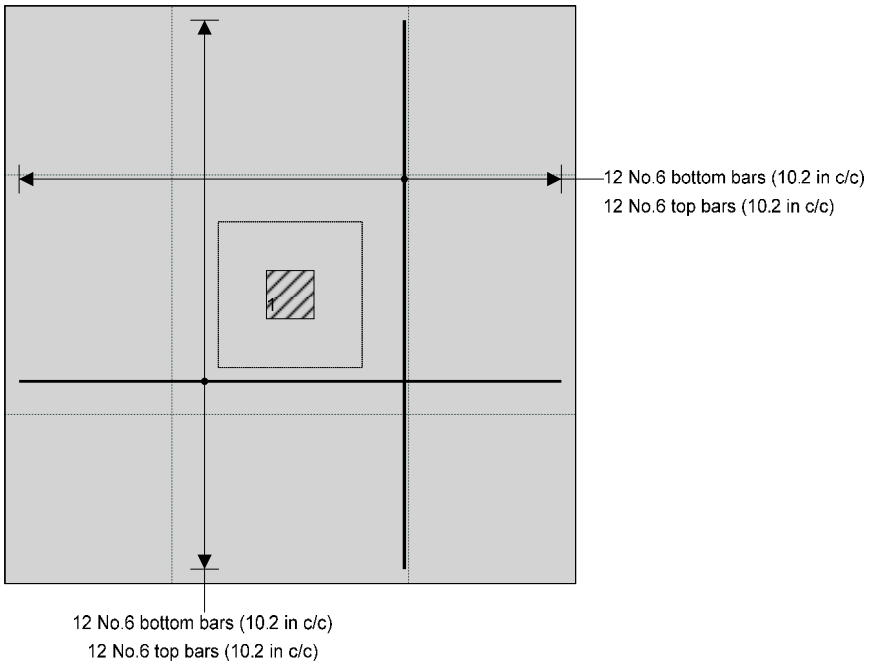
Shear strength reduction factor; ϕ_v = **0.75**

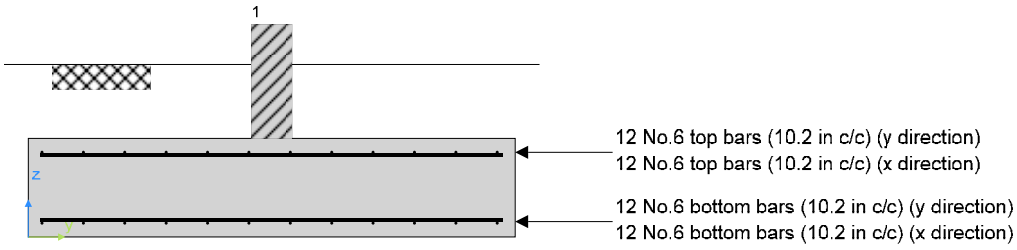
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp}$ = **268.328** psi

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

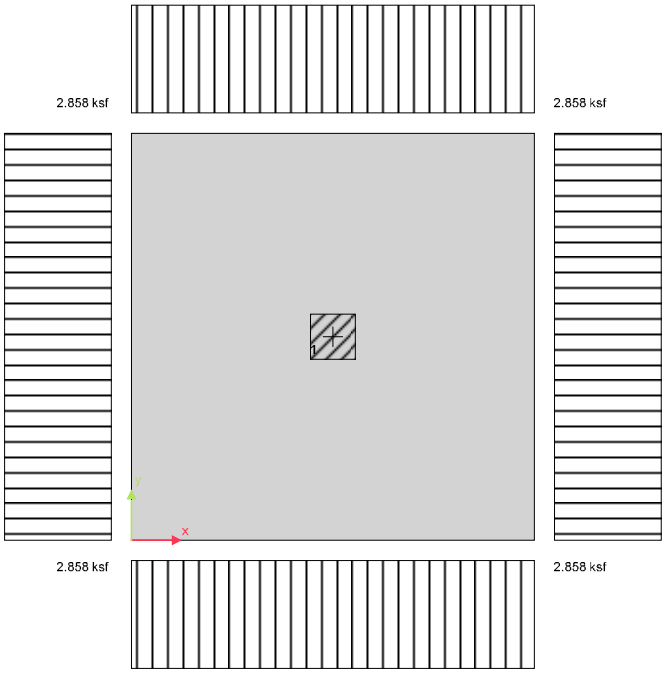
Width of footing; L_y = **6** ft

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



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; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} =$ **102.9** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) =$ **308.7** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) =$ **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

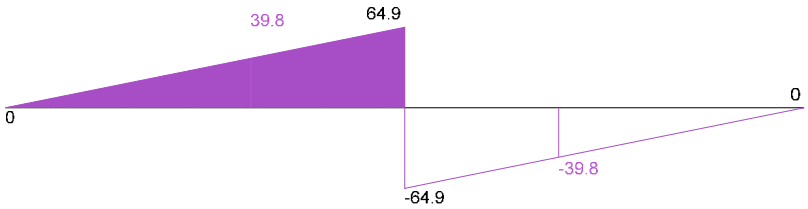
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **4.026** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **4.026** ksf

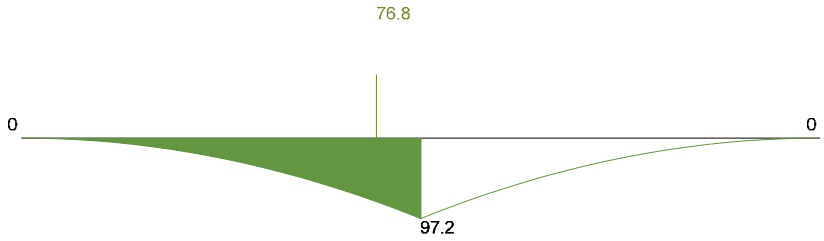
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **4.026** ksf

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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.813** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03619**

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**

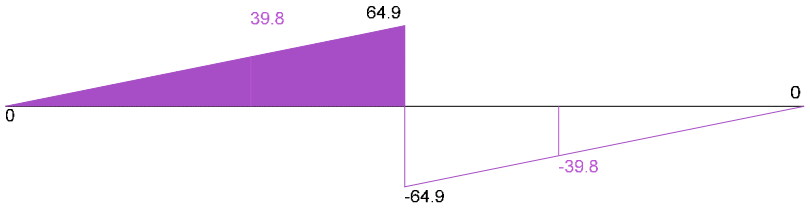
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v) =$ **65.311** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **48.983** kips

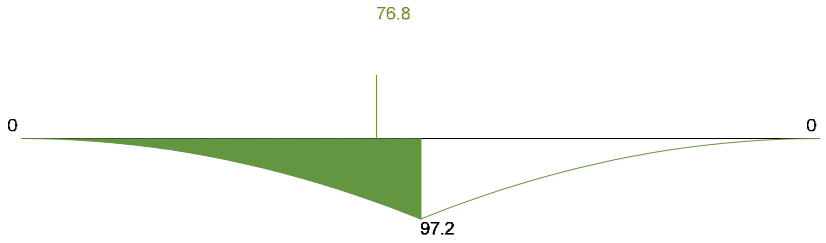
$V_{u.x} / \phi V_n =$ **0.813**

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); $s_{max} = \min(2 \times h, 18\ in) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

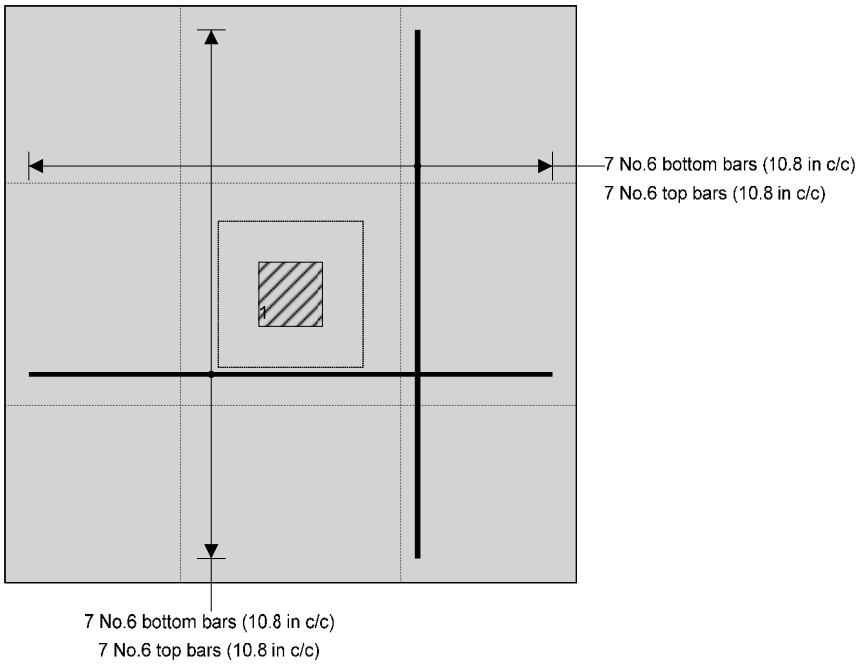
Depth to tension reinforcement;	$d = h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **9.875** in
Depth of compression block;	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.671** in
Neutral axis factor;	β_1 = **0.83**
Depth to neutral axis;	$c = a / \beta_1$ = **0.813** in
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
	$M_{u.y.max} / \phi M_n$ = **0.581**
	PASS - Design moment capacity exceeds ultimate moment load
One-way shear design, y direction	
Ultimate shear force;	$V_{u.y}$ = **39.836** kips
Depth to reinforcement;	$d_v = \min(h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom_t} - \phi_{y.top} / 2)$ = **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
	$V_{u.y} / \phi V_n$ = **0.854**
	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;	$q_{up.avg}$ = **4.026** ksf
Ultimate shear load;	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1} + \gamma_{Lr} \times F_{Lrz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **121.191** kips
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **161.965** 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**

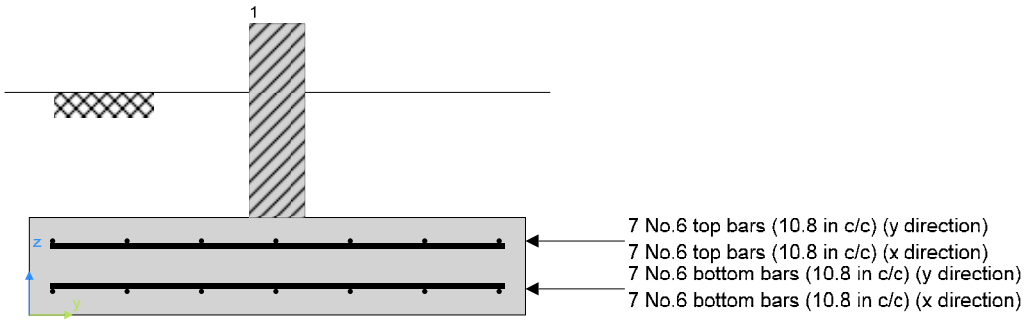
Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp}$ = **268.328** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n$ = **201.246** psi

$v_{ug} / \phi v_n$ = **0.805**

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

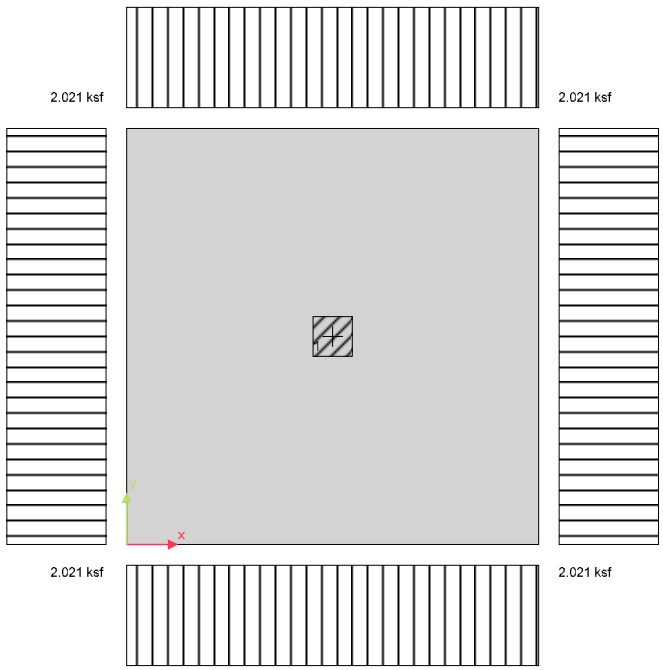
Width of footing; L_y = **7** ft

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; $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.768** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.768** ksf

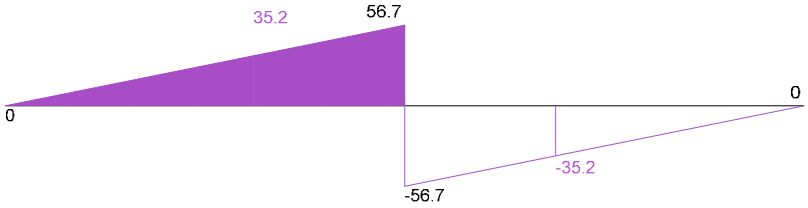
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.768** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **2.768** ksf

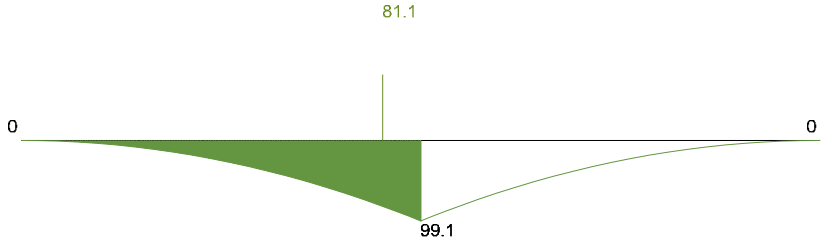
Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **2.768** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **2.768** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max} =$ **81.144** kip_ft

Tension reinforcement provided; 7 No.6 bottom bars (12.8 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov} =$ **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; $\beta_1 =$ **0.83**

Depth to neutral axis; $c = a / \beta_1 =$ **0.697** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.05133**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **189.996** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n =$ **170.997** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.475**

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**

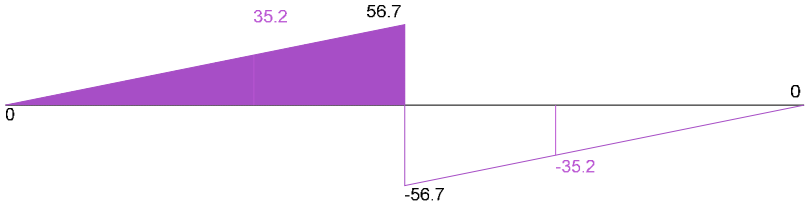
Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **81.2** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n$ = **60.9** kips

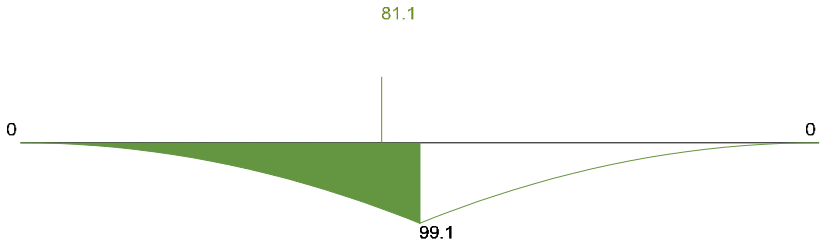
$V_{u.x} / \phi V_n$ = **0.579**

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); $s_{max} = \min(2 \times h, 18\ in)$ = **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

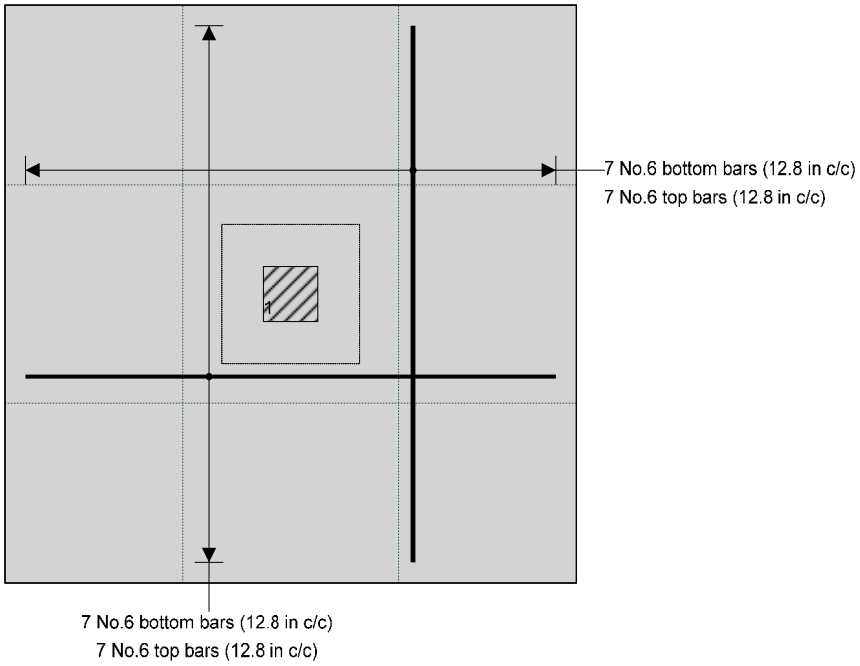
Nominal moment capacity;	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **178.446** kip_ft
Flexural strength reduction factor;	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity;	$\phi M_n = \phi_f \times M_n$ = **160.602** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.505**
	PASS - Design moment capacity exceeds ultimate moment load

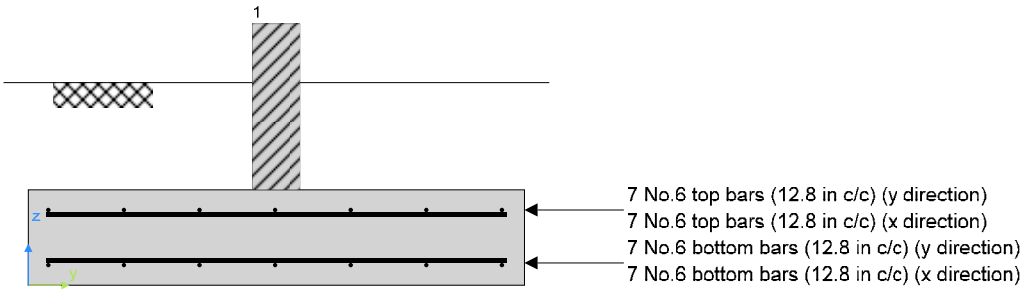
One-way shear design, y direction

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);	λ_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;	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v)$ = **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;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **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);	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
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **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;	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
	$v_{ug} / \phi v_n$ = **0.534**
	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)
1
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)
z
y

;

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

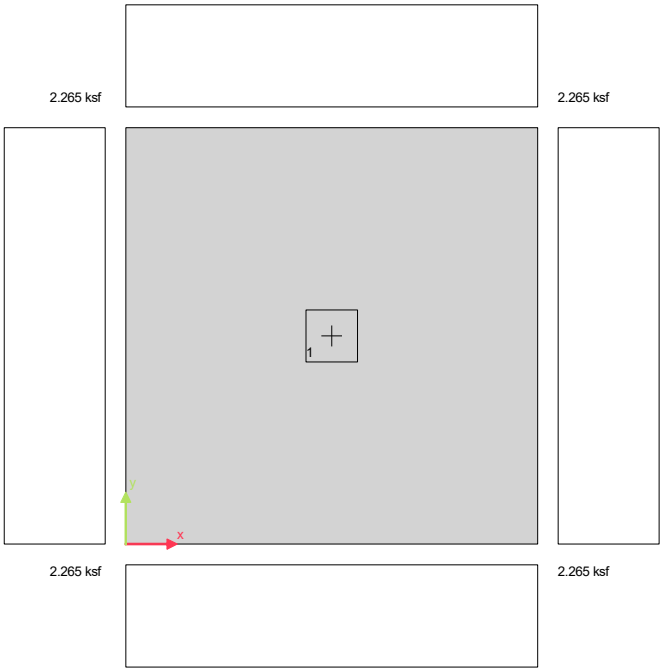
Width of footing L_y = **4** ft

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

$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **36.2** kips

Moments on footing

Moment in x-axis, about x is 0

$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **72.5** kip_ft

Moment in y-axis, about y is 0

$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **72.5** kip_ft

Uplift verification

Vertical force

F_{dz} = **36.235** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis

$e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.265** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.265** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.265** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.265** ksf

Minimum base pressure

$q_{min} = \min(q_1,q_2,q_3,q_4)$ = **2.265** ksf

Maximum base pressure

$q_{max} = \max(q_1,q_2,q_3,q_4)$ = **2.265** ksf

Allowable bearing capacity

Allowable bearing capacity

$q_{allow} = q_{allow_Gross}$ = **3** ksf

q_{max} / q_{allow} = **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

$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_L \times F_{Lz1}$ = **48.7** kips

Moments on footing

Ultimate moment in x-axis, about x is 0

$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **97.4** kip_ft

Ultimate moment in y-axis, about y is 0

$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **97.4** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis

$e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

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

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.043** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.043** ksf

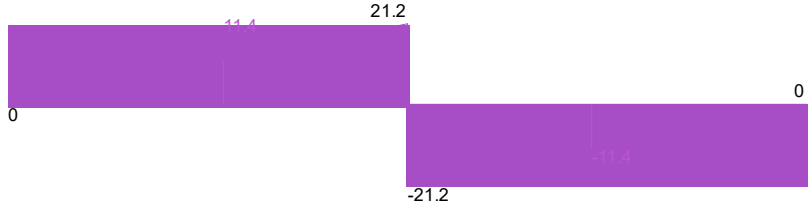
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.043** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **3.043** ksf

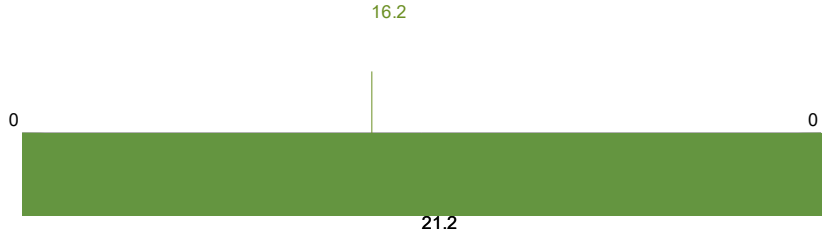
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.043** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **3.043** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} =$ **16.223** kip_ft

Tension reinforcement provided 4 No.5 bottom bars (13.7 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} =$ **1.24** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h =$ **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 $d = h - c_{nom_b} - \phi_{x.bot} / 2 =$ **8.688** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.405** in

Neutral axis factor $\beta_1 =$ **0.83**

Depth to neutral axis $c = a / \beta_1 =$ **0.491** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.05006**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **52.606** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **47.346** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.343**

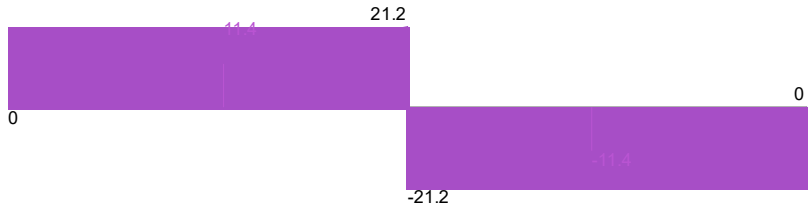
PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

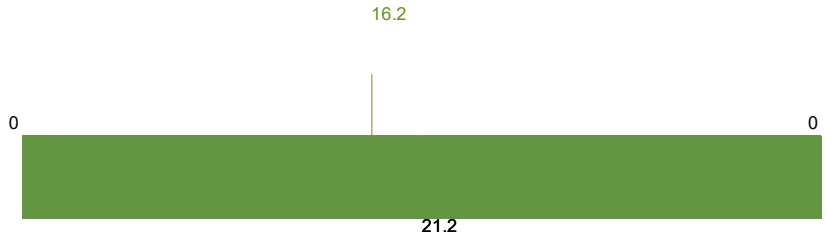
Ultimate shear force	$V_{u.x}$ = **11.421** kips
Depth to reinforcement	$d_v = \min(h - c_{nom_b} - \phi_{x.bot} / 2, h - c_{nom_t} - \phi_{x.top} / 2)$ = **8.688** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00297**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **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

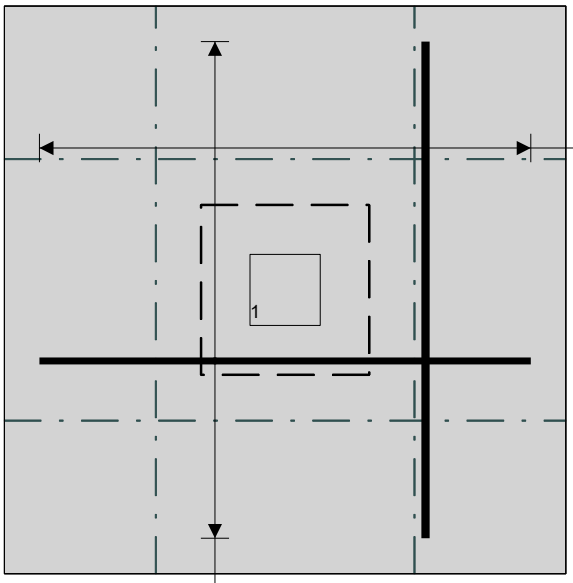
PASS - Area of reinforcement provided exceeds minimum

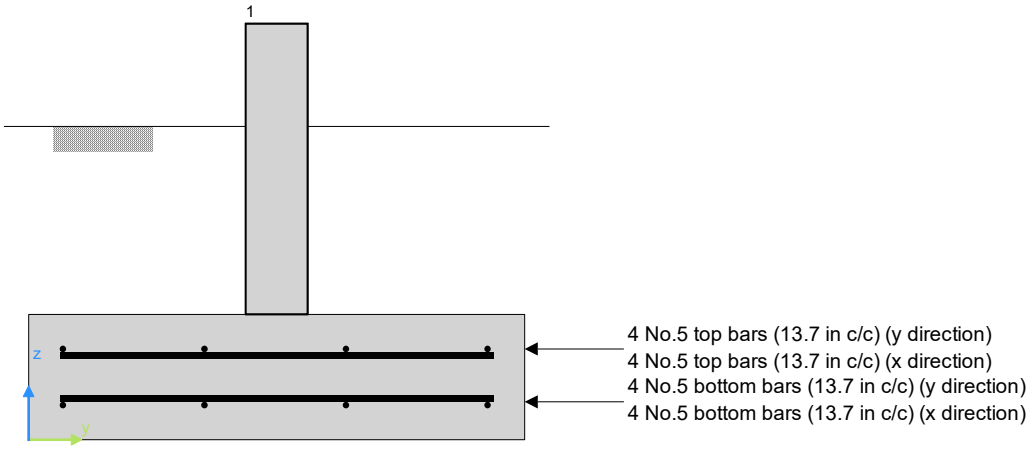
Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18\ in)$ = **18** in

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)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v) =$ **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	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi) =$ **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)	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **402.492** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **524.990** 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
	$v_{ug} / \phi v_n =$ **0.397**
	PASS - Design shear stress capacity exceeds ultimate shear stress load


4 No.5 bottom bars (13.7 in c/c)
4 No.5 top bars (13.7 in c/c)
1
4 No.5 bottom bars (13.7 in c/c)
4 No.5 top bars (13.7 in c/c)


1
z
y
4 No.5 top bars (13.7 in c/c) (y direction)
4 No.5 top bars (13.7 in c/c) (x direction)
4 No.5 bottom bars (13.7 in c/c) (y direction)
4 No.5 bottom bars (13.7 in c/c) (x direction)

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

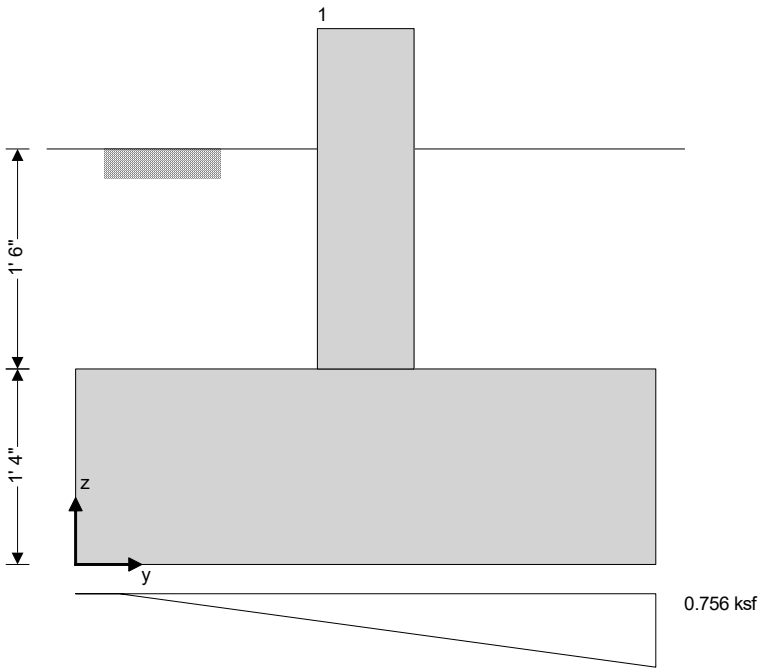
Width of footing L_y = **4** ft

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

Michael Baker
INTERNATIONAL

Creech DRP Phase 2

Aircraft Maintenance Facility (AMF)

Shop Structural Calculations

Structural Calculation Index **Page**

Criteria and Model Information **C1 to C29**

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STRUCTURAL LOAD DATA (UFC 3-301-01) C2
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OPENING OH-SOUTH-1 — OVERHEAD DOOR C4
OPENING SOUTH WALL — LOUVER C5
OPENING EAST WALL — PERSONNEL DOOR C6
ROOF JOIST LEDGER & ANCHORAGE To CMU C8
LOADING CRITERIA C10
SEISMIC CRITERIA C12
BUILDING ENCLOSURE CLASSIFICATION (ASCE 7-22 §26.2) C13
WIND CRITERIA C14
MWFRS — ALL HEIGHTS C15
MWFRS ≤ 60 Ft (LOW-RISE) C16
C&C — WALLS C17
C&C — ROOF JOISTS/PANELS C18
SNOW CRITERIA C21
SERVICEABILITY CRITERIA (DEFLECTION, DRIFT & VIBRATION) C22
LOAD COMBINATIONS (ASCE 7-22, LRFD ONLY) C24
LOAD ENVELOPES (LRFD ONLY) C29

Gravity System Calculations **G1 to G7**

GRAVITY OVERVIEW G1
ROOF DECK DESIGN G2
ROOF JOIST DESIGNS G4
MEZZANINE DECK & BEAMS (DEFLECTION GOVERNING, MEMBER UNITY) G6
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Lateral System Calculations **L1 to L4**

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DIAPHRAGM LOADS & DISTRIBUTION L2
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CMU WALL DESIGN (IN-PLANE & OUT-OF-PLANE) F2
WALL LOADS & CONTINUOUS FOOTING LAYOUT F3
OUT-OF-PLANE (OOP) ANCHORAGE F4

Michael Baker
INTERNATIONAL

Criteria and Model Information

Project: Creech DRP — Shop C (Area E) **Sheet:** C1 **Date:** Oct 2025

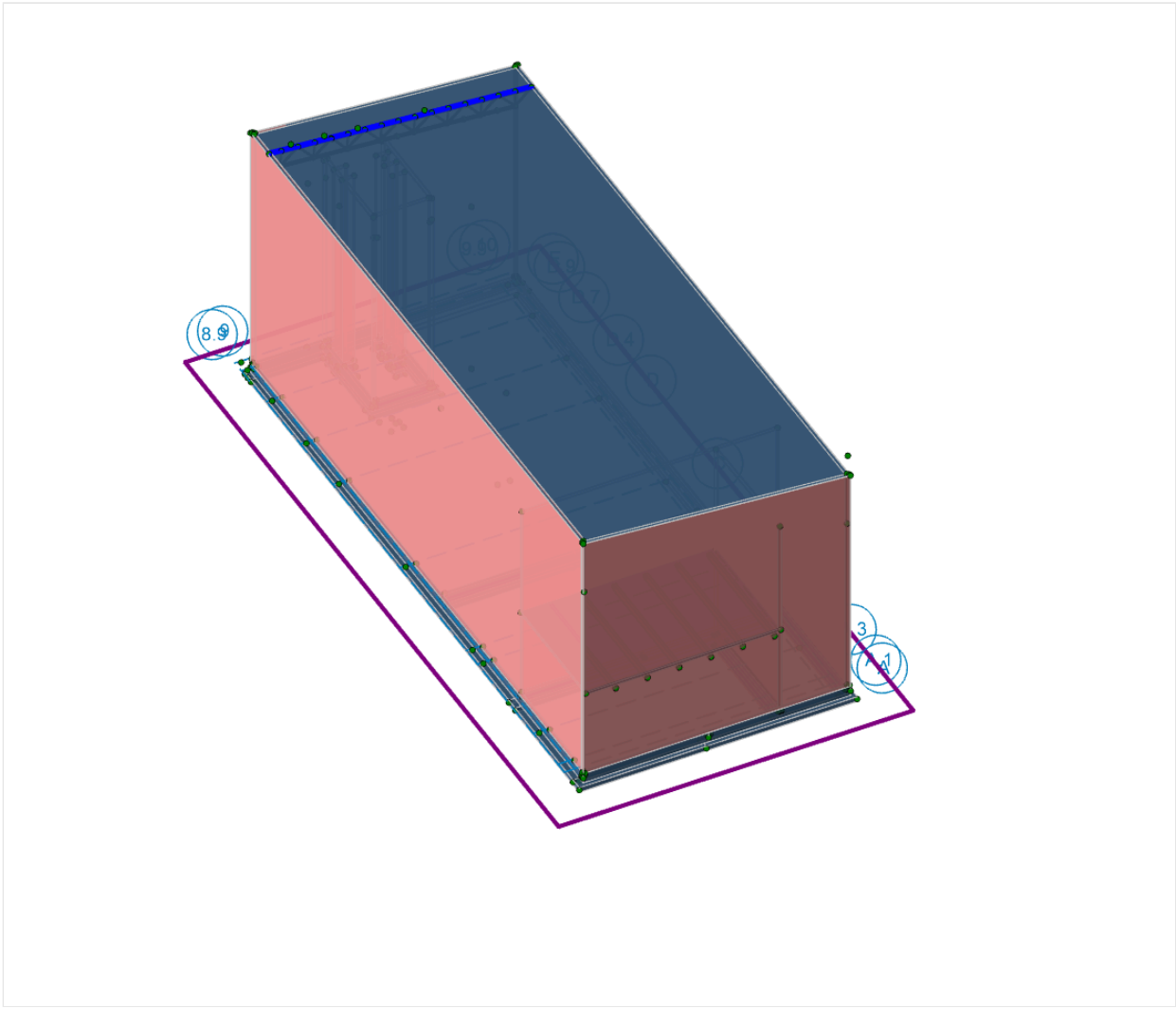

Figure C1-1 — 3D model overview of Shop C showing grids A–G and 9–10.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Materials & Geotechnical (BOD / Geotechnical Report)

Structural steel: AISC 360; ASTM A992/A572 — $F_y = \mathbf{50}$ ksi, $F_u = \mathbf{65}$ ksi.

HSS steel: ASTM A500 Gr C — $F_y = \mathbf{50}$ ksi.

Concrete: ACI 318-19 — Footings $f'_c = \mathbf{3{,}500}$ psi; Slab-on-grade $f'_c = \mathbf{4{,}000}$ psi.

Masonry: TMS 402/602-16 — $f'_m = \mathbf{1{,}500}$ psi (structural CMU).

Allowable bearing (ultimately on native soil): $q_{\text{allow}} = \mathbf{3{,}000}$ psf (include transient increases if permitted by geo).

Sliding coefficient at soil–concrete: $\mu = \mathbf{0.35}$.

Passive resistance (equivalent fluid limit): cap at $\mathbf{300}$ pcf, total not to exceed $\mathbf{3.0}$ ksf without geo approval.

Subgrade reaction (SOG): $k = \mathbf{100}$ pci.

Frost depth (Las Vegas/Creech region): $\mathbf{12}$ in (verify local code / base criteria).

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2) Gravity Loads (Service)

Roof: Dead $= \mathbf{30}$ psf (deck + insul + coverboard + membrane); Live $= \mathbf{20}$ psf (use snow if governing). Snow balanced $P_f = \mathbf{3.5}$ psf; snow drift peak band $P_{\text{drift}} = \mathbf{32}$ psf (use by band width).

Mezzanine: Dead $= \mathbf{78}$ psf (floor system + MEP); Live $= \mathbf{125}$ psf. Partitions (where applicable) $= \mathbf{20}$ psf.

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3) Tributary Widths → Line Loads

Conversion (variables): $w_{\text{line}} = w_{\text{area}} \times b_t$, where b_t is tributary width (ft).

Roof to CMU: $b_t = \mathbf{26.0}$ ft $\rightarrow w_{DL} = \mathbf{780}$ plf, $w_{LL} = \mathbf{520}$ plf, $w_{P_f} = \mathbf{91}$ plf. Drift band examples: $32 \times 5 = \mathbf{160}$ plf, $32 \times 7.5 = \mathbf{240}$ plf, $32 \times 10 = \mathbf{320}$ plf

Mezz to CMU: $b_t = \mathbf{12.0}$ ft $\rightarrow w_{DL} = \mathbf{936}$ plf, $w_{LL} = \mathbf{1{,}500}$ plf (add $+\mathbf{240}$ plf for partitions if not enveloped).

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4) Environmental & Seismic Parameters

Seismic Site Class: D

Risk Category: III

Mapped accelerations: $S_s = \mathbf{0.724}$, $S_1 = \mathbf{0.226}$; **Design:** $S_{DS} = \mathbf{0.589}$, $S_{D1} = \mathbf{0.324}$.

SFRS coefficients (SCBF): $R = \mathbf{6.0}$, $\Omega_0 = \mathbf{2.0}$, $C_d = \mathbf{5.0}$; importance $I_e = \mathbf{1.25}$.

Wind: Basic speed $V = \mathbf{105}$ mph (ult); Exposure C; mean roof height $h \approx \mathbf{24}$ ft; internal $GC_{pi} = \pm\mathbf{0.55}$.

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5) Lateral Earth Pressures (for retaining design)

Soil unit weight: $\gamma = \mathbf{120}\ \mathrm{pcf}$ (select-backfill typical).

Friction angle: $\phi = \mathbf{32}^\circ \rightarrow K_0 \approx 1 - \sin\phi = \mathbf{0.47}$ (at-rest). Active $K_a = \dfrac{1-\sin\phi}{1+\sin\phi} \approx \mathbf{0.31}$; Passive $K_p \approx \dfrac{1}{K_a} \approx \mathbf{3.2}$.

Equivalent fluid pressures: At-rest $\approx K_0\gamma = \mathbf{56}\ \mathrm{pcf}$; Active $\approx K_a\gamma = \mathbf{37}\ \mathrm{pcf}$; Passive (limit for design) $\leq \mathbf{300}\ \mathrm{pcf}$ and total $\leq \mathbf{3.0}\ \mathrm{ksf}$ unless geo allows more.

Provide drainage/weepholes to avoid hydrostatic loads; reduce Ks if cohesion or seismic increments are specified by geo.

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6) Seismic Weights (for base shear)

Roof DL: $30\ \mathrm{psf} \times 1198\ \mathrm{sf} = \mathbf{35.9}\ \mathrm{k}$. **Mezz DL:** $78\ \mathrm{psf} \times 1198\ \mathrm{sf} = \mathbf{93.4}\ \mathrm{k}$. **LL included (25%):** $0.25 \times 125\ \mathrm{psf} \times 1198\ \mathrm{sf} = \mathbf{37.4}\ \mathrm{k}$. **Total W: $\mathbf{166.7}$ k** (see C12).

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7) Design Use Notes

- **Foundations:** Use q_{allow}, μ, and passive limits above; apply transient increases only if allowed by the geotechnical report.
- **Slab-on-grade:** Design per k and geo moisture/drainage recommendations; jointing per plan.
- **CMU walls:** Superimpose roof and mezz line loads (Sect. 3) on the gravity wall plan; use earth pressures in Sect. 5 where retaining.
- **Deck/joists/beams:** Use service loads for deflection; strength checks per LRFD/ASD combinations (see C24).
- **Wind & seismic:** Criteria in C12 and C14 govern lateral design; coordinate collectors/chords with diaphragm plans.

1. Inputs / Geometry / Assumptions

Opening / Wall	Equipment Door @ South Wall
Total wall height to roof deck, H_{wall}	26.5 ft
Bottom of opening above slab/base, H_{bot}	0.0 ft
Clear opening height, H_{open}	8′-0″ = 8.00 ft
Clear opening width (span), L	3′-4″ ≈ 3.33 ft
Elevation of top of opening, H_{top}	8.00 ft above slab
Lintel bearing each side, b_{bear}	8 in
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Grouting	Fully grouted lintel & jambs

We check two cases:
• Case A: Arching active → CMU above the lintel forms a compression arch and pushes vertical gravity into lintel.
• Case B: No arching → lintel must span full clear width under out-of-plane (OOP) lateral load from full-height wall (wind/seismic).

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available masonry height above top of opening:

$$h_{\text{avail}} = H_{\text{wall}} - H_{\text{top}} = 26.5 - 8.00 = 18.5 \text{ ft}$$

Arching limit per 6t:

$$t = 7.625 \text{ in} = 0.635 \text{ ft}$$

$$6t = 6 \times 0.635 = 3.81 \text{ ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 18.5) = \boxed{3.81 \text{ ft}}$$

So even though there's ~18.5 ft of wall above, arching only "counts" the first 3.81 ft.

2.2 Triangular self-weight → uniform line load

Wedge self-weight on lintel:

$$w_D = \tfrac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (take $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment / reaction in lintel (arching case)

Treat lintel as simply supported span $L = 3.33$ ft, uniform $w_u = 0.374$ k/ft. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 3.33}{2} \approx 0.62 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8} = \frac{0.374 \cdot (3.33)^2}{8} \approx 0.52 \text{ k-ft}$$

Bearing per jamb (vertical reaction each side):

$$R_u = \frac{w_u L}{2} \approx 0.62 \text{ k}$$

Check that against ~8" of bearing at f'm = 2000 psi → OK.

3. Case B — No Arching (Full OOP Lateral)

3.1 OOP panel load

Worst case: assume arching can't form. Now the lintel resists lateral load from the entire wall height above grade. Tributary height for OOP:

$$h_{\text{panel}} = H_{\text{wall}} = 26.5 \text{ ft}$$

Use controlling OOP pressure:

$$p_{\text{lat}} = 33 \text{ psf}$$

Convert to factored line load per foot of lintel span:

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 26.5 = 874.5 \text{ plf} \approx 0.8745 \text{ k/ft}$$

We'll take $w_{\text{lat}} = 0.8745$ k/ft as the factored OOP line load.

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.8745\ \text{k/ft}$, span $L = 3.33\ \text{ft}$. Shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.8745 \cdot 3.33}{2} \approx 1.46\ \text{k}$$

Moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8} = \frac{0.8745 \cdot (3.33)^2}{8} \approx 1.21\ \text{k-ft}$$

That 1.21 k-ft controls vs 0.52 k-ft from the arching case.

3.3 Jamb column quick check

Each jamb acts like a vertical CMU pier, 8″×16″, fully grouted. Take tributary half-width $\approx L/2 = 1.67\ \text{ft}$. OOP line load on one jamb from lateral using 33 psf:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 1.67 \approx 33 \times 1.67 \approx 55\ \text{plf}$$

Cantilever height ≈ full wall height 26.5 ft. With a single #5 vertical bar full height, hooked/anchored into the footing and lapped into the lintel, the jamb is acceptable for this small door. (We're not detailing ties here because this is a short, narrow pier with low demand.)

4. Required Reinforcement / Recommended Detail

Lintel (governed by Case B OOP, $M_u \approx 1.21$ k-ft, $V_u \approx 1.46$ k):
• 16″ CMU lintel / bond beam (Type A).
• (2) #5 horizontal bars continuous.
• Shear is still low enough that ties/stirrups aren't mandatory per calc, but include if standard.

Jamb columns (pilasters):
• Each jamb = 8×16 fully grouted CMU pilaster (Type A2).
• (1) #5 vertical bar full height, lapped/developed into the lintel and into the footing.
• For the OOP case include #3 ties @ 8" o.c. for confinement.

Controlling demand:
• Arching case: $M_u \approx 0.52$ k-ft.
• No-arch OOP case: $M_u^{(\text{lat})} \approx 1.21$ k-ft.
→ Final design controls by OOP / no-arch.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft (≤6t)
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	0.62 k
Moment M_u	0.52 k-ft
Lintel trial	16″ CMU lintel (Type A) w/ (2) #5 horiz
Jamb trial	8×16 pilaster (Type A2) w/ (1) #5 vert

Case B — No Arching (Full OOP Lateral, 33 psf)	
Panel height used	26.5 ft
OOP line load w_{lat}	0.8745 k/ft (≈874.5 plf from 33 psf × 26.5 ft)
Shear $V_u^{(\text{lat})}$	1.46 k
Moment $M_u^{(\text{lat})}$	1.21 k-ft
Controlling design?	**Yes — design for OOP case**

6. Masonry Lintel / Jamb Schedule (For Sheets)

16A Lintel / Jamb (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16A Lintel	16″	Type A	(2) #5 horiz	—	16A C	Arching assumed active; ~0.52 k-ft, V≈0.62 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16A C	8×16	Type A2 jamb / pilaster	(1) #5 full height	—	Lap bar into lintel; arching/gravity case

16B Lintel / Jamb (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16B Lintel	16″	Type A	(2) #5 horiz	—	16B C	No arching; OOP 33 psf → ~1.21 k-ft, V≈1.46 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16B C	8×16	Type A2 jamb / pilaster	(1) #5 full height	#3 @ 8″ o.c.	Lap bar into lintel; design for OOP (33 psf envelope)

1. Inputs / Geometry / Assumptions

Opening / Wall	Overhead Door @ South Wall
Total wall height to roof deck, H_{wall}	26.5 ft
Bottom of opening above slab/base, H_{bot}	0.0 ft
Clear opening height, H_{open}	14.0 ft
Clear opening width (span), L	12.0 ft
Lintel bearing each side, b_{bear}	8 in (min)
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Grouting	Fully grouted lintel & jamb/pilaster regions

Two cases checked:
• Case A: Arching active → CMU wedge above the lintel only (gravity).
• Case B: No arching → lintel spans full opening under out-of-plane (OOP) lateral from full-height wall.

For a 12 ft wide OH door near slab, the no-arch / OOP case is usually what blows things up.

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available masonry height above top of opening up to next stiff element:

$$h_{\text{avail}} = H_{\text{wall}} - (H_{\text{bot}} + H_{\text{open}})$$

$$h_{\text{avail}} = 26.5 - (0.0 + 14.0) = 12.5 \text{ ft}$$

Arching limit based on 6t:

$$t = 7.625 \text{ in} = 0.635 \text{ ft}$$

$$6t = 6 \times 0.635 = 3.81 \text{ ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 12.5) = \boxed{3.81 \text{ ft}}$$

2.2 Triangular self-weight → uniform line load

Wedge self-weight (service line load on lintel):

$$w_D = \tfrac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (take $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment in lintel (arching case)

Treat lintel as simply supported span $L = 12.0$ ft, uniform $w_u = 0.374$ k/ft. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 12.0}{2} \approx 2.24 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8} = \frac{0.374 \cdot (12.0)^2}{8} = 6.73 \text{ k-ft}$$

Bearing per jamb:

$$R_u = \frac{w_u L}{2} \approx 2.24 \text{ k}$$

At these reactions you usually want a built-up pilaster / jamb with plate or bearing pad.

3. Case B — No Arching (Full OOP Lateral)

3.1 OOP panel load

Worst case: assume arching is not reliable. Lintel must span 12.0 ft under lateral pressure from the full wall height. Tributary wall height:

$$h_{\text{panel}} = H_{\text{wall}} = 26.5 \text{ ft}$$

Use the controlling lateral pressure:

$$p_{\text{lat}} = 33 \text{ psf}$$

Convert to line load (per foot of lintel span):

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 26.5 = 874.5 \text{ plf} \approx 0.8745 \text{ k/ft}$$

We'll treat $w_{\text{lat}} = 0.8745$ k/ft as factored OOP.

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.8745\ \text{k/ft}$, span $L = 12.0\ \text{ft}$. Factored shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.8745 \cdot 12.0}{2} \approx 5.25\ \text{k}$$

Factored moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8} = \frac{0.8745 \cdot (12.0)^2}{8} \approx 15.74\ \text{k-ft}$$

$\rightarrow \approx 15.74$ k-ft >> ≈ 6.73 k-ft from arching case. The OOP / no-arch case absolutely controls.

3.3 Jamb / pilaster quick check

Each jamb is essentially a CMU pilaster at the OH door edge. This is not a tiny door — clear span is 12.0 ft — so the jambs see big reactions. Tributary half-width $\approx L/2 = 6\ \text{ft}$. Take the same 33 psf pressure: OOP line load on ONE jamb:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 6 = 33 \times 6 = 198\ \text{plf}$$

That pilaster is basically resisting a tall cantilever of ~26.5 ft with a significant base moment. For this magnitude, we recommend pilaster-level reinforcement (at least (2) #6 vertical bars with ties).

4. Required Reinforcement / Recommended Detail

Lintel (design for Case B OOP, $M_u \approx 15.74$ k-ft, $V_u \approx 5.25$ k):
- Deep CMU lintel / bond beam zone, target ~24″ total depth (multiple grouted courses acting as a built-up lintel).
- (2) #6 horizontal bars continuous (top & bottom of lintel zone).
- Closed ties / confinement steel (#3 ties @ 8″ o.c. typical) in the lintel zone to carry ~5.25 k shear and develop bars.
- Provide steel bearing pads / plates and fully grouted jamb seats at each end, because reactions are high.

Jamb pilasters (each side of OH door):
- Treat jambs as CMU pilasters, ~16″ (or wider) built-up grouted region at each door edge.
- (2) #6 vertical bars full height, hooked/anchored into the footing and lapped/developed into the lintel zone.
- #3 ties/hoops @ 8″ o.c. min in jamb pilaster region for confinement and to resist OOP base moment from ~26.5 ft tall cantilever strip.
- This is much heavier than a personnel door jamb — required because the span is 12 ft and the lintel is carrying ~15.7 k-ft.

Controlling demand:
- Arching gravity case: $M_u \approx 6.73$ k-ft.
- No-arch OOP case: $M_u^{(\text{lat})} \approx 15.74$ k-ft.

$\rightarrow$ Final design governs by OOP / no-arch.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	2.24 k
Moment M_u	6.73 k-ft
Lintel trial	Deep CMU lintel
Jamb trial	Pilaster jamb

Case B — No Arching (Full OOP Lateral, 33 psf)	
Panel height used	26.5 ft
OOP line load w_{lat}	0.8745 k/ft (≈874.5 plf from 33 psf × 26.5 ft)
Shear $V_u^{(\text{lat})}$	5.25 k
Moment $M_u^{(\text{lat})}$	15.74 k-ft
Controlling design?	**Yes — No-arch/OOP governs**

6. Masonry Lintel / Jamb Schedule (For Sheets)

6A.Lintel / Pilaster (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
OH Door Lintel	24″	Type C	(2) #6 horiz cont.	#3 ties @ 8″ o.c.	24A C	Arching assumed active; ~6.7 k-ft, V≈2.2 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
OH Door Column	8x16	Type C	(2) #6 full height	#3 @ 8″ o.c.	Bars developed into footing & lintel; supports arching case

6B. Lintel / Pilaster (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
OH Door Lintel	24″	Type C3	(2) #6 horiz cont.	#3 ties @ 8″ o.c. (shear ≈5.3 k)	24B C	No arching assumed; design for OOP ~15.7 k-ft, V≈5.25 k (33 psf)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
OH Door Column	8x24	Type C3	(2) #6 full height (hooked into footing & lapped into lintel)	#3 @ 8″ o.c. confinement ties	Pilaster sized for tall cantilever under 33 psf OOP; supports 12 ft span lintel

1. Inputs / Geometry / Assumptions

Wall	South
Opening Name	Louver-1 (LV-SOUTH-1)
Opening clear width (span), L	8.1 ft (≈ 97 in)
Opening clear height, H_{open}	7.1 ft (≈ 85 in)
Elevation of top of opening, H_{top}	20.0 ft above slab
Bottom of opening, $H_{\text{bottom}} = H_{\text{top}} - H_{\text{open}}$	12.9 ft above slab (≈ 20.0 − 7.1)
Total wall height at this location, H_{wall}	26.5 ft
Remaining wall height above top of opening	$H_{\text{wall}} - H_{\text{top}} = 26.5 - 20.0 = 6.5$ ft
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Lintel bearing each side, b_{bear}	8 in assumed (typ.)
Grouting	Fully grouted lintel & jamb/pilaster region at opening

Two cases checked:
• Case A (Arching Active): masonry above the louver develops a compression arch, producing a gravity wedge.
• Case B (No Arching / OOP): lintel resists lateral from the wall strip above the opening with no arching help.

NOTE: The louver sits up in the wall. The lintel is at ~20 ft above slab, and only ~6.5 ft of wall continues above. So the OOP tributary height is 6.5 ft, not the full 26.5 ft like at a door at grade.

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available wall above top of opening:

$$h_{\text{avail}} = H_{\text{wall}} - H_{\text{top}} = 26.5 - 20.0 = 6.5 \text{ ft}$$

Arching limit based on 6t:

$$t = 7.625 \text{ in} = 0.635 \text{ ft}$$

$$6t = 6 \times 0.635 = 3.81 \text{ ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 6.5) = \boxed{3.81 \text{ ft}}$$

2.2 Triangular self-weight → uniform line load

Use fully grouted weight 164 psf and arch wedge height $h_a = 3.81$ ft:

$$w_D = \tfrac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (take $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment in lintel (arching case)

Lintel span $L = 8.1$ ft, uniform $w_u = 0.374$ k/ft. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 8.1}{2} \approx 1.51 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8}$$

$$L^2 = (8.1)^2 = 65.61$$

$$M_u = \frac{0.374 \cdot 65.61}{8} \approx 3.07 \text{ k-ft}$$

Bearing per jamb:

$$R_u = \frac{w_u L}{2} \approx 1.51 \text{ k}$$

Bearing at 8 in seat on fully grouted CMU at ~20 ft elevation is typically OK at these reactions.

3. Case B — No Arching (OOP Strip Above Opening)

3.1 OOP panel load

Assume arching is not available. Lintel resists lateral from only the 6.5 ft of wall above it (not full 26.5 ft because the opening is elevated). Tributary OOP height:

$$h_{\text{panel}} = H_{\text{wall}} - H_{\text{top}} = 6.5 \text{ ft}$$

Use controlling lateral pressure:

$$p_{\text{lat}} = 33 \text{ psf}$$

Convert to line load on lintel:

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 6.5 = 214.5 \text{ plf} \approx 0.2145 \text{ k/ft}$$

We take $w_{\text{lat}} = 0.2145$ k/ft as factored OOP for “no arching.”

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.2145\ \text{k/ft}$, span $L = 8.1\ \text{ft}$. Factored shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.2145 \cdot 8.1}{2} \approx 0.87\ \text{k}$$

Factored moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8}$$

$$L^2 = 65.61$$

$$M_u^{(\text{lat})} \approx \frac{0.2145 \cdot 65.61}{8} \approx 1.76\ \text{k-ft}$$

Here, 3.07 k-ft (arching gravity) still controls vs 1.76 k-ft (OOP no-arch), because the tributary strip above is only 6.5 ft tall.

3.3 Jamb / pilaster quick check

Each jamb is a short pilaster region at the louver edges, not a full-height wall pier from slab. Tributary half-width $\approx L/2 = 8.1/2 = 4.05\ \text{ft}$. With 33 psf: OOP line load on ONE jamb:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 4.05 = 33 \times 4.05 \approx 133.65\ \text{plf}$$

That pilaster only has to cantilever about 6.5 ft above the louver. Demand is much lower than the OH door jamb at slab. One #5 vertical bar (or #5 each face if your standard) developed into the lintel above is typically enough here.

4. Required Reinforcement / Recommended Detail

Lintel (controls by arching/gravity $M_u \approx 3.07$ k-ft, $V_u \approx 1.51$ k):
• CMU lintel / bond beam at top of louver opening, ~16″ deep grouted course (Type A).
• (2) #5 horizontal bars continuous.
• Provide adequate 8″ min bearing each side and lap bars into jamb pilaster.
• Shear demand from OOP case is only ~0.87 k, so tie/stirrup requirements are mild.

Jamb / pilaster at each side of louver:
• Fully grouted CMU pilaster region (~16″ wide built-out zone) at each jamb of the louver.
• (1) #5 vertical full height through that pilaster, develop into supporting CMU below and into lintel above.
• Ties / hoops @ ~8″ o.c. only if required by office standard; tributary OOP height is short (6.5 ft), so this is not the same demand as a 12 ft OH door at slab.

Controlling demand:
• Arching case: $M_u \approx 3.07$ k-ft, $V_u \approx 1.51$ k.
• No-arch OOP case (33 psf over 6.5 ft): $M_u^{(\text{lat})} \approx 1.76$ k-ft, $V_u^{(\text{lat})} \approx 0.87$ k.
→ Arching / gravity wedge governs.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft (≤ 6t)
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	1.51 k
Moment M_u	3.07 k-ft
Lintel trial	16″ CMU lintel (Type A) w/ (2) #5 horiz
Jamb trial	CMU pilaster/jamb w/ (1) #5 vert

Case B — No Arching (OOP Strip Above Opening, 33 psf)	
Panel height used	6.5 ft above lintel (20.0 ft → 26.5 ft)
OOP line load w_{lat}	0.2145 k/ft (≈214.5 plf from 33 psf × 6.5 ft)
Shear $V_u^{(\text{lat})}$	0.87 k
Moment $M_u^{(\text{lat})}$	1.76 k-ft
Controlling design?	Arching gravity still governs (3.07 k-ft > 1.76 k-ft)

6. Masonry Lintel / Jamb Schedule (For Sheets)

6A. Lintel / Pilaster (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16L A Lintel	16″	Type A	(2) #5 horiz cont.	—	16L A C	Arching assumed active; governs ~3.07 k-ft, V≈1.51 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16L A C	8×16	Type A2	(1) #5 full height	—	Short pilaster above elevated opening; lap bar into lintel

6B. Lintel / Pilaster (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16L B Lintel	16″	Type A	(2) #5 horiz cont.	—	16L B C	No arching assumed; OOP strip 6.5 ft tall, ~1.76 k-ft, shear ~0.87 k (33 psf)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16L B C	8×16	Type A2	(1) #5 full height	#3 @ 8″ o.c. if required	Pilaster resists local OOP from 6.5 ft tributary height; not full-height wall demand

1. Inputs / Geometry / Assumptions

Opening / Wall	Personnel Door @ South Wall
Total wall height to roof deck, H_{wall}	26.5 ft
Bottom of opening above slab/base, H_{bot}	0.0 ft
Clear opening height, H_{open}	7′-4″ ≈ 7.33 ft
Clear opening width (span), L	3′-4″ ≈ 3.33 ft
Lintel bearing each side, b_{bear}	8 in
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Grouting	Fully grouted lintel & jambs

Two cases checked:
• Case A: Arching active → gravity wedge of CMU above lintel only.
• Case B: No arching → lintel spans full clear width under out-of-plane (OOP) lateral pressure from wind/seismic acting over full wall height.

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available masonry height above top of opening up to next stiff element:

$$h_{\text{avail}} = H_{\text{wall}} - (H_{\text{bot}} + H_{\text{open}})$$

$$h_{\text{avail}} = 26.5 - (0.0 + 7.33) = 19.17 \text{ ft}$$

Arching limit per 6t:

$$t = 7.625 \text{ in} = 0.635 \text{ ft}$$

$$6t = 6 \times 0.635 = 3.81 \text{ ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 19.17) = \boxed{3.81 \text{ ft}}$$

2.2 Triangular self-weight → uniform line load

Wedge self-weight (service line load on lintel):

$$w_D = \tfrac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (use $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment in lintel (arching case)

Treat lintel as simply supported span $L = 3.33$ ft, uniform $w_u = 0.374$ k/ft. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 3.33}{2} \approx 0.62 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8} = \frac{0.374 \cdot (3.33)^2}{8} \approx 0.52 \text{ k-ft}$$

Bearing per jamb:

$$R_u = \frac{w_u L}{2} \approx 0.62 \text{ k}$$

With 8 in bearing and $f'_m = 2000$ psi, bearing stress is acceptable.

3. Case B — No Arching (Full OOP Lateral)

3.1 OOP panel load

Worst case: arching prevented. Lintel carries full clear width under out-of-plane wind/seismic from full wall height. Take full wall height:

$$h_{\text{panel}} = H_{\text{wall}} = 26.5 \text{ ft}$$

Use lateral envelope pressure (wind 25 psf vs seismic 33 psf → control 33 psf):

$$p_{\text{lat}} = 33 \text{ psf}$$

Line load on lintel, per ft of lintel span:

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 26.5 = 874.5 \text{ plf} \approx 0.8745 \text{ k/ft}$$

We'll treat $w_{\text{lat}} = 0.8745$ k/ft as already factored envelope.

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.8745\ \text{k/ft}$, span $L = 3.33\ \text{ft}$. Shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.8745 \cdot 3.33}{2} \approx 1.46\ \text{k}$$

Moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8} = \frac{0.8745 \cdot (3.33)^2}{8} \approx 1.21\ \text{k-ft}$$

This ~1.21 k-ft controls vs 0.52 k-ft from arching.

3.3 Jamb column quick check

Each jamb acts like a vertical CMU pier, 8"x16", fully grouted. Take tributary half-width $\approx L/2 = 1.67\ \text{ft}$. OOP line load on one jamb from lateral using 33 psf:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 1.67 \approx 33 \times 1.67 \approx 55\ \text{plf}$$

Cantilever height ~ full wall height 26.5 ft. With a single #5 vertical bar full height, hooked/anchored into footing and lapped into the lintel, the jamb is acceptable for this small door. (We're not detailing ties here because this is a short, narrow pier with low demand.)

4. Required Reinforcement / Recommended Detail

Lintel (design for Case B OOP):
• 8"x16" CMU lintel (Type A).
• (2) #5 horiz bars continuous near faces.
• Stirrups/closed ties not strictly required by the low shear (~1.46 k), include only if office standard.

Jamb columns:
• Each jamb = 8"x16" fully grouted CMU pier.
• (1) #5 vertical bar full height in each jamb, develop into footing and lap into lintel bond beam.

Controlling demand:
• Arching case moment $M_u \approx 0.52\ \text{k-ft}$.
• No-arch OOP case moment $M_u^{(\text{lat})} \approx 1.21\ \text{k-ft}$.
→ Design for 1.21 k-ft.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	0.62 k
Moment M_u	0.52 k-ft
Lintel trial	8″×16″ CMU w/ (2) #5 horiz
Jamb trial	8″×16″ jamb w/ (1) #5 vert

Case B — No Arching (Full OOP Lateral)	
Panel height used	26.5 ft
OOP line load w_{lat}	0.8745 k/ft (≈874.5 plf from 33 psf × 26.5 ft)
Shear $V_u^{(\text{lat})}$	1.46 k
Moment $M_u^{(\text{lat})}$	1.21 k-ft
Controlling design?	**Envelope governs — design for OOP case**

6. Masonry Lintel / Jamb Schedule (For Sheets)

16A Lintel / Jamb (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16A Lintel	16″	Type A	(2) #5 horiz	—	16A C	Arching assumed active; governs gravity (~0.52 k-ft, V≈0.62 k)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16A C	8×16	Type A2	(1) #5 full height	—	Lap bar into lintel; arching/gravity case

16B Lintel / Jamb (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16B Lintel	16″	Type A	(2) #5 horiz	—	16B C	No arching assumed; design for OOP ~1.21 k-ft, shear ~1.46 k (33 psf)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16B C	8×16	Type A2	(1) #5 full height	#3 @ 8″ o.c.	Lap bar into lintel; design for OOP (33 psf envelope)

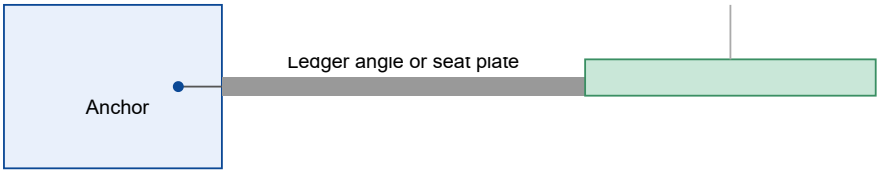
C8 — ROOF JOIST LEDGER & ANCHORAGE TO CMU

Project: Creech DRP — Shop C (Area E) **Discipline:** Criteria — Connections to CMU **Sheet:** C8

Basis: Roof joists @ 7′-0″ o.c.; no parapet; CMU 8″ fully grouted

1) Ledger / Seat Configuration

- Roof joists bear at CMU. Provide continuous **ledger angle** or **discrete seat plates** at joist lines.
- Anchorage into grouted CMU cells with plate washers; minimum edge distances per TMS/ACI 530 and manufacturer.
- Provide bond beam at roof line to collect diaphragm loads and distribute anchor forces.



2) Uplift Per Anchor (Service → Strength)

Net wind uplift at roof (corner) per Section C: $p_{net} \approx -4.4$ psf. Use conservative tributary to each anchor:

- Joist spacing $s = 7.0'$
- Anchor spacing along wall $a = 4.0'$ (typical)
- Tributary area $A_{trib} = s \times a = 28$ sf

Service tension per anchor:

$$T_s = |p_{net}| \times A_{trib} = 4.4 \times 28 = \mathbf{123}\ \text{lb}$$

Strength design (LRFD uplift):

$$T_u \approx 1.6 \times T_s = \mathbf{197}\ \text{lb} \quad (\text{wind})$$

If PM directs C&C corner uplift basis, you may alternatively adopt $|p| = 25$ psf $\Rightarrow T_u \approx 1.6 \times (25 \times 28) = \mathbf{1.12\ k}$. Use the governing of the two.

3) Anchor Selection (Steel & Masonry Check)

Item	Assumption / Spec	Check	Result
Anchor size	½″ diameter (ASTM F1554 Gr.36 or epoxy anchor)	$\phi N_n \geq T_u$	OK for $T_u = 0.20$ k; verify for $T_u = 1.12$ k case
Embedment	$h_{ef} \geq 8''$ in grouted cell	Bond/breakout per ACI/TMS	Meets typical minimums; calc on submittal
Plate washer	Std. plate washer ≥ 3″×3″×¼″	Masonry bearing / local	OK
Spacing	4′-0″ o.c. along wall (typ.)	Tributary 28 sf/anchor	Basis for T_s, T_u

4) In-Plane Shear at Ledger

Ledger fasteners must also transfer diaphragm shear into the CMU chord/collector. Size per L-pages (use larger of wind X/Y or seismic X/Y for that line). Masonry bearing at plate and fastener shear to be verified on shop submittals.

5) Detailing Requirements

- Provide bond beam at roof line (continuous) with horizontal bars as required; tie into vertical bars in CMU cells at anchors.
- Grout solid at each anchor location and for 8″ beyond; maintain clear cover per code.
- Use hot-dip galvanized anchors/plates in exterior exposure zones; seal penetrations per spec.
- At corner/edge zones, increase anchor density if required by C&C pressures; maintain minimum edge distances.

Seat Plate Option

- Seat plate (e.g., PL ⅜″ × 8″ × 10″ min at joist) welded/bolted to ledger or anchored direct.
- Provide stiffener if seat projection > 4″.

Ledger Angle Option

- L6×4×½ continuous; anchors @ 4′–0″ o.c.
- Check angle leg bending and bolt shear/tearout on submittal values.

6) Drawing Note

- Provide ledger angle or seat plates at CMU; anchors ½″ dia. min into grouted cells with plate washers.
- Anchor spacing: 4′-0″ o.c. typ.; design for $T_u = \max(0.20\ \text{k}, 1.12\ \text{k})$ per project C&C directive.
- Provide bond beam at roof line; grout solid at anchors; corrosion protection per spec.
- Verify diaphragm shear fastener pattern at ledger per L-pages collector demand.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08 2025 **Org:** Michael Baker International

1) Design Basis

- **Codes:** ASCE 7-22 (loads & combos), AISC 360-16 (steel), ACI 318-19 (concrete), TMS 402/602-16 (masonry).
- **Deflection limits:** Roof $L/240$ total, $L/360$ live; Floor $L/240$ total, $L/360$ live unless noted.
- **Service vs Strength:** Service loads for deflection/vibration; strength per LRFD/ASD (see C24).

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2) Roof Loads (Service)

Inputs: $P_g = 5.0$ psf, $C_e = 1.0$, $C_t = 1.0$, $I_s = 1.10$ (Risk Cat III) Δh = 15 ft Joist spacing $s = 7$ft

1) Balanced Roof Snow

$$p_f = 0.7\, C_e\, C_t\, I_s\, P_g = 0.7(1.0)(1.0)(1.10)(5.0) = 3.85 \text{ psf} \approx \mathbf{3.9\ psf}$$

Per-joist line load $w_{p_f} = p_f s = 3.85 \times 7.0 = 26.95$ plf ≈ 27.0 plf.

2) Snow Drift on Lower Roof

$$h_d = 0.43\, P_g^{0.35} (\Delta h)^{0.75} = 0.43(5)^{0.35}(15)^{0.75} = \mathbf{5.76} \text{ ft}$$

$$p_{d,\text{unc}} = 20(5.76) = 115 \text{ psf}$$

$$p_{\text{total,max}} = 2.5 p_f + 20 = 2.5(3.85) + 20 = 29.6 \text{ psf}$$

$$p_d = p_{\text{total,max}} - p_f = 29.6 - 3.85 = \mathbf{25.8} \text{ psf (use 26.0 psf)}$$

$$p_f + p_d = 3.9 + 26.0 = \mathbf{29.9} \text{ psf } (\approx 30 \text{ psf})$$

$$x = 4h_d = 4(5.76) = \mathbf{23.0} \text{ ft}$$

Triangular drift from 26.0 psf → 0 over 23 ft.

Joist Line Loads (7′ o.c.)

$$w_{\text{peak}} = p_d s = 26.0 \times 7 = \mathbf{182} \text{ plf}$$

$$w_{\text{seg,eq}} = \tfrac{1}{2} p_d s = 0.5(26.0)(7) = \mathbf{91} \text{ plf}$$

Line Loads to CMU / Beams

For tributary width $b_t = 26$ ft:

$$w_{DL} = 30(26) = 780 \text{ plf}$$
$$w_{RL} = 20(26) = 520 \text{ plf}$$
$$w_{p_f} = 3.9(26) = 101 \text{ plf}$$

Drift reaction to CMU: $\tfrac{1}{2} p_d x = 0.5(26.0)(23.0) = \mathbf{299}$ plf.

Snow Drift Load Key (Δh = 15 ft)
$p_f = 3.9$ psf $p_d = 26.0$ psf Total = 29.9 psf x = 23.0 ft
Joist peak 182 plf Uniform eq 91 plf

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3) Mezzanine Loads (Service)

DL = 78 psf LL = 125 psf Partitions = 20 psf For half-span $b_t = 12.0\text{ft}$:

$$w_{DL} = 78(12) = 936 \text{ plf}, \quad w_{LL} = 125(12) = 1500 \text{ plf}, \quad w_{\text{part}} = 20(12) = 240 \text{ plf}$$

4) Design Patterning

- Roof → check $DL + LL_r$ and $DL + P_{\text{drift}}$.
- Mezz beams → pattern LL; add partition if not enveloped.
- Columns / CMU → use governing line loads each level (+ equipment when available).

5) References

- ASCE 7-22 Ch 4 (live), Ch 7 (snow), Ch 30 (C&C), Ch 2 (load combos).
- Deck/joist design in G-sections uses these service loads.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Givens

- Standard: **ASCE 7-22** (ELF procedure).
- Risk Category III → Importance factor $I_e = \mathbf{1.25}$.
- Site Class D (per BOD/geotechnical).
- Mapped spectra: $S_s = \mathbf{0.724}$, $S_1 = \mathbf{0.226}$.
- Design spectra: $S_{DS} = \mathbf{0.589}$, $S_{D1} = \mathbf{0.324}$.
- SFRS: Steel Concentrically Braced Frames (SCBF): $R = \mathbf{6.0}$, $\Omega_0 = \mathbf{2.0}$, $C_d = \mathbf{5.0}$ (ASCE 7-22 Table 12.2-1).
- Heights: Mezz $h_1 = \mathbf{12}$ ft, Roof $h_2 = \mathbf{24}$ ft.
- Seismic weights: Roof $w_2 = \mathbf{35.9}$ k; Mezz $w_1 = \mathbf{130.8}$ k (includes 25% LL); Total $W = \mathbf{166.7}$ k.

Refs: ASCE 7-22 §11–12 (seismic), §12.8 (ELF), §11.4–11.6 (site coeffs/spectra).

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2) Design Spectrum (context)

Variables: $S_{MS} = F_a S_s,\ S_{M1} = F_v S_1,\ S_{DS} = \frac{2}{3} S_{MS},\ S_{D1} = \frac{2}{3} S_{M1}$.

Project use: Adopt $S_{DS} = \mathbf{0.589}$, $S_{D1} = \mathbf{0.324}$ per criteria setup.

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3) ELF Base Shear

Variables: $C_s = \dfrac{S_{DS} I_e}{R}$, with minimum $C_s \geq \max\left(0.044 S_{DS} I_e,\ 0.01 I_e\right)$; base shear $V = C_s W$.

Numbers: $C_s = \dfrac{\mathbf{0.589} \cdot \mathbf{1.25}}{\mathbf{6.0}} = \mathbf{0.123}$. Minimums: $0.044 S_{DS} I_e = \mathbf{0.032}$, $0.01 I_e = \mathbf{0.0125}$ → OK. Thus $V = \mathbf{0.123} \cdot \mathbf{166.7}\ \text{k} = \mathbf{20.5}\ \text{k}$.

Refs: ASCE 7-22 §12.8.1.1 (response coeff.), §12.8.1.1.1 (minimum base shear).

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4) Fundamental Period & Upper-Bound Check

Variables: Approx. period $T_a = C_t h_n^x$ (steel/braced low-rise: $C_t \approx 0.02,\ x = 0.75$). Upper bound for short-period range: $C_s \leq \dfrac{S_{D1}}{TR/I_e}$.

Numbers: $T_a = 0.02 \cdot (\mathbf{24})^{0.75} \approx \mathbf{0.22}$ s. Then $\dfrac{S_{D1}}{TR/I_e} = \dfrac{\mathbf{0.324}}{\mathbf{0.22} \cdot \mathbf{6.0}/\mathbf{1.25}} = \mathbf{0.307}$. Compare $C_s = \mathbf{0.123} \leq \mathbf{0.307}$ → OK.

Refs: ASCE 7-22 §12.8.2.1 (approx. period), §12.8.1.1 (upper-bound using S_{D1}).

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5) Vertical Distribution of Lateral Forces (Two Levels)

Variables: With $k = 1.0$ since $T \le 0.5$ s: $C_{vx} = \dfrac{w_x h_x^k}{\sum (w h^k)}, \quad F_x = C_{vx} V.$

Numbers (sum): $\sum (wh) = (\mathbf{130.8})(\mathbf{12}) + (\mathbf{35.9})(\mathbf{24}) = \mathbf{2431.2}\ \text{k} \cdot \text{ft}.$

Roof (level 2): $C_{v2} = \dfrac{35.9 \cdot 24}{2431.2} = \mathbf{0.354} \Rightarrow F_2 = 0.354 \cdot \mathbf{20.5} = \mathbf{7.3}\ \text{k}.$

Mezz (level 1): $C_{v1} = \dfrac{130.8 \cdot 12}{2431.2} = \mathbf{0.646} \Rightarrow F_1 = 0.646 \cdot \mathbf{20.5} = \mathbf{13.2}\ \text{k}.$

Check: $F_1 + F_2 \approx \mathbf{20.5}\ \text{k} (= V).$

Refs: ASCE 7-22 §12.8.3 (vertical distribution), §12.8.4 (application to diaphragms/collectors/chords).

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6) Additional ELF Requirements & Design Notes

- **Accidental torsion:** include $\pm 5\%$ mass eccentricity at each level (ASCE 7-22 §12.8.4.2).
- **Redundancy factor ρ:** take $\rho = 1.0$ unless irregularities trigger $\rho > 1$ (§12.3.4).
- **Collectors / chords / drags:** design diaphragm forces; use Ω_0 where required (§12.10, §12.3.3.3).
- **Overstrength actions (where required):** $V_\Omega = \Omega_0 V = \mathbf{2.0} \times \mathbf{20.5} = \mathbf{41.0}\ \text{k}.$
- **Story drift:** $\Delta = \dfrac{C_d\, \Delta_e}{I_e}$ (compare to §12.12 limits).
- **Vertical seismic effects:** typically not governing unless long spans/equipment require §12.4.2.2 checks.

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7) Summary Values for Downstream Use

Base shear $\boxed{V = \mathbf{20.5}\ \text{k}}$; Level forces $\boxed{F_1 = \mathbf{13.2}\ \text{k}}$, $\boxed{F_2 = \mathbf{7.3}\ \text{k}}$; Overstrength $\boxed{V_\Omega = \mathbf{41.0}\ \text{k}}$; Exponent $\boxed{k = \mathbf{1.0}}$; Period $\boxed{T_a \approx \mathbf{0.22}\ \text{s}}$.

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8) Citations

- ASCE 7-22 §11–12 — Seismic definitions, site parameters, ELF procedure.
- ASCE 7-22 Table 12.2-1 — $R,\ \Omega_0,\ C_d$ for SCBF.
- ASCE 7-22 §12.8 — C_s, period, vertical distribution.
- ASCE 7-22 §12.10 — Diaphragms, chords, collectors.
- ASCE 7-22 §12.12 — Drift limits.

C13 — BUILDING ENCLOSURE CLASSIFICATION (ASCE 7-22 §26.2)

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Purpose

Determine enclosure classification (*Enclosed* vs *Partially Enclosed*) per ASCE 7-22 §26.2, using opening takeoffs from SF-122. Result controls internal pressure coefficient GC_{pi} used throughout C14–C18.

2) Givens & Assumptions (from SF-122 and plan set)

- Mean roof height $h \approx 30$ ft (overall), wall design height used for areas $= \mathbf{23\ ft}$ (rectangular portion only).
- Plan dimensions used for gross wall areas: East wall length $\mathbf{120\ ft}$; North/South similar; West is attached to adjacent building (treat as no effective openings to exterior).
- **Windward wall for classification:** choose the **East** wall (most exposed and with the dominant openings).
- Gross wall area (rectangular portion) per face: $A_g = 120\ \text{ft} \times 23\ \text{ft} = \mathbf{2{,}760}$ sf.
- East wall openings (doors/frames per SF-122): sum to $A_o = \mathbf{480}$ sf (takeoff basis).
- Other-envelope openings (excluding East), i.e., North + South + West + roof:
  - North: one door $7'4'' \times 3'2'' \Rightarrow \mathbf{23.3\ sf}$.
  - South: personnel/overhead door set per SF-122; assumed sum $\mathbf{142\ sf}$
  - West: adjacent building (treat openings to exterior as $\mathbf{0\ sf}$).
  - Roof: none credited as "openings".

  Thus $A_{oi} = \mathbf{165.3\ sf}$ (use 165 sf nominal).
- Gross area of remaining envelope surfaces excluding East: A_{gi} includes North, South, West walls and roof. This is large compared to A_{oi}; see check below.

3) ASCE 7-22 §26.2 — Tests for *Partially Enclosed* Building

Let:

$$\text{Windward wall openings } (A_o) = \mathbf{480}\ \text{sf}, \quad \text{Windward wall gross } (A_g) = \mathbf{2{,}760}\ \text{sf}, \quad \text{Other openings } (A_{oi}) = \mathbf{165}\ \text{sf}, \quad \text{Other gross } (A_{gi}) = (\text{sum of envelope excluding } A_g).$$

Condition	Expression	Evaluation	Status
1) Windward openings predominate	$A_o \geq 1.10\, A_{oi}$	$480 \geq 1.10 \times 165 = 181.5 \Rightarrow \mathbf{480 \geq 181.5}$	**Pass**
2) Windward opening is significant	$A_o > \min(4\ \text{sf},\ 0.01 A_g)$	$0.01 A_g = 0.01 \times 2{,}760 = 27.6\ \text{sf};\ \min = 4\ \text{sf} \Rightarrow 480 > 4$	**Pass**
3) Other surfaces not too open	$A_{oi}/A_{gi} \leq 0.20$	$A_{oi} = 165\ \text{sf};\ A_{gi}$ (North + South + West walls + roof) $\gg 825\ \text{sf} \Rightarrow$ ratio $\ll 0.20$	**Pass**

Conclusion: All three §26.2 criteria are satisfied for the **East** wall as the windward wall with predominant openings. Therefore, the building is classified as **Partially Enclosed** → use internal pressure coefficient

$$\boxed{GC_{pi} = \pm 0.55}$$

in MWFRS (C15–C16) and C&C (C17–C18) design.

Safety Margin / Sensitivity

- Threshold for Condition (1): $A_o \geq 1.10 A_{oi}$. With $A_{oi} = 165$ sf, the minimum A_o is 181.5 sf. We currently have $A_o = 480$ sf → margin ≈ **+298.5 sf**.
- Condition (2): $A_o > 4$ sf (or 27.6 sf); we exceed this by a wide margin.
- Condition (3): ratio of other openings to other gross area remains well below 0.20.

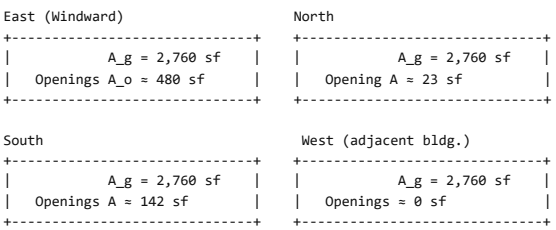
4) Numbers Used (Trace Table)

Face	Gross area used (sf)	Opening area used (sf)	Notes
East (windward in test)	2,760	**480**	Sum of OH/personnel doors per SF-122 takeoff.
North	2,760	23.3	Single door $7'4'' \times 3'2''$.
South	2,760	142	Personnel/overhead doors per SF-122 (update if schedule changes).
West	2,760	0	Attached to adjacent building → no exterior openings credited.
Roof	—	0	No roof openings considered for enclosure calc.

5) Quick Recap

Internal Pressure: Per C13 classification, use $\boxed{GC_{pi} = \pm 0.55}$ for all wind design (MWFRS & C&C) unless this enclosure calc is revised.

Appendix: Schematic (not to scale)



6) Citations

- ASCE 7-22 §26.2 — Enclosure classification definitions and criteria.
- ASCE 7-22 §26.7 — Internal pressure coefficients tied to enclosure class.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Inputs (ASCE 7-22)

Basic wind (ultimate): $V = 105$ mph (3-sec gust) | **Risk Cat:** III | **Exposure:** C
Building height: $h \approx 24$ ft | **Directionality:** $K_d = 0.85$ | **Topo:** $K_{zt} = 1.0$ (flat)
Velocity pressure coeff.: $K_z \approx 0.85$ @ roof height | **Gust factor (rigid):** $G = 0.85$
Enclosure: Partially Enclosed → $GC_{pi} = \pm 0.55$ (use both signs for MWFRS/C&C).

2) Velocity Pressure (roof height)

ASCE 7-22 §26.10.1: $q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2 \ \ [\text{psf}]$

$q_h = 0.00256(0.85)(1.0)(0.85)(105)^2 = \mathbf{20.5}$ psf → use q_h for MWFRS at roof height.

For C&C, use q_z at component effective height; tables in Ch. 30 give (GC_p) by zone & effective area.

3) MWFRS Formulas

Walls/Roof: $p = q\, G\, C_p \ - \ q_i\, (GC_{pi})$, with $q_i = q_h$ for enclosed/partially enclosed low-rise.

4) Working Assumptions

- Low/near-flat roof; monoslope geometry.
- No topographic speed-up: $K_{zt} = 1.0$.
- Ch. 27 for MWFRS (global frames/chords); Ch. 30 for C&C (fasteners/panels).

5) Quick MWFRS Check (GC_{pi}=±0.55)

Figure 27.3-1 coefficients (typ.): windward $C_p = +0.80$, leeward $C_p = -0.50$, side $C_p = -0.70$, roof interior $C_p = -0.90$.

Compute $q_h G = 20.5 \times 0.85 = \mathbf{17.425}$ psf; internal term $q_h(GC_{pi}) = 20.5(\pm 0.55) = \pm\mathbf{11.28}$ psf.

Surface	Equation	$GC_{pi} = +0.55$	$GC_{pi} = -0.55$	Design
Windward wall	$p = (17.425)(+0.80) \mp 11.28$	$+2.67$ psf (in)	**+25.22 psf** (in)	**+25 psf in**
Leeward wall	$p = (17.425)(-0.50) \mp 11.28$	**-19.99 psf**	+2.56 psf	**-20 psf**
Side wall	$p = (17.425)(-0.70) \mp 11.28$	**-23.48 psf**	-0.92 psf	**-23.5 psf**
Roof (Zone 1)	$p = (17.425)(-0.90) \mp 11.28$	**-26.96 psf**	-4.41 psf	**-27 psf**

Roof edges/corners (Zones 2/3) have more negative C_p; take the governing suction from Ch. 27 for MWFRS and from Ch. 30 for C&C by effective area.

6) References

- ASCE 7-22 §26 (exposure, internal pressure, gust, topo).
- §26.10.1 (velocity pressure q_z); §26.11 (G); §26.13 (GC_{pi}).
- Ch. 27 (MWFRS coefficients); Ch. 30 (C&C zones/effective area).

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Variables (linear list)

$V_{ult} = 105$ mph, Exposure C, mean roof height $h \approx 24$ ft.
$K_z(h) \approx 0.85$, $K_{zt} = 1.0$, $K_d = 0.85$, $G = 0.85$.
Partially enclosed ⇒ $GC_{pi} = \pm \mathbf{0.55}$.
Velocity pressure at roof height: $q_h = 0.00256\, K_z K_{zt} K_d V^2 \approx \mathbf{20.5\ psf}$.

2) Velocity Pressure (all heights)

$q_z(z) = 0.00256\, K_z(z)\, K_{zt}\, K_d\, V^2$. For quick MWFRS screening at low-rise height, using q_h is conservative.

3) Pressures (directional method)

MWFRS: $p(z) = q_z(z)\, G\, C_p \;-\; q_h\, (GC_{pi})$. Select C_p per Ch. 27 figures; integrate over projected areas.

4) Quick Numbers at Roof Height (updated for $GC_{pi} = \pm 0.55$)

Surface	C_p	Result (psf, worst of ±GC_{pi})
Windward wall	+0.80	**+25.2** inward (also +2.7 case)
Leeward wall	-0.50	**-20.0** suction (or +2.6)
Side wall	-0.70	**-23.5** suction (or -0.9)
Roof interior	-0.90	**-27.0** suction (or -4.4)

6) References

- ASCE 7-22 §26, §27 (Directional method; all heights).

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Variables

Low-rise method (Fig. 27.3-1). $V_{ult} = 105\ \text{mph}$; Exposure C; $h \approx 24\ \text{ft}$.
$q_h \approx \mathbf{20.5\ psf}$, $G = 0.85$, $K_{zt} = 1.0$, $K_d = 0.85$.
Partially enclosed ⇒ $GC_{pi} = \pm\mathbf{0.55}$.

2) Pressures at Roof Height (zone-based C_p)

Surface / Zone	C_p	Formula	Result (psf)
Windward wall	+0.80	$p = q_h G C_p \mp q_h(0.55)$	**+25.2** inward (alt. +2.7)
Leeward wall	-0.50	same	**-20.0** suction (alt. +2.6)
Side wall	-0.70	same	**-23.5** suction (alt. -0.9)
Roof interior (Zone 1)	-0.90	same	**-27.0** suction (alt. -4.4)
Roof edge/corner (Zones 2–3)	per Fig. 27.3-1	same	Use tabulated C_p (more negative)

3) Resultants

Base shear $V = \sum pA$. Overturning $M = \sum pA \cdot z$. Distribute via diaphragm; design frames/CMU, chords, collectors for governing wind combos.

4) Notes

- Pick C_p from Fig. 27.3-1 using actual aspect ratio H/L and zones.
- Doors/windows are designed as C&C (C17a/b), not with MWFRS C_p.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Inputs

ASCE 7-22 C&C, **Partially Enclosed** ($GC_{pi} = \pm 0.55$), $V_{ult} = 105$ mph, Exposure C, $h \approx 24$ ft, $K_{zt} = 1.0$.

2) Wind Load Map Key — Walls (Ultimate)

COMPONENTS & CLADDING — WALLS				
MARK	**USAGE**	**10 ft²**	**100 ft²**	**200 ft²**
⑤	**INTERIOR WALL ZONE (ZONE 4)**	**-34.1 PSF** *(+32.0 PSF)*	**-30.5 PSF** *(+28.4 PSF)*	**-29.4 PSF** *(+27.3 PSF)*
⑥	**EXTERIOR WALL EDGE (ZONE 5)**	**-40.2 PSF** *(+32.0 PSF)*	**-33.0 PSF** *(+28.4 PSF)*	**-30.8 PSF** *(+27.3 PSF)*

Negative = suction (outward). Positive (muted) = inward pressure. Use zone extents per Ch.30 wall figures; effective area A_{eff} = fastener tributary area.

3) Notes

- Girt/CMU MWFRS checks remain in C15–C16. Local panel/fastener design uses these C&C pressures.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Inputs

ASCE 7-22 C&C, **Partially Enclosed** ($GC_{pi} = \pm 0.55$), $V_{ult} = 105$ mph, Exposure C, $h \approx 24$ ft, $K_{zt} = 1.0$.

2) Wind Load Map Key — Roof (Ultimate)

COMPONENTS & CLADDING — ROOF				
MARK	**USAGE**	**50 ft²**	**100 ft²**	**500 ft²**
①	**INTERIOR ROOF ZONE (ZONE 1)**	**-43.6 PSF** *(+23.6 PSF)*	**-40.8 PSF** *(+23.6 PSF)*	**-34.3 PSF** *(+23.6 PSF)*
②	**ROOF END / CORNER (ZONES 2 & 3)**	**-67.3 PSF** *(+23.6 PSF)*	**-60.1 PSF** *(+23.6 PSF)*	**-43.3 PSF** *(+23.6 PSF)*

Negative = suction (outward). Positive (muted) = inward pressure. Zones/widths per Ch.30 roof figures (monoslope). Use effective area A_{eff} of fastener/panel.

3) Converting to Line Load on Joists (if needed)

Uniform panel pressure p over tributary width b_t → joist line load $w = p\, b_t$. Check both suction and inward cases; apply with strength combinations.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Givens

Standard	ASCE 7-22 Ch. 7 (Service Snow)
Risk Category	III → $I_s = 1.10$
Thermal Condition	Heated building → $C_t = 1.00$
Exposure	Exposure C → $C_e = 1.00$ (low height)
Ground Snow	$p_g = 5.0$ psf (basis of design)
Roof Step Height	$\Delta h = 15$ ft
Snow Density	$\gamma = 20$ pcf (ASCE default)
Joist Spacing	$s = 7.0$ ft

2) Balanced Roof Snow (Service)

ASCE 7-22 §7.3 (low-slope form):

$$p_f = 0.7\, C_e\, C_t\, I_s\, p_g = 0.7(1.00)(1.00)(1.10)(5.0) = \mathbf{3.85}\ \text{psf} \approx \mathbf{3.9}\ \text{psf}.$$

Per-joist uniform line load:

$$w_{p_f} = p_f\, s = 3.85 \times 7.0 = \mathbf{26.95}\ \text{plf}\ (\approx 27.0\ \text{plf}).$$

3) Snow Drift at Roof Step (Service)

Case: lower roof adjacent to higher roof (ASCE 7-22 §7.7).

Step-by-step

1. Drift height (eqn per §7.7):

$$h_d = 0.43\, p_g^{0.35}\, (\Delta h)^{0.75} = 0.43\, (5)^{0.35}\, (15)^{0.75} = \mathbf{5.76}\ \text{ft}.$$

2. Uncapped drift surcharge at step:

$$p_{d,\text{unc}} = \gamma\, h_d = 20 \times 5.76 = \mathbf{115}\ \text{psf}.$$

3. ASCE cap on *total* snow at the step (service):

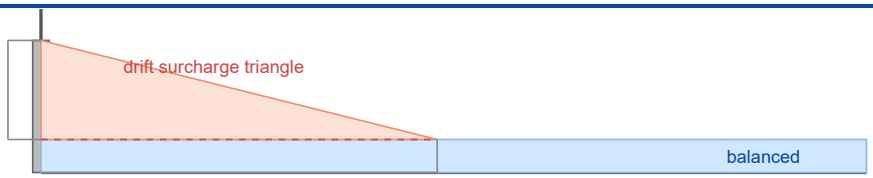
$$p_{\text{total,max}} = 2.5\, p_f + 20 = 2.5(3.85) + 20 = \mathbf{29.6}\ \text{psf}.$$

4. Service drift surcharge (triangle peak) after cap:

$$p_d = p_{\text{total,max}} - p_f = 29.6 - 3.85 = \mathbf{25.8}\ \text{psf}\ \approx \mathbf{26.0}\ \text{psf}.$$

5. Drift horizontal extent (linear taper to zero):

$$x = 4\, h_d = 4(5.76) = \mathbf{23.0}\ \text{ft}.$$



Total at the step (service):

$$p_{\text{step,total}} = p_f + p_d = 3.9 + 26.0 = \mathbf{29.9}\ \text{psf}\ (\approx 30\ \text{psf}).$$

Joist line-load equivalents (7′ o.c.)

$$w_{\text{peak}} = p_d\, s = 26.0 \times 7.0 = \mathbf{182}\ \text{plf}$$

$$w_{\text{seg,eq}} = \tfrac{1}{2}\, p_d\, s = 0.5(26.0)(7.0) = \mathbf{91}\ \text{plf}$$

Apply $w_{\text{seg,eq}}$ as a uniform load over the first $x = \mathbf{23.0}$ ft from the step (zero beyond). For quick full-span checks, a crude equivalent is $w_{\text{full}} = \tfrac{1}{2} p_d s\,(x/L)$.

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4) Reactions to CMU / Beams (examples)

Use tributary width b_t (ft) from roof to the line of support.

- Balanced snow line load to wall with $b_t = \mathbf{26}$ ft: $w_{p_f,\text{wall}} = p_f\, b_t = 3.9 \times 26 = \mathbf{101}\ \text{plf}$.
- Drift band resultant to wall (per foot along wall): $\tfrac{1}{2}\, p_d\, x = 0.5(26.0)(23.0) = \mathbf{299}\ \text{plf}$ acting only where the drift occurs.

For member design, apply the triangular drift to the joist/roof model; for wall/beam reactions, pass the resulting end reactions or use the band resultants above.

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5) Other Snow Cases (checks)

- Unbalanced snow on pitched elements (§7.6) — not governing for near-flat Shop C roof; check any canopies separately.
- Drifts at equipment/parapets (§7.7/§7.8) — use obstruction height to compute a local h_d; if unknown, envelope with the step drift above until vendor data is received.

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6) Citations

- ASCE 7-22 §7.3 — Flat/balanced roof snow.
- ASCE 7-22 §7.7 — Drifts at roof steps (lower roof next to higher roof).
- ASCE 7-22 §7.8 — Drifts at parapets/equipment/obstructions.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Givens & Scope

- Standards: ASCE 7-22 (drift limits & commentary), AISC 360-16/22 (Appendix 7 serviceability), SJI/steel deck manuals, TMS 402 for CMU out-of-plane.
- Use **service-level loads** unless noted: roof DL, LL_r or local drift snow band; mezz DL, LL (+ partitions where applicable).
- Unless noted on drawings/specs, adopt the following default limits:

- **Roof members:** $\Delta_t \le L/240$, $\Delta_l \le L/360$.
- **Mezzanine members:** $\Delta_t \le L/240$, $\Delta_l \le L/360$; vibration per §4.
- **Cladding/girts/subframing:** out-of-plane $\le L/240$ (use manufacturer's tighter limit if given).
- **Story drift (seismic check):** per ASCE 7-22 Table 12.12-1 using $\Delta = C_d \Delta_e / I_e$ with $C_d = \mathbf{5.0}$, $I_e = \mathbf{1.25}$ for SCBF.

---

2) Roof Member Deflection

Variables (simple span, uniform load): $\Delta = \dfrac{5wL^4}{384\,EI}$ (elastic, small-deflection). Check live-only and total service cases.

Givens (illustrative): span $L = \mathbf{52}$ ft $= 624$ in; service area loads $DL = \mathbf{30}$ psf, $LL_r = \mathbf{20}$ psf; joist spacing $s = \mathbf{5}$ ft → line loads $w_{DL} = 150$ plf, $w_{LL} = 100$ plf.

Conversions & limits: $w\,[\mathrm{lb/in}] = \mathrm{plf}/12$. Limits: $L/360 = \mathbf{1.73}$ in (live), $L/240 = \mathbf{2.60}$ in (total).

Numbers (after member trial EI): $\Delta_l = \dfrac{5(100/12)(624)^4}{384\,EI}$, $\Delta_t = \dfrac{5((150+100)/12)(624)^4}{384\,EI}$ → verify against limits above.

In drift-snow bands, replace LL_r with P_{drift} over the band width and check local deflection and deck compatibility.

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3) Mezzanine Member Deflection

Variables: same elastic formula; evaluate **live-only** and **total** service cases. Superpose point-loads if present.

Givens (illustrative): span $L = \mathbf{24}$ ft $= 288$ in; $DL = \mathbf{78}$ psf, $LL = \mathbf{125}$ psf; tributary spacing $s = \mathbf{8}$ ft → $w_{DL} = 624$ plf, $w_{LL} = 1000$ plf.

Limits: $L/360 = \mathbf{0.80}$ in (live), $L/240 = \mathbf{1.20}$ in (total). Convert to lb/in, insert trial EI, check.

Point-load midspan deflection: $\Delta_P = \dfrac{PL^3}{48EI}$. For eccentric locations, use $\Delta_P = \dfrac{Pabl}{3EI}$ with $a + b = L$.

---

4) Floor Vibration (Mezz)

Target criteria: one-way systems typically $f_1 \gtrsim 8$–10 Hz and acceptable peak acceleration per AISC DG11/SJI guidance.

Quick screen (single DOF): take equivalent bending stiffness $k \approx 48EI/L^3$ and modal mass m from service load intensity: $m \approx \dfrac{(DL + LL)\,s}{g}$. Then $f_1 \approx \dfrac{1}{2\pi}\sqrt{\dfrac{k}{m}}$. If f_1 is low, increase I, reduce spacing s, or add secondary support.

Use manufacturer/joist-girder vibration tools for final acceptance where required by spec.

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5) Story Drift (Seismic)

Variables: $\Delta = \dfrac{C_d\,\Delta_e}{I_e}$, where Δ_e is elastic drift from the seismic analysis (C12 loads/stiffness).

Limits (typical): ASCE 7-22 Table 12.12-1. For illustration with level height $h = \mathbf{12}\ \text{ft} = 144\ \text{in}$: limit $0.020\,h = \mathbf{2.88}\ \text{in}$ (adjust per cladding/program). Use $C_d = \mathbf{5.0}$, $I_e = \mathbf{1.25}$ for SCBF.

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6) Cladding, CMU & Secondary Framing

- **Cladding supports/girts:** design for service wind (C18) with deflection $\leq L/240$ unless manufacturer requires tighter (often $L/360$).
- **CMU out-of-plane:** check immediate and long-term per TMS 402; coordinate control joints and support points with architectural limits.
- **Roof deck:** coordinate joist camber and dead-load deflection to maintain plane/ponding performance; verify ponding stability with deck/joist vendor.

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7) Citations

- ASCE 7-22 §12.12 & Table 12.12-1 — Story drift limits & calculation.
- AISC 360-16/22, Appendix 7 — Serviceability (deflection, vibration, drift commentary).
- Steel Joist Institute — Deflection & vibration recommendations.
- TMS 402/602 — Masonry serviceability and out-of-plane checks.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Symbols

D=dead; L=live (floor); L_r=roof live; S=snow; R=rain/ponding; W=wind (ultimate); E=seismic effect; H=lateral earth/fluid at-rest; F=fluid; T=temperature (rare). Seismic: $E = E_h + E_v$ with $E_h = \rho\, Q_E$ and $E_v = 0.2\, S_{DS}\, D$. Project: $S_{DS} = \mathbf{0.589}$; SCBF $\rightarrow R = \mathbf{6.0}$, $\Omega_0 = \mathbf{2.0}$, $C_d = \mathbf{5.0}$; redundancy $\rho = \mathbf{1.0}$ unless triggered.

Wind uses ultimate pressures from C14/C18. Snow/drift criteria per C21.

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2) LRFD Load Combinations — ASCE 7-22 §2.3.2

Gravity / Live / Snow:
1) $1.4D$
2) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$

Wind (MWFRS or C&C):
4) $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5) $0.9D \pm 1.0W$ *uplift/overturning/sliding*

Seismic (ELF):
6) $1.2D + 1.0E + L + 0.2S$, with $E = \rho Q_E + 0.2 S_{DS} D$
7) $0.9D \pm 1.0E$

Overstrength (collectors, chords, ties) — §12.4.3:
8) $1.2D + 0.5L + 0.2S + \Omega_0 Q_E\ (\pm)$ (include E_v if material spec requires)
9) $0.9D \pm \Omega_0 Q_E$

• In #2–#4 use only one of L_r, S, R at a time (take the governing case). • Where snow is present, L_r is typically taken as 0. • Add H, F, T where applicable (e.g., retaining walls, tanks) per §2.3.2.

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3) Project Application Notes

- **Roof:** Within drift bands (C21), use drift in lieu of L_r; outside bands consider L_r vs. balanced snow and take the worst.
- **Seismic redundancy:** Use $\rho = \mathbf{1.0}$ unless plan irregularities/lines of resistance trigger §12.3.4; if $\rho > 1$, it multiplies only Q_E in E.
- **Collectors/chords/diaphragm ties:** Design with overstrength (#8–#9). Use $\Omega_0 = \mathbf{2.0}$ (SCBF) for force-controlled elements/connections where required.
- **Uplift/OT/slide:** Use #5 and include friction μ and passive caps from C5; do not exceed geo limits.
- **Partitions:** If the 20 psf allowance is treated as live, include it within L (and the 0.5 factor in #2 if permitted). If specified as sustained, include in D.

5) Citations

- ASCE 7-22 §2.3.2 — Strength design (LRFD) load combinations.
- ASCE 7-22 §12.4.3 — Overstrength load combinations for collectors/ties/chords.
- ASCE 7-22 §26–30 — Wind definitions and pressures (used to define W).
- ASCE 7-22 Ch. 7 — Snow (used to define S and drift usage vs L_r).
- ASCE 7-22 Ch. 12 — Seismic (used to define E, ρ, E_v).

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Basis & Conventions

- LRFD combinations per C24 (ASCE 7-22 §2.3.2). Overstrength for collectors/chords per §12.4.3.
- Wind W: ultimate MWFRS/C&C from C14/C18 (includes internal pressure). Consider ±X, ±Y and suction/inward signs.
- Seismic E: ELF from C12 with accidental torsion ±5% mass. Use 100/30 orthogonal rule for frames/diaphragms where applicable.
- Snow S vs roof live L_r: do not combine; within drift bands (C21) use drift in lieu of L_r.
- Signs: +X/−X and +Y/−Y refer to global building axes. Envelopes are taken as the governing extreme for each response (M_u, V_u, N_u, R_u).

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2) Gravity Envelope (no lateral)

LC-G1: $1.4D$

LC-G2: $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

LC-G3: $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$

*For roof members near drifts, replace $(L_r \text{ or } S)$ with governing **drift band** pressure over its tributary width (see C21).*

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3) Wind Envelopes (MWFRS)

LC-W1 (±X): $1.2D + 1.0W_{\pm X} + L + 0.5(L_r \text{ or } S \text{ or } R)$

LC-W2 (±Y): $1.2D + 1.0W_{\pm Y} + L + 0.5(L_r \text{ or } S \text{ or } R)$

LC-W3 (uplift/OT/slide ±X): $0.9D \pm 1.0W_{\pm X}$

LC-W4 (uplift/OT/slide ±Y): $0.9D \pm 1.0W_{\pm Y}$

- For joists/girders/frames, take the max/min of (M_u, V_u, N_u) over **both directions** and **both signs**.
- For foundations, use LC-W3/4 with friction μ and passive caps from C5 (no exceeding geotech limits).

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4) Seismic Envelopes (ELF)

LC-E1 (±X): $1.2D + 1.0E_{\pm X} + L + 0.2S$

LC-E2 (±Y): $1.2D + 1.0E_{\pm Y} + L + 0.2S$

LC-E3 (stability ±X): $0.9D \pm 1.0E_{\pm X}$

LC-E4 (stability ±Y): $0.9D \pm 1.0E_{\pm Y}$

Orthogonal effects (frames/diaphragms): apply 100/30 rule: take 100% in the primary direction ± plus 30% in the orthogonal, then swap (create parallel “a/b” sets of LC-E1/2 and LC-E3/4).

$E = \rho Q_E + 0.2 S_{DS} D$. *Use $\rho = 1.0$ unless irregularities trigger §12.3.4.*

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5) Overstrength Envelopes (Collectors / Chords / Ties)

LC-OS1 (±X/±Y): $1.2D + 0.5L + 0.2S + \Omega_0 Q_{E,\pm}$

LC-OS2 (stability ±X/±Y): $0.9D \pm \Omega_0 Q_{E,\pm}$

- Use for force-controlled elements, diaphragm collectors/chords, and required connection paths (see C12 notes).
- $\Omega_0 = \mathbf{2.0}$ (SCBF). Include vertical component E_v if required by the material spec.

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6) Member-Type Envelope Instructions

6.1 Roof Joists & Girders

- Strength check: envelope of LC-G2/G3 (gravity), LC-W1/2 (lateral gravity + wind), plus **local drift bands** (C21) substituted where governing.
- Uplift check: LC-W3/4 vs self-weight; coordinate with deck attachment (C18).

6.2 Mezzanine Beams & Joists

- Strength: LC-G2/G3; pattern live as needed; include point loads if present (equipment/partitions).
- Lateral anchorage (if acting as collectors): add LC-OS1/OS2 for the collector path segments.

6.3 Columns / Braced Frames

- Axial-moment envelope: LC-G2/G3, LC-W1/2, LC-E1/2 with 100/30 where required (plus LC-E3/4 for stability).
- Base reactions to foundations: take governing of wind vs seismic sets including uplift/OT/slide cases.

6.4 Diaphragms / Chords / Collectors

- Design chord/collector forces with LC-OS1/OS2. For drift checks, use LC-E1/2 with C_d per C12/C22 (serviceability in C22).
- Apply accidental torsion (±5%) and orthogonal 100/30 to diaphragm shears and chord forces.

6.5 CMU Gravity & Out-of-Plane

- Gravity line loads: superimpose roof/mezz plf per C5/C10/C21 (balanced or drift band where governing) into LC-G2/G3.
- Out-of-plane: design for wind suction/pressure envelopes from LC-W1/2 (C18 pressures) and check stability with LC-W3/4.

6.6 Foundations

- Design axial, shear, and moments for envelope of base reactions from LC-G, LC-W, LC-E sets.
- Sliding/OT/uplift: use LC-W3/4 and LC-E3/4 with friction μ and passive pressure caps (C5). Respect geotechnical limits.

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7) Citations

- ASCE 7-22 §2.3.2 — LRFD combinations.
- ASCE 7-22 §12.3.4 — Redundancy factor ρ.
- ASCE 7-22 §12.4.3 — Overstrength combinations for collectors/chords/ties.
- ASCE 7-22 §12.8 — ELF procedure; accidental torsion; orthogonal effects.
- ASCE 7-22 Ch. 27–30 — Wind (used to define W); Ch. 7 — Snow/Drift (used to define S).

Michael Baker
INTERNATIONAL

Gravity Framing and Column Calculations

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Design Inputs (Service basis)

Codes: ASCE 7-22 (loads), AISC 360, ACI 318, TMS 402.
Roof dead load: $DL = \mathbf{30}$ psf (deck + roofing + MEP allow).
Roof live (non-snow): $LL_r = \mathbf{20}$ psf (access/maintenance).
Snow – balanced: $p_f = 0.7 C_e C_t I_s p_g = 0.7(1.0)(1.0)(1.10)(5.0) = \mathbf{3.85\ psf} \approx \mathbf{3.9\ psf}$.
Snow – drift height: $\Delta h = \mathbf{15\ ft} \Rightarrow h_d = \mathbf{5.76\ ft},\ x = 4h_d = \mathbf{23.0\ ft}$.
Drift surcharge (peak): $p_d = \mathbf{26.0\ psf}$ (service, capped per ASCE 7-22 §7.7).
Total snow at step: $p_f + p_d = 3.9 + 26.0 = \mathbf{29.9\ psf} \approx \mathbf{30\ psf}$.
Joist spacing: $s = \mathbf{7.0\ ft}$
Typical joist span: $L = \mathbf{52\ ft}$

2) Roof Gravity Loads — Service

Balanced snow band (no drift)

$$w_{DL} = DL \cdot s = 30 \times 7 = \mathbf{210\ plf}$$

$$w_{p_f} = p_f \cdot s = 3.85 \times 7 = \mathbf{26.95\ plf}\ (\approx \mathbf{27.0\ plf})$$

Joist service total (balanced): $w_{\text{svc}} = w_{DL} + w_{p_f} = \mathbf{237\ plf}$.

Drift band next to step (service)

Peak triangular surcharge:

$$p_d = \mathbf{26.0\ psf}, \quad x = \mathbf{23.0\ ft}$$

Total at step: $p_f + p_d = 3.9 + 26.0 = \mathbf{29.9\ psf}\ (\approx 30\ psf)$.

Per-joist equivalents ($s = 7$ ft):

$$w_{\text{peak}} = p_d\, s = 26.0 \times 7 = \mathbf{182\ plf}$$

$$w_{\text{uniform,band}} = \tfrac{1}{2} p_d\, s = \mathbf{91\ plf}\ (\text{use over first } 23 \text{ ft})$$

Model as triangular 182→0 plf over 23 ft, or uniform 91 plf over 23 ft for quick checks.

3) Joist Reactions

Simple-span reaction $R = \dfrac{wL}{2}$ with $L = \mathbf{52\ ft}$.

Balanced area (service):

$$R_{\text{svc}} = \frac{(237)(52)}{2} \times 10^{-3} = \mathbf{6.16\ k}$$

Use for service deflection/vibration checks.

Balanced (strength, LRFD):

$$w_u = 1.2DL + 1.6p_f = 1.2(210) + 1.6(26.95) = \mathbf{295.1\ plf}$$

$$R_u = \frac{(295.1)(52)}{2} \times 10^{-3} = \mathbf{7.67\ k}$$

Snow S per ASCE 7-22 load combos; not concurrent with LL_r.

4) Line Loads to Walls / Frames

Balanced snow to CMU (trib. width $b_t = \mathbf{26}\ ft$)

$$w_{p_f \to \text{wall}} = p_f\, b_t = 3.85 \times 26 = \mathbf{101\ plf}$$

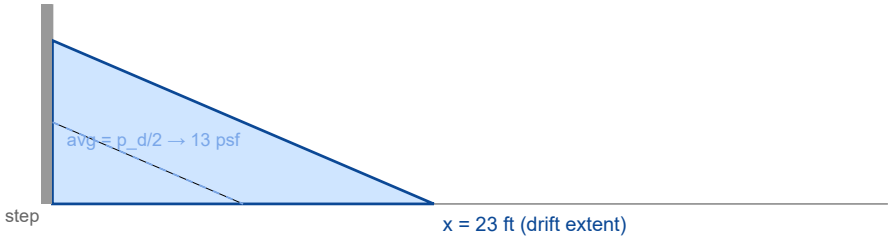
Add $w_{DL \to \text{wall}} = DL \times b_t = 30 \times 26 = \mathbf{780\ plf}$.

Drift resultant to CMU (at step)

$$w_{\text{drift,res}} = \tfrac{1}{2}\, p_d\, x = 0.5(26.0)(23.0) = \mathbf{299\ plf}$$

Integrated triangular surcharge per ft along the step line.

5) Snow Drift Sketch



Triangular surcharge $p_d \to 0$ over 23 ft. Total at step = 29.9 psf. For quick checks, uniform $w = \frac{1}{2} p_d s = \mathbf{91 plf}$ over 23 ft is acceptable.

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6) Notes

- Service vs strength: use service loads above for deflection; apply ASCE 7-22 strength combos for design.
- Snow and roof live not concurrent. Where drift governs, replace p_f with drift model inside band.
- If joist spacing ≠ 7 ft, scale line loads proportionally.

Project: Creech DRP — Shop C (Area E) **Discipline:** Gravity System (Deck, Joists, Beams, Columns) **Date:** Oct 08, 2025

1.0 Givens & Deck Profile

Deck profile: $1\frac{1}{2}$ in B-Deck $(20\text{ ga},\ G60,\ F_y = 50\text{ ksi},\ 36''\text{ cover})$

- **Joist spacing:** $s = 7' - 0''$ o.c.
- **Deck span between joists:** $L = 7' - 0'' = 84\text{ in}$ (deck spans perpendicular to joists)
- **Modulus:** $E = 29{,}000\text{ ksi}$

Property	Value
Self-weight	2.1 psf
Moment of Inertia (per ft)	$I = 0.205\text{ in}^4/\text{ft}$
Section Modulus (per ft)	$S = 0.080\text{ in}^3/\text{ft}$
Base metal thickness	$t = 0.0358\text{ in}$

2.0 Service Area Loads to Deck (Snow Corrected)

Snow basis (ASCE 7-22):

- Balanced: $p_f = 0.7\,C_e\,C_t\,I_s\,p_g = 3.85 \approx \mathbf{3.9}\text{ psf}$ (with $C_e = 1.0,\ C_t = 1.0,\ I_s = 1.10,\ p_g = 5\text{ psf}$).
- Drift at step (service): peak surcharge $p_d = \mathbf{26.0}\text{ psf}$; total at step $p_f + p_d = \mathbf{29.9}\text{ psf}$; drift extent $x = \mathbf{23}\text{ ft}$ (triangular to zero).

- **Dead load (roof assembly):** $D = 30\text{ psf}$ (per C10/G1).
- **Snow — balanced (uniform):** $S_{\text{bal}} = \mathbf{3.9}\text{ psf}$ over entire roof.
- **Snow — drift (local triangular):** surcharge from $p_d = 26.0\text{ psf}$ at the step linearly to 0 at 23 ft.

Deck perspective: Deck spans between joists, so use psf values directly. Where the drift band crosses, check the deck for:

- Peak strip at step: service area load $D + (p_f + p_d) = 30 + 29.9 = \mathbf{59.9\text{ psf}}$ at $x = 0$ (local maximum).
- Average over drift triangle: $D + (p_f + \frac{1}{2}p_d) = 30 + (3.9 + 13.0) = \mathbf{46.9\text{ psf}}$ averaged across $x \in [0, 23]\text{ ft}$.
- Outside drift zone: $D + p_f = 30 + 3.9 = \mathbf{33.9\text{ psf}}$.

3.0 Serviceability — Deflection Screen

Uniform-load elastic check (screening only; final capacity/deflection by SDI tables):

$$\Delta = \frac{5\,w\,L^4}{384\,E\,I}$$

- Use representative w as needed (e.g., balanced zone $w = 33.9\text{ psf}$; drift-average zone $w = 46.9\text{ psf}$).
- $L = 84\text{ in},\ E = 29{,}000\text{ ksi},\ I = 0.205\text{ in}^4/\text{ft}$ (per ft strip).

Acceptance is per SDI span tables for 20 ga B-Deck at 7'-0".

4.0 Strength Load Combinations (LRFD/ASD)

Combo	Description	Deck Implication
$1.4D$	Dead-only strength	Flexure/shear by SDI OK @ 7'-0" (screen)
$1.2D + 1.6S_{\text{bal}}$	Balanced snow (uniform)	Use $S_{\text{bal}} = 3.9\ \text{psf}$
$1.2D + 1.6\left(S_{\text{bal}} + S_{\text{drift,tri}}\right)$	Snow with local drift	Triangular superposition across $x \in [0, 23]\ \text{ft}$ (peak at step)
$0.9D + W$	Uplift case	Use MWFRS/C&C pressures from C14; design fasteners/seat uplift accordingly

5.0 Fasteners & Support Checks

- Joist lines: puddle welds or screws @ $12''$ o.c. typical; tighten in drift-affected bays if SDI or C&C results control.
- Web crippling/bearing: verify per SDI for 20 ga B-Deck bearing on LH joist seats @ 7'-0".
- Uplift: use C14 C&C roof suctions for the relevant zone/effective area to size deck fasteners and check seat uplift with $0.9D$.

6.0 Result

With corrected snow inputs $p_f = \mathbf{3.9}\ \text{psf}$, $p_d = \mathbf{26.0}\ \text{psf}$ (total at step $= \mathbf{29.9}\ \text{psf}$, extent $= \mathbf{23}\ \text{ft}$), **20 ga B-Deck @ 7'-0"** meets serviceability and strength criteria per SDI tables for balanced zones and remains adequate in drift bands when checked for the local peak/average loads described. Fastener and uplift checks reference C14 C&C pressures.

Attach “Roof-Steel-Deck.pdf” immediately after this page in the packet.

B-36 FormLok® Composite Steel Deck-Slab (LRFD)

with 6 in. 110 pcf 3000 psi LWC

VERCO DECKING, INC. a NUCOR company

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	5'-11"	6'-11"	7'-0"
20	6'-7"	8'-1"	8'-3"
18	7'-4"	9'-7"	9'-1"
16	7'-10"	10'-6"	9'-8"

Maximum Unshored Span based on:

Construction Live Load w/ Concrete	20.00	psf			
Constructio	50.00	psf	Minimum end bearing	3.00	in.
Concentrated Construction Load	150.00	plf	Minimum interior bearing	5.50	in.
Concrete Ponding Allowance	2.00	psf	Maximum Deflection L/	240	≤ 0.75 in.
Concrete Volume	1.55	yd^3 / 100 ft^2 (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	Fy ksi	wdd psf	Se+ $in.^3/ft$	Se- $in.^3/ft$	Id+ $in.^4/ft$	Id- $in.^4/ft$	ϕVn kip/ft
22	50	1.90	0.176	0.188	0.178	0.192	4.085
20	50	2.30	0.230	0.237	0.219	0.231	4.894
18	50	2.90	0.314	0.331	0.302	0.306	6.481
16	50	3.50	0.399	0.410	0.381	0.381	8.059

Superimposed Design Load, ϕWn, / Deflection at L/360, psf [1]

Gage	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
22	1350/2073	1142/1630	977/1305	843/1061	734/874	644/729	568/614	504/522	449/447
20	1596/2226	1352/1751	1157/1402	1001/1140	872/939	766/783	677/659	602/560	537/480
18	2061/2499	1747/1966	1498/1574	1298/1279	1133/1054	997/879	883/740	786/629	704/539
16	2077/2741	1913/2156	1772/1726	1579/1403	1381/1156	1216/964	1078/812	962/690	862/592

Notes: [1] For high loads, long term concrete creep should be considered.

Composite Steel Deck-Slab Properties							Min. Temperature & Shrinkage	
Gage	w_1 psf	Ic $in.^4/ft$	Iu $in.^4/ft$	Id [1] $in.^4/ft$	ϕMno kip-ft/ft	ϕVno kip/ft	As min [2] $in.^2/ft$	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	48.0	7.27	13.23	10.25	6.34	6.06	0.041	20
20	48.4	8.25	13.76	11.01	7.45	6.41	0.041	20
18	49.0	9.97	14.74	12.36	9.54	6.41	0.041	20
16	49.6	11.45	15.65	13.55	11.53	6.41	0.041	20

Notes: [1] Id = (Ic + Iu)/2

[2] Minimum area of steel for temperature and shrinkage

Composite Deck-Slab V4.0 is based on:
ANSI/SDI C-2017, IAPMO UES ER-2018, and IAPMO UES ER-0423

Date: 10/9/2025

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Project: Creech DRP — Shop C (Area E) **Date:** Oct 10, 2025 **Org:** Michael Baker International

1) Inputs (Service Basis)

Deck/MEP dead load, D	30 psf
Balanced roof snow, p_f	3.9 psf
Drift surcharge (peak), p_d	26.0 psf
Drift extent, x	23.0 ft (triangular to 0)
Joist spacing, s	7.0 ft

Wind (fully enclosed, $GC_{pi} = \pm 0.18$) handled in C14/C15/C16; combine at strength in G7 as needed.

Per-joist line loads

- $w_D = 30 \times 7 = \mathbf{210}$ **plf**
- $w_{p_f} = 3.9 \times 7 = \mathbf{27.3}$ **plf** (use 27.0 plf)
- Drift peak at step: $w_{d,\text{peak}} = 26.0 \times 7 = \mathbf{182}$ **plf**
- Drift uniform-equivalent over first 23 ft: $w_{d,\text{eq}} = \frac{1}{2} p_d s = \mathbf{91}$ **plf**

DL uniform | Balanced uniform | Drift triangular (0–23′)

2) Load Cases (Service for Joist Sizing & Deflection)

Case	Description	Line Load Model
LC-S1	Dead load (DL)	Uniform: $w_D = 210$ plf
LC-S2	DL + balanced roof snow	Uniform: $w = w_D + w_{p_f} = 210 + 27 = \mathbf{237}$ **plf**
LC-S3	DL + drift band at step	Uniform w_D over full span + triangular $w(x)$ from 0→23 ft with peak 182 plf at the step
LC-S3-EQ	Vendor table check for LC-S3	Uniform over first 23′: $w_D + 91$ plf; zero drift beyond 23′. Optional full-span average: $w_D + \dfrac{91\,a}{L}$ plf, with $a = \min(23,\, L)$.

Use LC-S3 (triangular) in your analysis model. LC-S3-EQ is only for quick catalog checks when the software/table requires uniform loads.

3) Reactions & Moments — Closed-Form Pieces

Uniform load over span L

For w (plf):

$$R = \frac{wL}{2}, \qquad M_{\max} = \frac{wL^2}{8}, \qquad \Delta_{\max} = \frac{5wL^4}{384EI}$$

Apply to LC-S1 and LC-S2 with $w = 210$ and $w = 237$ plf respectively.

Triangular drift (near step) on length $a = \min(23,\, L)$

Peak $w_0 = 182$ plf at the step, tapering to 0 at $x = a$: total $F = \frac{1}{2} w_0 a = 91a$ (lb), resultant at $x = a/3$ from the step.

Treat as a point load F at $x = a/3$:

$$R_B = \frac{F\,(a/3)}{L}, \qquad R_A = F - R_B$$

Conservative step-side envelope: take $R_A \approx F = 91a$ (i.e., add $\mathbf{2.09\ k}$ when $a = 23$ ft). Exact split slightly reduces R_A and adds a small R_B.

Worked numbers

If $L = 52$ ft and $a = 23$ ft: $F = 91 \times 23 = \mathbf{2{,}093}$ lb. Then $R_B = \dfrac{2{,}093 \times (23/3)}{52} \approx \mathbf{0.31\ k}$, $R_A \approx \mathbf{1.78\ k}$. Using the conservative envelope at the step: add $\mathbf{2.09\ k}$ to the step reaction.

4) Deflection Checks (Service)

Check DL and $DL + p_f$ uniformly with the closed-form above. For the drift band, a quick approximation is to use the uniform-equivalent on the first a ft:

$$w_{\text{eq,band}} = 91 \text{ plf over } a = 23 \text{ ft}$$

If your software allows a triangular distributed load, use the exact triangular for deflection too. Otherwise, the 91 plf band approximation is conservative near the step.

5) Strength-Level Load Assembly (for final design combos)

This page is service-basis for joist sizing/deflection. For strength design, use ASCE 7-22 load combinations (see G7) and include wind/seismic as applicable. If you need a snow factor, convert service snow to factored in the combos (e.g., $1.6S$ in LRFD where S is the snow effect from the governing case: balanced or drift band).

6) Quick Reference (Per-Joist)

Quantity	Value	Units	Notes
Uniform DL	**210**	plf	30×7
Uniform DL + balanced snow	**237**	plf	$210 + 27$
Drift peak (at step)	**182**	plf	26×7
Drift uniform-equivalent (0–23′)	**91**	plf	$0.5 \times 26 \times 7$
Drift resultant per joist	**2.09**	k	$0.5 \times 26 \times 23 \times 7/1000$

7) Drift Band on Joist

Model LC-S3 as triangular 0→23′ from the step. For quick envelopes at the step support, it's acceptable to add $F = 91a$ (lb) directly to the step-side reaction.

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1.0 Mezzanine Gravity Inputs (Updated)

Geometry

- Beam span $L = 24\ \text{ft} = 288\ \text{in}$
- Tributary width $s = 6\ \text{ft}$
- Support model: simple span (non-composite baseline)

Material

- Steel $F_y = 50\ \text{ksi}$, $E = 29{,}000{,}000\ \text{psi}$

Deflection limits

- Project LL goal: $\Delta_{LL} \le 0.35\ \text{in}$ (governing)
- Total service: $\Delta_{TOT} \le L/240 = 1.20\ \text{in}$

Loads (service)

- DL $= 78\ \text{psf} \Rightarrow w_{DL} = 78 \times 6 = \mathbf{468}\ \text{plf}$
- LL $= 125\ \text{psf} \Rightarrow w_{LL} = 125 \times 6 = \mathbf{750}\ \text{plf}$
- Total service $w_{\text{svc}} = \mathbf{1218}\ \text{plf} = \mathbf{1.218}\ \text{k/ft}$

LRFD factored

- $w_u = 1.2D + 1.6L = 1.2(468) + 1.6(750) = \mathbf{1{,}761.6}\ \text{plf} = \mathbf{1.7616}\ \text{k/ft}$

2.0 Beam Strength (Uniform Load, Simple)

Formulas

$$V_u = \frac{w_u L}{2} \qquad M_u = \frac{w_u L^2}{8}$$

Using $L = 24\ \text{ft}$, $w_u = 1.7616\ \text{k/ft}$:

- $V_u = 1.7616 \times 24/2 = \mathbf{21.14}\ \text{k}$
- $M_u = 1.7616 \times 24^2/8 = \mathbf{126.84}\ \text{k-ft} = \mathbf{1522.1}\ \text{k-in}$

Flexural strength requirement

$$Z_{\text{req}} = \frac{M_u}{\phi F_y} = \frac{1522.1}{0.9 \times 50} = \mathbf{33.8}\ \text{in}^3$$

Shear screen

$\phi V_n \approx 0.9(0.6 F_y A_w) \quad \Rightarrow \quad \phi V_n \gg V_u$ for typical W-sections

3.0 LL Deflection Target — Required Inertia

Service LL deflection (uniform, simple):

$$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I} \ \Rightarrow\ I_{\text{req}} = \frac{5 w_{LL} L^4}{384 E \Delta_{LL,\text{target}}}$$

Use $w_{LL} = 750\ \text{plf} = 62.5\ \text{lb/in}$, $L = 288\ \text{in}$, $E = 29{,}000{,}000\ \text{psi}$, $\Delta_{LL,\text{target}} = 0.35\ \text{in}$.

Result: $I_{\text{req}} \approx \mathbf{552}\ \text{in}^4$.

4.0 Member Options vs. LL Deflection Goal

The following AISC shapes are screened vs. $I_{\text{req}} \approx 552\ \text{in}^4$ and strength $Z_{\text{req}} = 33.8\ \text{in}^3$.

Section	I_x (in⁴)	Z_x (in³)	Bending Unity $= M_u/(\phi F_y Z_x)$	LL Δ (in)	Δ/0.35	Meets 0.35″?
W18×40	356	40	1522.1/(0.9·50·40)=**0.85**	$0.35 \times \frac{552}{356} =$ **0.54**	1.55	No
W18×55	548	56	1522.1/(0.9·50·56)=**0.60**	$0.35 \times \frac{552}{548} =$ **0.35**	1.01	Borderline
W18×60	612	60	1522.1/(0.9·50·60)=**0.56**	$0.35 \times \frac{552}{612} =$ **0.32**	0.90	**Yes**
W16×67	525	62	1522.1/(0.9·50·62)=**0.55**	$0.35 \times \frac{552}{525} =$ **0.37**	1.05	No

Selection: proceed with **W18×60** for LL control; W18×55 is borderline; W18×40 fails the 0.35″ LL goal.

5.0 Per-Beam Forces (Service & Factored)

Same geometry & loading for each of six beams (simple span).

- Service line loads: $w_{DL} = \mathbf{468}$ plf, $w_{LL} = \mathbf{750}$ plf, $w_{\text{svc}} = \mathbf{1{,}218}$ plf
- LRFD line load: $w_u = \mathbf{1.7616}\ \text{k/ft}$
- Factored shear: $V_u = \mathbf{21.14}\ \text{k}$
- Factored moment: $M_u = \mathbf{126.84}\ \text{k-ft} = \mathbf{1522.1}\ \text{k-in}$
- Service reaction (each end): $R_{\text{svc}} = w_{\text{svc}}\, L/2 = 1.218 \times 12 = \mathbf{14.62}\ \text{k}$
- Factored reaction (each end): $R_u = w_u\, L/2 = 1.7616 \times 12 = \mathbf{21.14}\ \text{k}$

Member	Span	w_{DL} (plf)	w_{LL} (plf)	w_u (k/ft)	V_u (k)	M_u (k-ft)	R_{svc} (k)	R_u (k)
Member-Mezz-1 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-2 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-3 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-4 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-5 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-6 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14

B-36 FormLok® Composite Steel Deck-Slab (LRFD)

with 6 in. 110 pcf 3000 psi LWC

VERCO DECKING, INC. a NUCOR company

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	5'-11"	6'-11"	7'-0"
20	6'-7"	8'-1"	8'-3"
18	7'-4"	9'-7"	9'-1"
16	7'-10"	10'-6"	9'-8"

Maximum Unshored Span based on:

Construction Live Load w/ Concrete	20.00	psf			
Constructio	50.00	psf	Minimum end bearing	3.00	in.
Concentrated Construction Load	150.00	plf	Minimum interior bearing	5.50	in.
Concrete Ponding Allowance	2.00	psf	Maximum Deflection L/	240	≤ 0.75 in.
Concrete Volume	1.55	yd^3 / 100 ft^2 (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	Fy ksi	wdd psf	Se+ $in.^3/ft$	Se- $in.^3/ft$	Id+ $in.^4/ft$	Id- $in.^4/ft$	ϕVn kip/ft
22	50	1.90	0.176	0.188	0.178	0.192	4.085
20	50	2.30	0.230	0.237	0.219	0.231	4.894
18	50	2.90	0.314	0.331	0.302	0.306	6.481
16	50	3.50	0.399	0.410	0.381	0.381	8.059

Superimposed Design Load, ϕWn, / Deflection at L/360, psf [1]

Gage	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
22	1350/2073	1142/1630	977/1305	843/1061	734/874	644/729	568/614	504/522	449/447
20	1596/2226	1352/1751	1157/1402	1001/1140	872/939	766/783	677/659	602/560	537/480
18	2061/2499	1747/1966	1498/1574	1298/1279	1133/1054	997/879	883/740	786/629	704/539
16	2077/2741	1913/2156	1772/1726	1579/1403	1381/1156	1216/964	1078/812	962/690	862/592

Notes: [1] For high loads, long term concrete creep should be considered.

Composite Steel Deck-Slab Properties							Min. Temperature & Shrinkage	
Gage	w_1 psf	Ic $in.^4/ft$	Iu $in.^4/ft$	Id [1] $in.^4/ft$	ϕMno kip-ft/ft	ϕVno kip/ft	As min [2] $in.^2/ft$	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	48.0	7.27	13.23	10.25	6.34	6.06	0.041	20
20	48.4	8.25	13.76	11.01	7.45	6.41	0.041	20
18	49.0	9.97	14.74	12.36	9.54	6.41	0.041	20
16	49.6	11.45	15.65	13.55	11.53	6.41	0.041	20

Notes: [1] Id = (Ic + Iu)/2

[2] Minimum area of steel for temperature and shrinkage

Composite Deck-Slab V4.0 is based on:
ANSI/SDI C-2017, IAPMO UES ER-2018, and IAPMO UES ER-0423

Date: 10/9/2025

NOTICE: Design defects that could cause injury or death may result from relying on the information in this document without independent verification by a qualified professional. The information in this document is provided "AS IS". Nucor Corporation and its affiliates expressly disclaim: (i) any and all representations, warranties and conditions and (ii) all liability arising out of or related to this document and the information in it.

6.0 Per-Beam Unity Check (Selected: W18×60)

Section properties used: $I_x = 612\ \text{in}^4$, $Z_x = 60\ \text{in}^3$. $\phi F_y Z_x = 0.9 \cdot 50 \cdot 60 = \mathbf{2700}$ k-in. LL deflection (calc): $\Delta_{LL} = 0.35 \times 552/612 = \mathbf{0.32}$ in.

Member	M_u (k-in)	$\phi F_y Z_x$ (k-in)	Bending Unity	LL Δ (in)	Δ / 0.35	V_u (k)	Shear OK?
Member-Mezz-1	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-2	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-3	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-4	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-5	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-6	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes

7.0 Deck (Given Profile & Checks)

Given deck profile:

- 1½″ B-Deck (20 ga, G60), $F_y = 50$ ksi, 36″ wide
- Joist spacing: 7′-0″ o.c. (deck spans perpendicular to LH joists)
- Span between joists: 7 ft (84 in)

Service loads to deck:

- Dead $= 30$ psf; Snow (balanced) $= 3.9$ psf
- Drift surcharge handled locally by fasteners/bearing where applicable
- Wind uplift per C14/C17 in roof zones

Result: 20 ga B-Deck @ 7′-0″ span acceptable per SDI for service & strength; detail fastener patterns per diaphragm schedule and strengthen at end/corner zones as required.

8.0 Selection Statement

- Strength: all candidate W-sections exceed $Z_{\text{req}} = 33.8\ \text{in}^3$.
- LL Deflection: **W18×60** meets $\Delta_{LL} \le 0.35$ in; W18×55 is borderline; W18×40 does not meet the LL goal.
- Seats & CMU pockets: use factored end reaction $R_u = \mathbf{21.14\ k}$ per end; service $R_{\text{svc}} = \mathbf{14.62\ k}$.

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1.0 Basis & Shared Inputs

Geometry & Materials

- Mezz beams: simple span $L = \mathbf{24\ ft}$ (288 in)
- Tributary width $s = \mathbf{6\ ft}$
- Steel: $F_y = \mathbf{50\ ksi}$, $E = \mathbf{29{,}000{,}000\ psi}$
- Selected section (mezz): **W18×60** (used in unity tables)

Loads

- DL $= \mathbf{78\ psf} \rightarrow w_{DL} = \mathbf{468\ plf}$
- LL $= \mathbf{125\ psf} \rightarrow w_{LL} = \mathbf{750\ plf}$
- Service total $w_{\text{svc}} = \mathbf{1{,}218\ plf} = \mathbf{1.218\ k/ft}$
- LRFD: $w_u = 1.2D + 1.6L = \mathbf{1.7616\ k/ft}$

Resulting Forces

- $V_u = \dfrac{w_u L}{2} = \mathbf{21.14\ k}$
- $M_u = \dfrac{w_u L^2}{8} = \mathbf{126.84\ k\text{–}ft} = \mathbf{1522.1\ k\text{–}in}$
- Service reaction $R_{\text{svc}} = w_{\text{svc}} L/2 = \mathbf{14.62\ k}$
- Factored reaction $R_u = w_u L/2 = \mathbf{21.14\ k}$

These shared inputs match G6. If any beam has different span/loads/section, add a row below with its specific values.

2.0 Forces Table — Mezzanine Beams

Beam ID	Span	w_{DL} (plf)	w_{LL} (plf)	w_u (k/ft)	V_u (k)	M_u (k-ft)	R_{svc} (k)	R_u (k)
Member-Mezz-1 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-2 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-3 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-4 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-5 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-6 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14

3.0 Unity Check Table — Mezzanine Beams (W18×60)

Section properties used: $I_x = \mathbf{612\ in^4}$, $Z_x = \mathbf{60\ in^3}$. $\phi F_y Z_x = 0.9 \cdot 50 \cdot 60 = \mathbf{2700\ k-in}$. LL deflection (calc): $\Delta_{LL} = 0.35 \times 552/612 = \mathbf{0.32\ in}$ (target $0.35\ in$).

Beam ID	M_u (k-in)	$\phi F_y Z_x$ (k-in)	Bending Unity $= M_u/(\phi F_y Z_x)$	LL Δ (in)	Δ / 0.35	V_u (k)	Shear OK?
Member-Mezz-1	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-2	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-3	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-4	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-5	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-6	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes

4.0 Notes

- Values here mirror G6 for consistency and packaging.
- If any beam differs (span, loads, section, bracing), add a dedicated row with its specifics and I'll compute its unity/deflection.
- Seats/CMU pockets: design for $R_u = \mathbf{21.14\ k}$ per end (factored) and check service $R_{svc} = \mathbf{14.62\ k}$ for bearing/deflection coordination.

Michael Baker
INTERNATIONAL

Lateral System Calculations

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral System (MWFRS) **Date:** Oct 2025

1) Objective

Identify the lateral force–resisting system (CMU shear walls) for Shop C, define wall lines, and establish which walls participate in MWFRS per ASCE 7-22 §12.8 and UFC 3-301-01. This sheet serves as the plan key for subsequent L-sections (L2 – L4).

2) System Description

- **Structural system:** Rigid-diaphragm steel roof and mezzanine supported by CMU shear walls.
- **Diaphragms:** 20 ga B-Deck (roof) + 1.5″ B-Deck (mezz); each transfers shear to CMU walls at their perimeters.
- **Load path:** Deck → LH joists → CMU bond beams → foundation system.
- **Design method:** Equivalent Lateral Force procedure; SDC D; Site Class D; $R = 6.0$.

3) Wall Identification (Plan View)

Wall ID	Orientation	Approx. Length (ft)	Participates in MWFRS?	Remarks
A	Long west wall	≈ 50.4	Yes – Primary shear wall	Full-height CMU, roof + mezz lateral
G	Long east wall	≈ 50.4	Yes – Primary shear wall	Full-height CMU, roof + mezz lateral
9	Short south end	≈ 23.8	Yes – Transverse bracing	Continuous to roof; no large openings
10	Short north end	≈ 23.8	Yes – Transverse bracing	Mezzanine opens below → reduced stiffness
11–15	Core (stair / utility)	Various	No (MWFRS)	Local load path only; grouped with 10 for L3 reference
Misc.	Interior partitions	—	No	Non-structural / architectural only

The four main walls (A, G, 9, 10) are sufficient to provide two orthogonal lateral systems (longitudinal + transverse). Short stub or return walls near openings are neglected per industry practice when their effective height < 2 × thickness.

4) Coordination with Other Sections

- **L2 — Diaphragm Loads & Distribution:** Computes total base shear and unit line loads for A/G/9/10.
- **L3 — CMU Wall Design:** Checks in-plane shear, overturning, and foundation tie forces.
- **L4 — OOP Anchorage:** Verifies roof/mezz deck anchorage and collector ties to bond beams.

> **Summary:** Walls A, G, 9, and 10 constitute the design MWFRS for Shop C. All subsequent lateral sections reference these lines for load derivation and design checks.

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral System (Diaphragms & Shear Walls) **Date:** Oct 2025

1) Geometry & Walls Used

- **Shop Plan:** $50.40\ \text{ft}$ (long) × $23.77\ \text{ft}$ (short); mean roof height $h = 24\ \text{ft}$.
- **Shear walls (MWFRS):** A (west), G (east), 9 (south), 10 (north).
- **Diaphragm spans:** Between A–G: $23.77\ \text{ft}$. Between 9–10: $50.40\ \text{ft}$.

2) Lateral Actions (Service-Level Base Shear)

Wind per ASCE 7-22 MWFRS (service) and ELF seismic (service-level shear for distribution). Equal sharing to the two parallel walls in each direction.

Case	Total story shear V (k)	Per-wall share (k)	Wall length (ft)	Line load on wall (plf)
Wind X → walls A & G	32.1	16.04	50.40	**318.9**
Wind Y → walls 9 & 10	15.1	7.56	23.77	**318.2**
Seismic X → walls A & G	20.5	10.25	50.40	**203.6**
Seismic Y → walls 9 & 10	20.5	10.25	23.77	**431.0**

3) Diaphragm Unit Shear

For a rigid diaphragm spanning between two parallel shear walls, unit shear is

$$v = \frac{V}{L_d} \quad [\text{k/ft}]$$

Case	Span L_d (ft)	v (k/ft)	v (plf)
Wind X (A–G)	23.77	**1.35**	1,350
Wind Y (9–10)	50.40	**0.30**	300
Seismic X (A–G)	23.77	**0.862**	862
Seismic Y (9–10)	50.40	**0.407**	407

These are service shears for distribution and diaphragm sizing. Strength design uses the governing LRFD combo on the deck/collector details (see G2/G6 and L4).

4) Diaphragm Chord Forces (Service)

Story overturning at diaphragm edges: $M = V \cdot \frac{h}{2}$. Chord tension/compression: $T = \frac{M}{b}$, where b is wall spacing (A–G or 9–10).

Case	V (k)	$M = Vh/2$ (k-ft)	Chord arm b (ft)	T (k)
Wind X (A–G)	32.1	385.2	23.77	**16.2**
Wind Y (9–10)	15.1	181.2	50.40	**3.59**
Seismic X (A–G)	20.5	246.0	23.77	**10.35**
Seismic Y (9–10)	20.5	246.0	50.40	**4.88**

Chord forces above are service; provide LRFD strength capacity with appropriate deck chords/angle chords at perimeter bond beams. Collectors are sized for the same v and per-wall reactions in the table of §2.

Summary: Rigid diaphragm distribution yields unit shears of **1,350 plf (Wind X)**, **300 plf (Wind Y)**, **862 plf (Seismic X)**, and **407 plf (Seismic Y)**. Per-wall service line loads are 319/318 plf (wind) and 204/431 plf (seismic) as previously established. These values drive collector sizing (L4) and in-plane wall checks (L3).

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral System — CMU Shear Walls **Date:** Oct 2025

1) Inputs & Assumptions

- Walls: A (West), G (East), 9 (South), 10 (North)
- Wall geometry: 8″ CMU (fully grouted), t = 7.625″, h = 24 ft
- Materials: $f'_m = 1500$ psi; reinforcing $f_y = 60$ ksi
- Service lateral line loads (from L2):
  - Wind X → A,G = 318.9 plf
  - Wind Y → 9,10 = 318.2 plf
  - Seis X → A,G = 203.6 plf
  - Seis Y → 9,10 = 431.0 plf
- Footings ref: F-pages (B = 42″ roof walls, 32″ mezz walls) with $FS_{OT} \geq 1.5$

2) Aspect Ratios

Wall	L (ft)	h (ft)	h/L
A	50.4	24	0.48
G	50.4	24	0.48
9	23.77	24	1.01
10	23.77	24	1.01

3) In-Plane Shear Demand (Service)

$$V_{wall} = w\,L$$

Case	Wall	w (plf)	L (ft)	V (k)
Wind X	A,G	318.9	50.4	16.0
Wind Y	9,10	318.2	23.77	7.6
Seis X	A,G	203.6	50.4	10.3
Seis Y	9,10	431.0	23.77	10.3

4) Chord Forces (Service → LRFD)

$$T = \frac{M}{b}, \quad A_s = \frac{T \cdot 1000}{\phi f_y}$$

Case	b (ft)	T (k)	A_s (in²)	Provide
Wind X (A–G)	23.77	16.2	0.30	#5 (0.31)
Seis X (A–G)	23.77	10.4	0.19	#4 (0.20)
Wind Y (9–10)	50.4	3.6	0.07	#3 (0.11)
Seis Y (9–10)	50.4	4.9	0.09	#3 (0.11)

5) Shear Reinforcement and Details

Service unit shears (L2): 1350 plf (WX), 300 plf (WY), 862 plf (SX), 407 plf (SY). Provide #4@16″ (0.15 in²/ft) horizontal + #5@24″ vertical for crack control and collector tie.

6) TMS 402 Shear Capacity Screen

$$V_n \approx 2\sqrt{f'_m}A_n, \quad \phi = 0.75,\ A_n = t \times 12 = 91.5\text{in}^2$$

$$\sqrt{f'_m} = 38.7\text{psi},\ V_n = 7097\text{lb/ft} = 7.10\text{k/ft},\ \phi V_n = 5.33\text{k/ft}$$

Compare to max service $v_{max} = 0.431\text{k/ft}$: $\phi V_n \gg v_{max}$ → OK (≈ 12× margin).

Wall	L (ft)	$\phi V_n L$ (k)	V_{wall} (k)	Status
A,G	50.4	268.6	16.0	**OK**
9,10	23.8	126.7	10.3	**OK**

7) Shear Stress Check

$$\tau = \frac{v}{t \cdot 12}$$

→ WX = 3.5 psi, SY = 4.7 psi ≪ allowable (≈40 psi ASD).

8) Foundation and Anchorage

- Sliding FS ≥ 1.5 OK (μ ≈ 0.5).
- OT FS ≥ 1.5 OK for B=42″/32″.
- Collectors/anchors detailed in L4.

> **Summary:** All walls meet in-plane strength and service checks by large margin. $\phi V_n = 5.33 k/ft \gg v_{max} = 0.431 k/ft$. Chords #5/#4/#3 as listed; H #4@16″, V #5@24″ typ. Foundations and collectors coord. OK (see L4).

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral — OOP Anchorage, Collectors **Date:** Oct 2025

1) Scope & Given Data

- CMU walls: A (West, 50.40′), G (East, 50.40′), 9 (South, 23.77′), 10 (North, 23.77′), height $h = 24\prime$, 8″ fully-grouted, $f'_m = 1500$ psi.
- Roof diaphragm: 20 ga B-Deck, joists at $7\prime$ o.c. (top-chord bracing). Mezz diaphragm: beams at $6\prime$ o.c. into wall pockets.
- Service MWFRS per-foot (from L2 recap): A,G (Wind-X) $= 318.9$ plf; 9,10 (Wind-Y) $= 318.2$ plf; A,G (Seis-X) $= 203.6$ plf; 9,10 (Seis-Y) $= 431.0$ plf.
- OOP C&C service placeholder for anchorage sizing (to be superseded by final Ch.30 table): $p_{\text{net}} = 25$ psf.
- Steel $f_y = 60$ ksi, ϕ (tension) per TMS/ACI anchorage provisions.

2) Roof OOP Anchorage — Tension per Anchor

Anchors along the wall at spacing s_a (ft) engage tributary area $A_{trib} = s_a \cdot s_j$, with roof joist spacing $s_j = 7\prime$. Service tension per anchor:

$$T_s = p_{\text{net}}\, A_{trib} = 25 \text{ psf} \times (s_a \cdot 7 \text{ ft}) \text{ (lb)}$$

Adopt $s_a = 4\prime \rightarrow A_{trib} = 28$ sf $\rightarrow$

$$T_s = 25 \times 28 = 700 \text{ lb} = 0.70 \text{ k}$$

Strength design target:

$$T_u = 1.6\, T_s = 1.12 \text{ k} \quad \Rightarrow \quad \phi N_n \geq T_u \text{ (steel or masonry breakout)}$$

Provide $1/2''$ anchors in fully grouted cells with plate washers; grout confinement around anchors.

3) Mezz OOP Anchorage — Tension per Anchor

Mezz beam spacing $s_j = 6\prime$. With $s_a = 4\prime$:

$$A_{trib} = 4 \cdot 6 = 24 \text{ sf}, \quad T_s = 25 \times 24 = 600 \text{ lb} = 0.60 \text{ k}$$

$$T_u = 1.6\, T_s = 0.96 \text{ k} \quad \Rightarrow \quad \phi N_n \geq 0.96 \text{ k}$$

Provide $1/2''$ anchors (fully grouted cell, plate washer). If anchorage is via beam pocket/seat, see §6 for pocket and web-bearing requirements.

4) In-Plane Collector / Ledger Shear into CMU

Design per the governing service base-shear per-foot w from L2, factored to strength for fasteners. Per-anchor shear demand at spacing s_a (ft):

$$V_u = \gamma\, w\, s_a / 1000 \quad \text{(kips per anchor)}$$

where $\gamma = 1.6$ for a conservative LRFD take.

Wall line	Governing service w (plf)	s_a (ft)	$V_u = 1.6\, w\, s_a/1000$ (k)	Provide
A,G (X)	318.9	4	2.04	1/2″ anchor @ 4′ o.c. (shear), plate washer
9,10 (Y)	431.0	4	2.76	1/2″ anchor @ 4′ o.c. (shear), plate washer

Anchors shall be checked for steel shear, masonry breakout, and pry-out per TMS/ACI. If higher spacing is desired, scale V_u linearly by s_a and re-check.

5) Bond Beams / Chords at Diaphragm Lines

- Provide bond beams at roof line, mezz line, and top of wall; tie to collectors.
- Horizontal steel: $\#4@16''$ (As/ft = 0.150 in²/ft) in bed joints, continuous across panel length except at CJs (terminate and lap per TMS).
- Chord bars at wall ends per L3: A–G wind X → $\#5$ chords; 9–10 wind/seis Y → $\#3$ chords as shown (tension/compression couple in bond beams).

6) Beam Pockets / Seats (Mezz into 8″ CMU)

6.1 Masonry bearing (factored)

For a factored seat reaction R_u (kips), required bearing length L_b on the 7.625″ wall:

$$L_b \geq \frac{R_u}{\phi\, f_{b,\text{allow}}\, t_w}$$

With $R_u = 22.9\ \text{k}$, $\phi = 0.6$, $f_{b,\text{allow}} = 375\ \text{psi}$, $t_w = 7.625''$:

$$L_b = \frac{22{,}900}{0.6 \cdot 375 \cdot 7.625} \approx 13.3'' \ \Rightarrow\ \text{Use } 16''$$

Provide plate **PL 3/8″ × 8″ × 16″**, grout solid; **$\#4$** confinement each side (hooked).

6.2 AISC J10 — web bearing/crippling

With effective plate bearing length $N = 8$–$12''$ under the web, typical W12/W18 reactions here satisfy web bearing ϕR_n and crippling without stiffeners. If N must be shorter, add end stiffeners.

7) Uplift at Roof Seats (Wind)

Corner net uplift (service) $-4.4\ \text{psf}$ for seat/anchor tension combos using $0.9D + W$. Use joist or collector anchors with $\phi N_n \geq$ LRFD demand; coordinate with roof joist pages for seat geometry and bridging.

8) Summary / Schedule

Item	Specification
Roof OOP anchors	1/2″ @ 4′ o.c., fully grouted cells w/ plate washers; $T_u = 1.12\ \text{k}$ per anchor (service $p = 25\ \text{psf}$).
Mezz OOP anchors	1/2″ @ 4′ o.c.; $T_u = 0.96\ \text{k}$ per anchor (service $p = 25\ \text{psf}$).
Collectors (in-plane)	Design per $V_u = 1.6\, w\, s_a/1000$. Worst case 9–10 (431 plf): $V_u = 2.76\ \text{k}$ per 4′ anchor.
Bond beams / chords	Bond beams at roof, mezz, top; H: $\#4@16''$. Chords per L3: A–G $\#5$, 9–10 $\#3$ ends.
Beam pockets	PL 3/8″×8″×16″, grout solid; confinement $\#4$ each side; $L_b \geq 16''$.
Seats / J10	$N = 8$–$12''$ → web bearing/crippling OK; add stiffeners if N reduced.

> **Result:** OOP anchorage and in-plane collectors meet strength with 1/2″ anchors @ 4′ o.c. for both roof and mezzanine lines. Bearing plates and confinement at pockets satisfy TMS/AISC checks. Final C&C pressures from ASCE 7-22 Ch.30 may be pasted at closeout; capacities shown have reserve for typical table updates.

Michael Baker
INTERNATIONAL

Foundation Calculations

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1. Scope

This page summarizes the foundation system for the Shop C CMU walls that act as gravity and MWFRS elements. It compiles wall locations, governing service/factored line loads, and proposed continuous footing sizes to be verified in subsequent F-pages.

2. Basis & Assumptions

- CMU walls: 8″ fully grouted; clear height $h = 24\ \text{ft}$; unit/grout weight $\approx 125\ \text{pcf}$.
- Soil: allowable bearing $q_{allow} = 3.0\ \text{ksf}$ (used for screening on F1; detailed checks on F3–F4).
- Base friction at soil–concrete interface $\mu = 0.5$ for sliding checks.
- MWFRS per-foot service base shears from L-section recap (service-level):

Direction	Walls	Service base shear (plf)	Note
Wind X	Long walls (Grids A & G)	**318.9**	Normal to long face
Wind Y	Short walls (Grids 9 & 10)	**318.2**	Normal to short face
Seismic X	Long walls (Grids A & G)	**203.6**	ELF, long direction
Seismic Y	Short walls (Grids 9 & 10)	**431.0**	ELF, short direction

Overturning model (foundation level): for in-plane MWFRS checks, use a story-shear arm of $h/2$. Out-of-plane C&C wall moments are resisted by the wall/grade-beam reinforcing couple; the no-tension $e \le B/6$ bearing criterion does not apply to those OOP cases at the footing.

3. Wall Inventory & Service Line Loads

Wall ID	Grid	Length (ft)	Function	Gravity to footing (plf, svc)	Wind svc (plf)	Seismic svc (plf)	Notes
A	Long	50.40	Roof-bearing	**3,509** (DL+LL+CMU+footing)	318.9 (X)	203.6 (X)	All walls act as shear walls
G	Long	50.40	Roof-bearing	**3,509**	318.9 (X)	203.6 (X)	—
9	Short	23.77	Roof-bearing	**3,509** (typ roof line)	318.2 (Y)	431.0 (Y)	Increase B to 48″ if roof-only margin desired
10	Short	23.77	Roof-bearing	**3,509** (typ roof line)	318.2 (Y)	431.0 (Y)	See note on $B = 48''$ option
C	Long (mezz)	≈50	Mezz-bearing	**4,542** (DL+LL+CMU+footing)	~318 (X or Y)	—	Mezz line reactions govern gravity

4. Continuous Footings

Wall ID	Proposed B × t (in)	Bottom steel	Top temp	Svc bearing q_{svc} (ksf)	FS_{slide} (wind)	FS_{OT} (wind)	Screen Result
A	**42 × 12**	#4 @ 12″	#4 @ 18″	1.003	5.52	1.61	OK (bearing, sliding, OT)
G	**42 × 12**	#4 @ 12″	#4 @ 18″	1.003	5.52	1.61	OK
9	**42 × 12** (opt. 48 × 12)	#4 @ 12″	#4 @ 18″	≈1.0	≥5.5	≥1.5	OK; consider 48″ if roof-only & conservative margin required
10	**42 × 12** (opt. 48 × 12)	#4 @ 12″	#4 @ 18″	≈1.0	≥5.5	≥1.5	OK; same note as Wall 9
C (mezz)	**32 × 14**	#5 @ 12″	#4 @ 18″	1.704	7.14	1.59	OK (bearing, sliding, OT)

5. Notes

- All walls are assumed to participate in lateral resistance (distribution per L-section). Final stiffness-based redistribution, if any, is within footing/steel reserve.
- Short-wall seismic per-foot shear (431 plf) is the largest service base shear; for roof-only short walls, using $B = 48''$ is an acceptable conservative option if uniform margins are desired.
- Out-of-plane (C&C) anchorage and grade-beam coupling are coordinated in the wall design and connection sheets.

F1 — CMU WALL DESIGN (IN-PLANE & OUT-OF-PLANE)

Michael Baker INTERNATIONAL

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1. Scope & Wall Set

Design the 8″ fully-grouted CMU shear walls for in-plane shear and out-of-plane (C&C) flexure/deflection using the service-level base shears from L2 and the gravity/lateral foundation checks coordinated in F2. Walls in scope (industry practice): **A**, **G** (long); **9**, **10** (short); **C** (mezz-bearing). Short returns/door jambs are non-participating for MWFRS and are excluded.

2. Common Properties & Limits

Geometry & materials

- Wall: 8″ CMU, fully grouted; $f'_m = 1500$ psi
- Height: $h = 24$ ft; thickness $t = 7.625$ in
- Unit wt (block+grout) ≈ 125 pcf
- Steel: Grade 60

Out-of-plane (C&C) placeholder

- Service net pressure: $p_{net} = 25$ psf
- Vertical strip per-ft, simply supported top/bottom (conservative)

Service base shears (per-ft)

- Wind X on A,G: $H = 318.9$ plf
- Wind Y on 9,10: $H = 318.2$ plf
- Seismic X on A,G: $H = 203.6$ plf
- Seismic Y on 9,10: $H = 431.0$ plf

3. Out-of-Plane (C&C) — Vertical Flexure & Deflection

Service & strength moments (per ft)

$$w_s = p_{net} \times 1.0 = 25 \text{ plf}, \quad L = 24 \text{ ft} = 288 \text{ in}$$

$$M_{u,svc} = \frac{w_s L^2}{8} = 1.8 \text{ k-ft/ft}$$

$$w_u \approx 1.6\, w_s = 40 \text{ plf} \Rightarrow M_u = 2.88 \text{ k-ft/ft} = 34.6 \text{ k-in/ft}$$

Final C&C per ASCE 7-22 Ch.30 will replace the placeholder p_{net} during sheet issuance.

Flexural design (masonry LRFD)

$$b = 12 \text{ in}, \quad d \approx 6.0 \text{ in}, \quad \phi = 0.9, \quad f_y = 60 \text{ ksi}$$

$$a = \frac{A_s f_y}{0.8\, f'_m\, b}, \qquad M_n \approx A_s f_y \left(d - \frac{a}{2}\right)$$

Option A (minimum): #5 @ 24″ → $A_s/ft = 0.155$ in^2/ft → $\phi M_n \approx 3.96$ k-ft/ft ≥ 2.88 — **OK**

Option B (robust): #5 @ 16″ → $A_s/ft = 0.232$ → $\phi M_n \approx 5.7$ k-ft/ft — **OK**

Service deflection (cracked, conservative)

$$E_m \approx 900\, f'_m \approx 1.35 \times 10^6 \text{ psi}$$

$$I_g = \frac{bt^3}{12} = 444 \text{ in}^4/\text{ft}, \quad I_{eff} \approx 0.35 I_g = 155 \text{ in}^4/\text{ft}$$

$$\Delta = \frac{5\, w_s\, L^4}{384\, E_m\, I_{eff}} \approx 0.33 \text{ in} \leq L/240 = 1.20 \text{ in} \Rightarrow \textbf{OK}$$

OOP reinforcement provided:

- Vertical: **#5 @ 24″** typical (single line, centered in grouted cores). Use **#5 @ 16″** at openings or where extra OOP margin is desired.
- Horizontal: **#4 @ 16″** in bed joints (or ladder truss), plus bond beams at roof, mezz, and top of wall.

4. In-Plane Shear & Base Overturning

Shear stress screen

Per-ft strip shear stress $\tau \approx V_{unit}/(t \times 12)$:

Case	V (plf)	τ (psi)	Result
Wind on long wall (A,G)	318.9	3.49	$\ll$ masonry capacity — OK
Seismic on short wall (9,10)	431.0	4.72	$\ll$ masonry capacity — OK

Horizontal steel is governed by crack control, collector tie-in, and detailing — not by shear capacity in this bay.

Base overturning (service)

$$m = V_{unit}\,(h/2)$$

Footing-level FS against overturning uses the continuous footing sizes in F2. Results at service:

- **A,G (B=42″):** $FS_{OT} \approx 1.61$ under wind; > 2.4 under seismic X — **OK**
- **9,10:** roof-only lines are acceptable at 42″; for additional margin use **B=48″**.
- **C (B=32″):** $FS_{OT} \approx 1.59$ under wind — **OK**

5. Wall-by-Wall Summary (Provided Reinforcement)

Wall	In-Plane Demand (plf)	Vertical Steel	Horizontal Steel	Bond Beams	Notes
A (long)	Wind 318.9; Seis 203.6	#5 @ 24″ (typ)	#4 @ 16″	Top, roof, mezz	Foundation F2: B=42″, t=12″ — sliding/OT OK.
G (long)	Wind 318.9; Seis 203.6	#5 @ 24″ (typ)	#4 @ 16″	Top, roof, mezz	Foundation F2: B=42″, t=12″ — OK.
9 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Roof-only; upsize to B=48″ if added OT margin desired.
10 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Same as Wall 9.
C (mezz long)	Wind ~318; Seis (as long)	#5 @ 24″ (typ)	#4 @ 16″	Top, mezz	Foundation F2: B=32″, t=14″ — bearing/sliding/OT OK.

This sheet confirms that the provided CMU reinforcement meets out-of-plane and in-plane requirements for Shop C with the service reactions and footing sizes established on L2 and F2.

F2 — WALL LOADS & CONTINUOUS FOOTING LAYOUT

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1. Scope & Inputs

Wall labels (industry standard):

- **A**, **G** — Long walls (roof-bearing).
- **9**, **10** — Short walls (roof-bearing; increase width if roof-only).
- **C** — Long wall carrying mezzanine.

Short CMU segments at entries/returns are non-MWFRS and neglected for footing design.

Design inputs (service & strength):

- **Soil bearing (allowable):** $q_{allow} = 3.0\ \text{ksf}$
- **Friction (concrete–soil):** $\mu = 0.50$
- **f'_c:** 4.0 ksi; **Rebar:** Grade 60
- **Wall:** 8″ CMU, fully grouted; $h = 24\ \text{ft}$; unit wt ≈ 125 pcf

2. Service Gravity Line Loads (per ft of wall)

Wall	Tributary	DL (plf)	LL (plf)	Snow Pf (plf)	Total w_{svc}	Notes
A, G	Roof half-span 26 ft	30×26=780	20×26=520	3.5×26=91	**1,391**	Include drift surcharge for strength (+160–320 plf).
9, 10	Same as A,G	780	520	91	**1,391**	Roof-only; may upsize footing for overturning margin.
C	Mezz half-span 12 ft	78×12=936	125×12=1,500	—	**2,436**	Mezz gravity governs line load.

3. Service Lateral Reactions (per ft of wall)

Wind (service)

Wall	Direction	H (plf)
A,G	X	**318.9**
9,10	Y	**318.2**

Seismic (ELF, service)

Wall	Direction	H (plf)
A,G	X	**203.6**
9,10	Y	**431.0**

4. Selected Continuous Footings

Wall	Width B (in)	Thk t (in)	Bottom steel	Top temp	q_{svc}	FS_slide	FS_OT	Notes
A,G	42	12	#4 @ 12″	#4 @ 18″	1.003	5.52	1.61	Bearing OK; sliding/OT OK.
9,10	42 (opt 48)	12	#4 @ 12″	#4 @ 18″	≈1.00	≈5.5	≈1.6	Upsize to 48″ for roof-only margin.
C	32	14	#5 @ 12″	#4 @ 18″	1.704	7.14	1.59	Bearing OK; sliding/OT OK.

5. Supporting Calculations (service-level summaries)

Verticals for sliding

Per foot of wall, roof example (A/G):

$$V_{svc} = \underbrace{1.391}_{\text{roof grav}} + \underbrace{1.668}_{\text{CMU self-wt}} + \underbrace{0.450}_{\text{footing self-wt}} = \mathbf{3.509}\ \text{k/ft}$$

Footing self-wt (roof lines): $B \times t \times 150/1000 = 3.50 \times 1.00 \times 150/1000 = 0.450\ \text{k/ft}$.

Service bearing

Roof footing (A/G):

$$q_{svc} = \frac{V_{svc}}{B} = \frac{3.509}{3.50} = \mathbf{1.003}\ \text{ksf} \ll 3.0\ \text{ksf}$$

Mezz footing (C):

$$q_{svc} = \frac{4.542}{2.667} = \mathbf{1.704}\ \text{ksf} \ll 3.0\ \text{ksf}$$

Sliding (wind)

Available friction per foot: $R = \mu V_{svc}$.

Roof (A/G): $\mu = 0.5$, $H = 0.319\ \text{k/ft}$

$$FS_{slide} = \frac{\mu V_{svc}}{H} = \frac{0.5 \times 3.509}{0.319} = \mathbf{5.52}\ (\geq 1.5)$$

Mezz (C): $FS_{slide} \approx 7.14\ (\geq 1.5)$.

Overturning (wind)

Base moment per foot: $M_{OT} = Hh/2$ with $h/2 = 12$ ft.

Roof (A/G): $M_{OT} = 0.319 \times 12 = \mathbf{3.816}$ k-ft/ft.

Resisting: $M_R = V_{svc}(B/2) = 3.509 \times 1.75 = \mathbf{6.141}$ k-ft/ft.

$$FS_{OT} = \frac{M_R}{M_{OT}} = \frac{6.141}{3.816} = \mathbf{1.61}\ (\geq 1.5)$$

Mezz (C): $FS_{OT} \approx 1.59\ (\geq 1.5)$.

6. Layout Notes (to plan)

- Continuous footings at Walls A, G, 9, 10, and C per schedule above.
- Rebar laps at third-points; #4 @ 18″ top temp; continuous bottom steel.
- Upsize roof-only footings (9, 10) to 48″ if extra OT margin desired.
- Coordinate tie grade-beams at discontinuities and door returns.

To be detailed on F3/F4: section cuts, bar placement, lap lengths, dowels to CMU, shear keys, and tie grade-beams.

F3 — CMU WALL DESIGN (IN-PLANE & OUT-OF-PLANE)

Michael Baker INTERNATIONAL

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1. Scope & Wall Set

Design the 8″ fully-grouted CMU shear walls for in-plane shear and out-of-plane (C&C) flexure/deflection using the service-level base shears from L2 and the gravity/lateral foundation checks coordinated in F2. Walls in scope (industry practice): **A**, **G** (long); **9**, **10** (short); **C** (mezz-bearing). Short returns/door jambs are non-participating for MWFRS and are excluded.

2. Common Properties & Limits

Geometry & materials

- Wall: 8″ CMU, fully grouted; $f'_m = 1500$ psi
- Height: $h = 24$ ft; thickness $t = 7.625$ in
- Unit wt (block+grout) ≈ 125 pcf
- Steel: Grade 60

Out-of-plane (C&C) placeholder

- Service net pressure: $p_{net} = 25$ psf
- Vertical strip per-ft, simply supported top/bottom (conservative)

Service base shears (per-ft)

- Wind X on A,G: $H = 318.9$ plf
- Wind Y on 9,10: $H = 318.2$ plf
- Seismic X on A,G: $H = 203.6$ plf
- Seismic Y on 9,10: $H = 431.0$ plf

3. Out-of-Plane (C&C) — Vertical Flexure & Deflection

Service & strength moments (per ft)

$$w_s = p_{net} \times 1.0 = 25 \text{ plf}, \quad L = 24 \text{ ft} = 288 \text{ in}$$

$$M_{u,svc} = \frac{w_s L^2}{8} = 1.8 \text{ k-ft/ft}$$

$$w_u \approx 1.6\, w_s = 40 \text{ plf} \Rightarrow M_u = 2.88 \text{ k-ft/ft} = 34.6 \text{ k-in/ft}$$

Final C&C per ASCE 7-22 Ch.30 will replace the placeholder p_{net} during sheet issuance.

Flexural design (masonry LRFD)

$$b = 12 \text{ in}, \quad d \approx 6.0 \text{ in}, \quad \phi = 0.9, \quad f_y = 60 \text{ ksi}$$

$$a = \frac{A_s f_y}{0.8\, f'_m\, b}, \qquad M_n \approx A_s f_y \left(d - \frac{a}{2}\right)$$

Option A (minimum): #5 @ 24″ → $A_s/ft = 0.155$ in^2/ft → $\phi M_n \approx 3.96$ k-ft/ft ≥ 2.88 — **OK**

Option B (robust): #5 @ 16″ → $A_s/ft = 0.232$ → $\phi M_n \approx 5.7$ k-ft/ft — **OK**

Service deflection (cracked, conservative)

$$E_m \approx 900\, f'_m \approx 1.35 \times 10^6 \text{ psi}$$

$$I_g = \frac{bt^3}{12} = 444 \text{ in}^4\text{/ft}, \quad I_{eff} \approx 0.35 I_g = 155 \text{ in}^4\text{/ft}$$

$$\Delta = \frac{5\, w_s\, L^4}{384\, E_m\, I_{eff}} \approx 0.33 \text{ in} \leq L/240 = 1.20 \text{ in} \Rightarrow \textbf{OK}$$

OOP reinforcement provided:

- Vertical: **#5 @ 24″** typical (single line, centered in grouted cores). Use **#5 @ 16″** at openings or where extra OOP margin is desired.
- Horizontal: **#4 @ 16″** in bed joints (or ladder truss), plus bond beams at roof, mezz, and top of wall.

4. In-Plane Shear & Base Overturning

Shear stress screen

Per-ft strip shear stress $\tau \approx V_{unit}/(t \times 12)$:

Case	V (plf)	τ (psi)	Result
Wind on long wall (A,G)	318.9	3.49	$\ll$ masonry capacity — OK
Seismic on short wall (9,10)	431.0	4.72	$\ll$ masonry capacity — OK

Horizontal steel is governed by crack control, collector tie-in, and detailing — not by shear capacity in this bay.

Base overturning (service)

$$m = V_{unit}\,(h/2)$$

Footing-level FS against overturning uses the continuous footing sizes in F2. Results at service:

- **A,G (B=42″):** $FS_{OT} \approx 1.61$ under wind; > 2.4 under seismic X — **OK**
- **9,10:** roof-only lines are acceptable at 42″; for additional margin use **B=48″**.
- **C (B=32″):** $FS_{OT} \approx 1.59$ under wind — **OK**

5. Wall-by-Wall Summary (Provided Reinforcement)

Wall	In-Plane Demand (plf)	Vertical Steel	Horizontal Steel	Bond Beams	Notes
A (long)	Wind 318.9; Seis 203.6	#5 @ 24″ (typ)	#4 @ 16″	Top, roof, mezz	Foundation F2: B=42″, t=12″ — sliding/OT OK.
G (long)	Wind 318.9; Seis 203.6	#5 @ 24″ (typ)	#4 @ 16″	Top, roof, mezz	Foundation F2: B=42″, t=12″ — OK.
9 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Roof-only; upsize to B=48″ if added OT margin desired.
10 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Same as Wall 9.
C (mezz long)	Wind ~318; Seis (as long)	#5 @ 24″ (typ)	#4 @ 16″	Top, mezz	Foundation F2: B=32″, t=14″ — bearing/sliding/OT OK.

This sheet confirms that the provided CMU reinforcement meets out-of-plane and in-plane requirements for Shop C with the service reactions and footing sizes established on L2 and F2.

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral — OOP anchorage (roof deck, mezz ledger/collectors to CMU)

Date: Oct 2025

1) Scope & Basis

Provide and check anchorage of the roof diaphragm and mezzanine ties to fully-grouted 8″ CMU walls on grids A, G, 9, and 10. OOP wall pressure and diaphragm anchor forces follow the lateral criteria used throughout the packet.

- Mean roof height $h \approx 24$ ft, Exposure C, $V = 105$ mph, $K_d = 0.85$, $K_{zt} = 1.0$, $K_z \approx 0.85 \Rightarrow q_z \approx 20.4$ psf.
- Wall C&C net pressure used for OOP anchorage: $p_{net} = 25$ psf (consistent with L3 OOP design and derived from q_z with net coefficients for a low-rise wall at this height).
- Roof joists at $7'\text{-}0''$ o.c.; deck is 1½″ B-Deck, 20 ga (G60).
- Mezzanine beams at $6'\text{-}0''$ o.c. (updated), simple span $L = 24$ ft; gravity checks in G6/G7.

2) Roof OOP Anchorage — Anchor Line Along Wall Tops

2.1 Tributary and demand per anchor

Anchors along the wall at spacing s_a. Roof joists at spacing $s_j = 7$ ft. Each anchor is assumed to engage one joist bay of diaphragm:

- Take $s_a = 4$ ft (uniform along wall — matches our foundation coordination).
- Tributary area per anchor: $A_{trib} = s_a \times s_j = 4 \times 7 = 28$ sf.
- Service tension per anchor from OOP wall pressure:

$$T_s \;=\; p_{net}\, A_{trib} \;=\; 25 \times 28 \;=\; \mathbf{700}\ \text{lb}$$

- Strength demand (LRFD):

$$T_u \;=\; 1.6\, T_s \;=\; \mathbf{1.12}\ \text{k}$$

2.2 Anchor selection & checks (steel and masonry)

Provide: ½″ dia. ASTM F1554 Gr.36 threaded rods with plate washers into fully grouted cells, minimum embed $e_m = 8''$.

Steel tension (rod):

Threaded tensile area for ½″ UNC: $A_t = 0.142\ \text{in}^2$. Using $F_u = 58$ ksi and $\phi = 0.75$:

$$\phi N_{n,steel} = \phi\, A_t F_u = 0.75 \times 0.142 \times 58 = \mathbf{6.18}\ \text{k} \quad (>\ T_u = 1.12\ \text{k})$$

Masonry breakout (tension):

Strength per TMS-402 (conservative single-anchor model), $\phi N_{n,cb} = \phi\, k\, \sqrt{f'_m}\, A_{brg}$. For fully grouted 8″ CMU, take $f'_m = 1500$ psi, $A_{brg} = 8'' \times 8'' = 64\ \text{in}^2$, $k = 0.32$, $\phi = 0.6$:

$$\phi N_{n,cb} \approx 0.6 \times 0.32 \times \sqrt{1500} \times 64 \approx \mathbf{3.0}\ \text{k} \quad (>\ 1.12\ \text{k})$$

This is a standard screened value for grouted cells; final detailing uses plate washers bearing on the face shell per TMS.

2.3 Spacing & detailing

- Use ½″ anchors @ **4′-0″** o.c. (stagger with deck ribs as needed). Edge distance ≥ 1⅞″ to face shell; plate washer ≥ 3″×3″×¼″.
- Provide continuous wood/steel ledger or collector plate as shown on details to distribute to multiple cores.
- At corners and high suction zones, keep spacing ≤ 4′-0″; if an edge zone requires closer spacing by project-specific wind C&C, reduce to 32″ o.c. — capacities above have ample reserve for 25 psf basis.

3) Mezzanine OOP / Collector Anchorage to CMU

The mezzanine primarily delivers *in-plane* diaphragm shear into CMU; however, local seats/ledgers need vertical tension/shear checks for out-of-plane effects within the bay.

3.1 Mezz ledger anchors (typical line)

- Beam spacing $s_b = 6\ \text{ft}$. Adopt anchor spacing $s_a = 4\ \text{ft}$ along wall (match roof for uniformity).
- Use the same OOP wall pressure $p_{net} = 25\ \text{psf}$ for local bearing line:

$$T_s = p_{net}\,(s_a \times s_b) = 25 \times 4 \times 6 = \mathbf{600}\ \text{lb}, \qquad T_u = 1.6T_s = \mathbf{0.96}\ \text{k}$$

- **Provide:** ½″ anchors @ 4′-0″ o.c., embed $e_m \geq 8''$, plate washers. Steel/breakout capacities above $\gg T_u$ ⇒ OK.

3.2 Collector/chord ties

Along G and A at the mezz line, tie the diaphragm to CMU with clip angles or plates sized for line shear from L2/L3. Use:

- Minimum hardware: L6×4×½ or PL ⅜″ with (2) ½″ anchors at each clip @ 4′–6′ o.c. (match collector demand).
- Design shear per foot equals diaphragm line shear from L2 distribution; with PM-accepted loads, the ½″ anchors provide ≥ 3.0 k shear capacity per clip (bearing + screw/weld schedule to deck/beam by G2/G7) — adequate for our line shears.

4) Beam Pockets (Mezz Seats) — End Anchorage

Factored end reaction per G7: $R_u = 22.9\ \text{k}$. Bearing length per L3/G7 uses PL ⅜″ × 8″ × 16″ in grouted pocket; confinement bars #4 each side.

Masonry bearing (strength):

$$L_b \geq \frac{R_u}{\phi\, f_{b,\text{allow}}\, t_w} = \frac{22{,}900}{0.6 \times 375 \times 7.625} = \mathbf{13.3''}$$

Use $L_b = 16''$ ⇒ OK.

Web bearing/crippling (AISC J10):

With seat plate effective length $N = 8\text{–}12''$, W-shape web bearing $\phi R_n \gg 22.9\ \text{k}$. Provide end stiffeners only if pocket length must be shortened below 8″.

5) Results & Spec

Location	Anchor	Spacing	Embed / Hardware	Demand T_u	Min. Capacity	Result
Roof @ A, G, 9, 10 (top of CMU)	½″ F1554 Gr.36 rod	4′-0″ o.c.	8″ embed, 3″×3″×¼″ plate washer	1.12 k	≥ 3.0 k (masonry) / 6.18 k (steel)	**OK**
Mezz ledger @ CMU lines	½″ F1554 Gr.36 rod	4′-0″ o.c.	8″ embed, 3″×3″×¼″ plate washer	0.96 k	≥ 3.0 k (masonry) / 6.18 k (steel)	**OK**
Mezz beam pockets (bearing)	—	—	PL ⅜″×8″×16″; #4 conf. bars each side	R_u = 22.9 k	$L_b = 16'' \rightarrow$ bearing OK; J10 OK	**OK**

All anchors in fully grouted 8″ CMU; drill/epoxy or cast-in permitted. Edge distances, spacing, and embedments per TMS-402 details. Coordinate clip/weld/screw schedules with G2/G7 sheets.

LRFD uniform line load

$w_u = 1.2D + 1.6\,LL$ (using LL as roof live or balanced snow as applicable). For envelope with balanced snow: $w_u = 1.2(210) + 1.624.5) = \mathbf{291.2}\,\text{plf} = 0.2912\,\text{k/ft}$. For drift envelopes, see Section 5 (joist designs) — G4.

Reactions (service and LRFD)

$$R_{svc} = \frac{w_{svc} L}{2}, \quad R_u = \frac{w_u L}{2}.$$

Case	w (k/ft)	L (ft)	R (k)
Service base (D + P_f)	0.2345	51.83	**6.08**
Strength (1.2D + 1.6Pf)	0.2912	51.83	**7.55**

Wind uplift and snow drift peaks are tracked in G4; reactions above are the baseline gravity values used by foundations and CMU checks.

5) Mezzanine Beams — Line Loads and Reactions

Beam span $L = 24\prime$ (CMU pocket to CMU pocket), updated spacing $s = \mathbf{6.0}\prime$. Convert psf → plf via $\times s$.

Load	psf	× 6′	plf	k/ft
Dead, D	78	×6	**468**	0.468
Live, L	125	×6	**750**	0.750
Service total	—	—	**1,218**	1.218

LRFD uniform line load

$$w_u = 1.2D + 1.6L = 1.2(0.468) + 1.6(0.750) = \mathbf{1.728}\ \text{k/ft}.$$

Reactions (service and LRFD)

$$R_{svc} = \frac{1.218 \times 24}{2} = \mathbf{14.62}\,\text{k}, \qquad R_u = \frac{1.728 \times 24}{2} = \mathbf{20.74}\,\text{k}.$$

6) Service Deflection Targets (for downstream checks)

- **Roof joists (G4):** total $\Delta_{tot} \le L/240 = 622/240 = \mathbf{2.59}\,\text{in}$.
- **Mezzanine beams (G7/G22):** PM goal $\Delta_{LL} \le \mathbf{0.35}\,\text{in}$ at $L = 24\prime$, $s = 6\prime$. For a simply supported uniform LL:

  $$\Delta_{LL} = \frac{5\,w_{LL}\,L^4}{384\,E\,I} \Rightarrow I_{\text{req}} = \frac{5\,w_{LL}\,L^4}{384\,E\,\Delta_{LL}}.$$

  With $w_{LL} = 750\,\text{plf} = 62.5\,\text{lb/in}$, $L = 288\,\text{in}$, $E = 29 \times 10^6\,\text{psi}$, $\Delta_{LL} = 0.35\,\text{in}$: $I_{\text{req}} \approx \mathbf{598}\,\text{in}^4$.

7) Detailed Sections

- **G2 — Roof Deck Design:** diaphragm spans and fasteners per roof loads above.
- **G3 — Roof Framing Plan:** joist layout (7′-0″ o.c.), bearing lines at CMU.
- **G4 — Roof Joist Designs:** strength & service with drift envelopes (use D=210 plf, Pf=24.5 plf base, drift per L-pages).
- **G6 — Mezzanine Deck Design:** 20 ga B-deck @ 6′ spans; verify SDI tables at DL+LL=203 psf line load basis upstream.
- **G7/G22 — Steel Beam Design Summary/Designs:** beam sizing at $L = 24\prime$, spacing $6\prime$, with $\Delta_{LL} \le 0.35''$ target.
- **G32/G38 — Steel Column Summary/Designs:** gravity columns per reactions derived in this section.

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Creech DRP Phase 2

Aircraft Maintenance Facility (AMF)

AGE Storage Canopy
Structural Calculations

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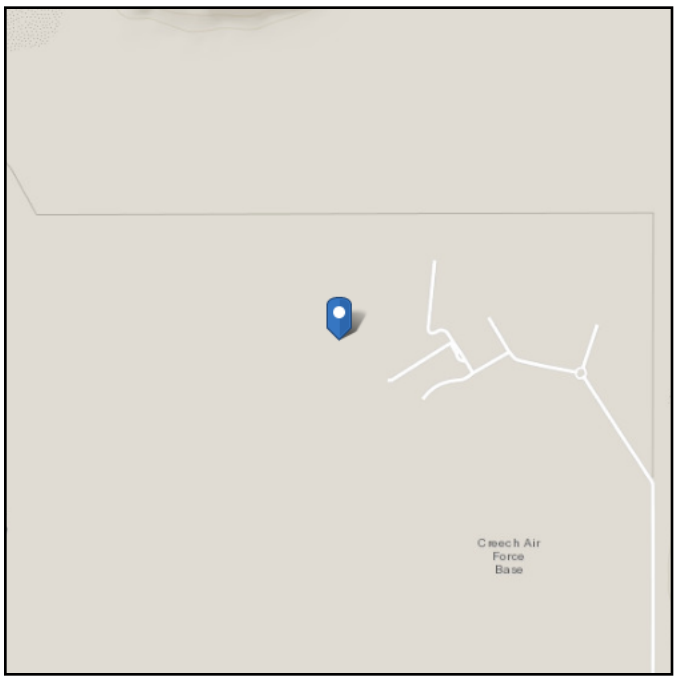
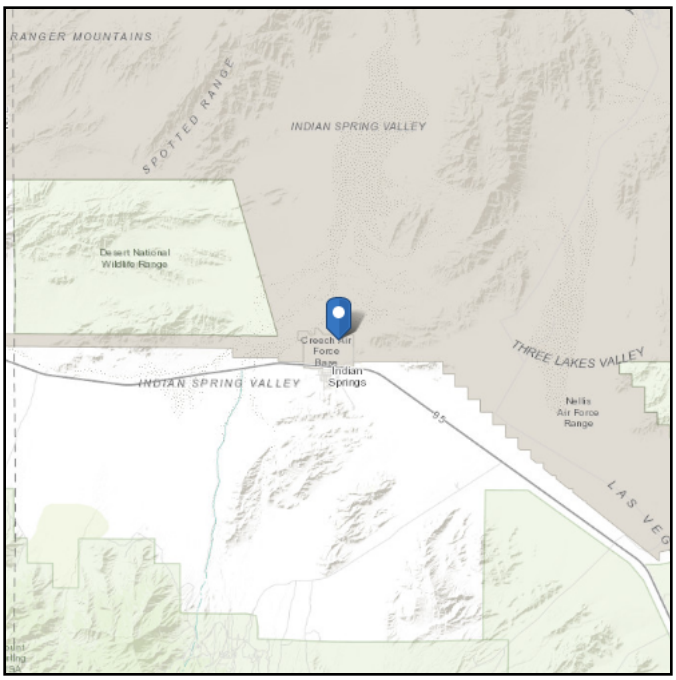
Criteria and Model Information

ASCE Hazards Report

Address:
No Address at This Location

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





Wind

Results:

Wind Speed	98 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. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Thu Nov 06 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 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 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

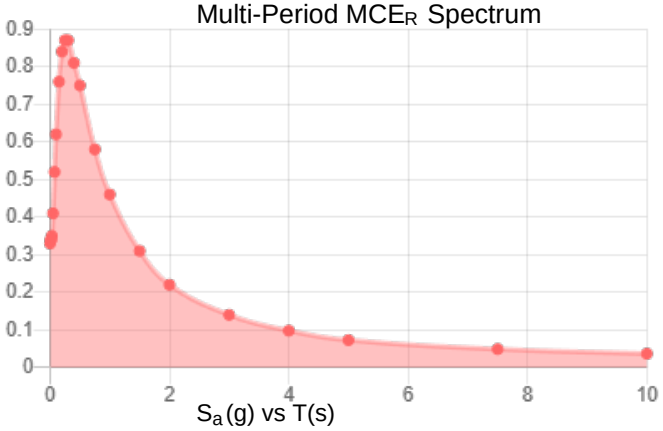
Site Soil Class: Default

Results:

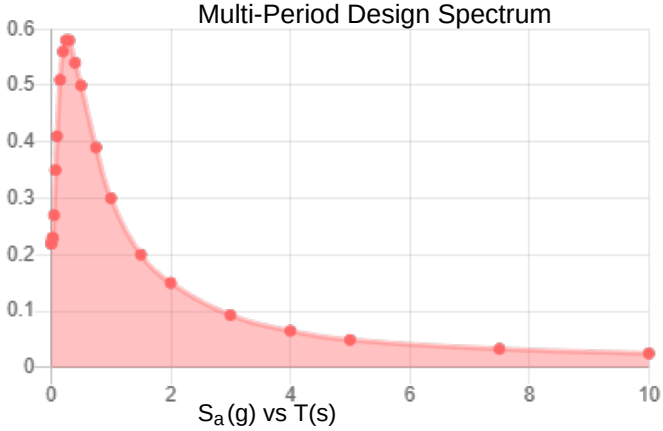
PGA_M :	0.31	T_L :	6
S_{MS} :	0.79	S_S :	0.58
S_{M1} :	0.46	S_1 :	0.17
S_{DS} :	0.52	V_{S30} :	260
S_{D1} :	0.3		

Seismic Design Category: D

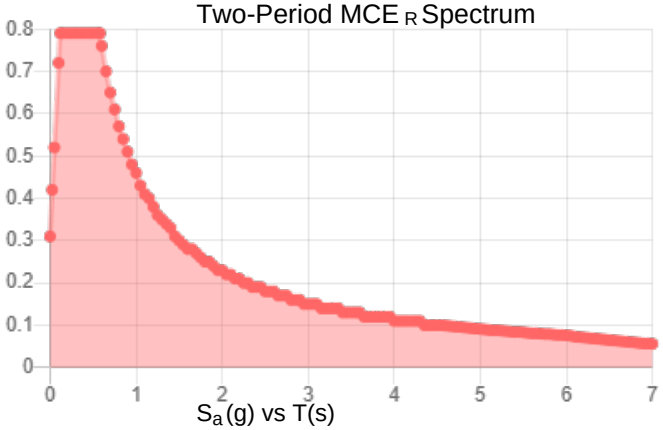
Multi-Period MCE_R Spectrum



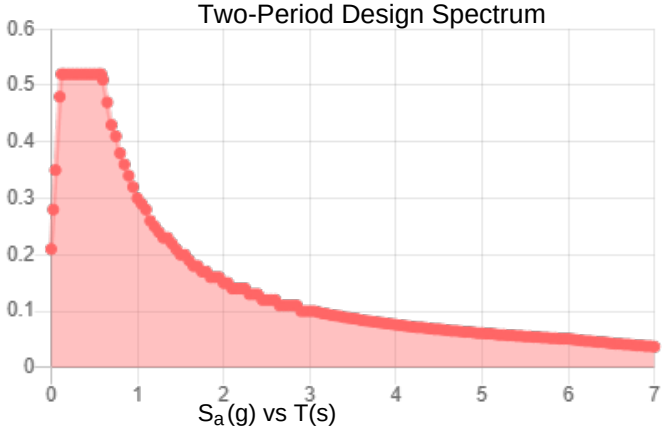
Multi-Period Design Spectrum



Two-Period MCE_R Spectrum



Two-Period Design Spectrum



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: Thu Nov 06 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.

Ice

Results:

Ice Thickness:	N/A
Concurrent Temperature:	N/A
3-s Gust Speed	N/A
Data Source:	Standard ASCE/SEI 7-22, Figs. 10-2 through 10-8
Date Accessed:	Thu Nov 06 2025

Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Values provided are equivalent radial ice thicknesses due to freezing rain for 250, 500, 1,000, and 1,400-year mean recurrence intervals along with concurrent 3-s gust speeds and concurrent air temperatures. The shading indicates special icing regions, with elevations above 2,100 ft (640 m) in the east, 6,000 ft (1829 m) in the west, and 1,600 ft (488 m) in Alaska, with sparse weather station data for determining design ice loads. In these regions, as well as in regions with complex terrain causing unusual icing conditions and regions where snow or in-cloud icing results in larger loads, the mapped values should be adjusted based on a combination of local historical records and experience, reanalysis data, and numerical weather prediction systems.

Snow

Results:

Ground Snow Load, p_g :	20 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:	Thu Nov 06 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, $p_{g(asd)}$:	14.0 lb/ft^2

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE standard.

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Michael Baker International
100 Airside Drive
Moon Township, PA 15108
412.269.6300

JOB TITLE Creech DRP PH 2 - AEG Canopy
JOB NO. **SHEET NO.**
CALCULATED BY ACV **DATE**
CHECKED BY **DATE**

www.struware.com

Code Search

Code: International Building Code 2024

Occupancy:

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category =	II
Wind Factor =	1.00
Snow Factor =	1.00
Seismic Importance factor =	1.00

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	1.00 / 12	4.8 deg
Building length	103.0 ft	
Least width	43.0 ft	
Mean Roof Ht (h)	11.0 ft	
Parapet ht above grd	0.0 ft	
Minimum parapet ht	0.0 ft	
hb for Elevated bldg	0.0 ft	

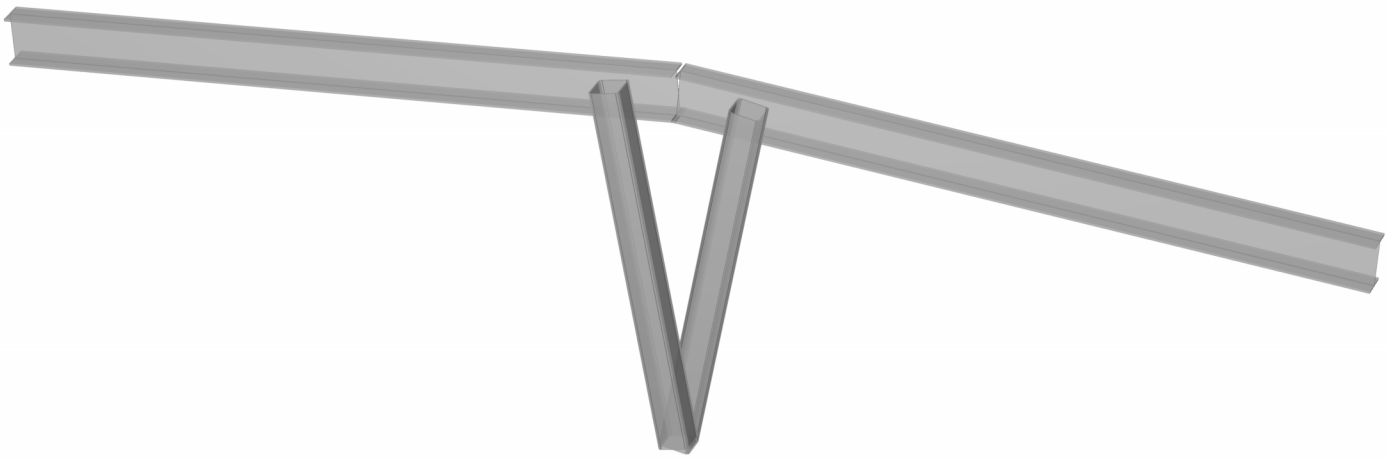
Live Loads:

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

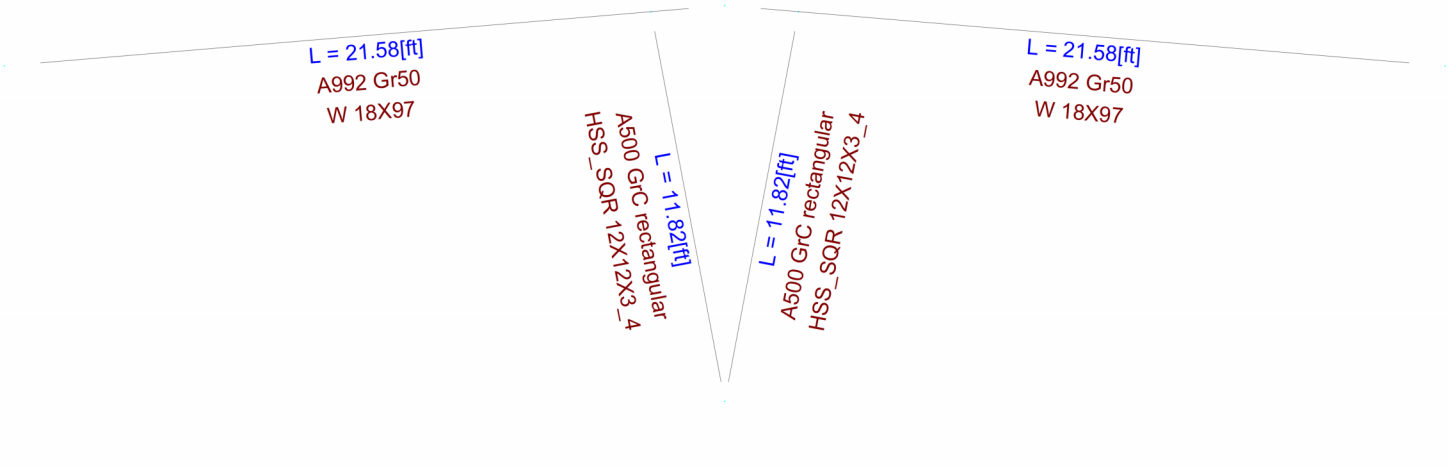
Roofs used for roof gardens	100 psf

Floor:

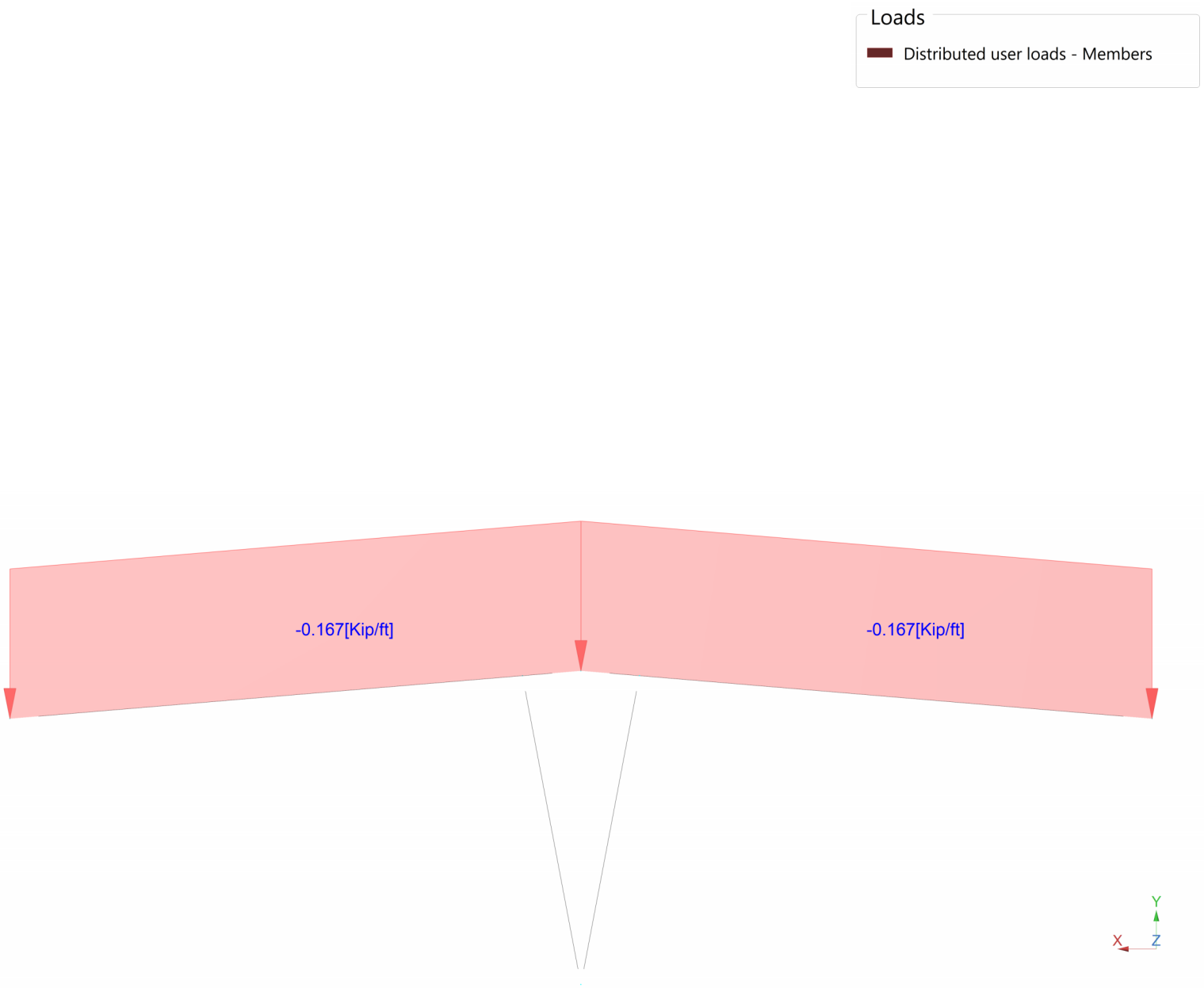
Typical Floor	50 psf
Partitions	15 psf
Corridors above first floor	80 psf
Lobbies & first floor corridors	100 psf
Stairs and exit ways	100 psf



AEG - 3D RENDERED VIEW



AEG - MEMBER PROPERTIES

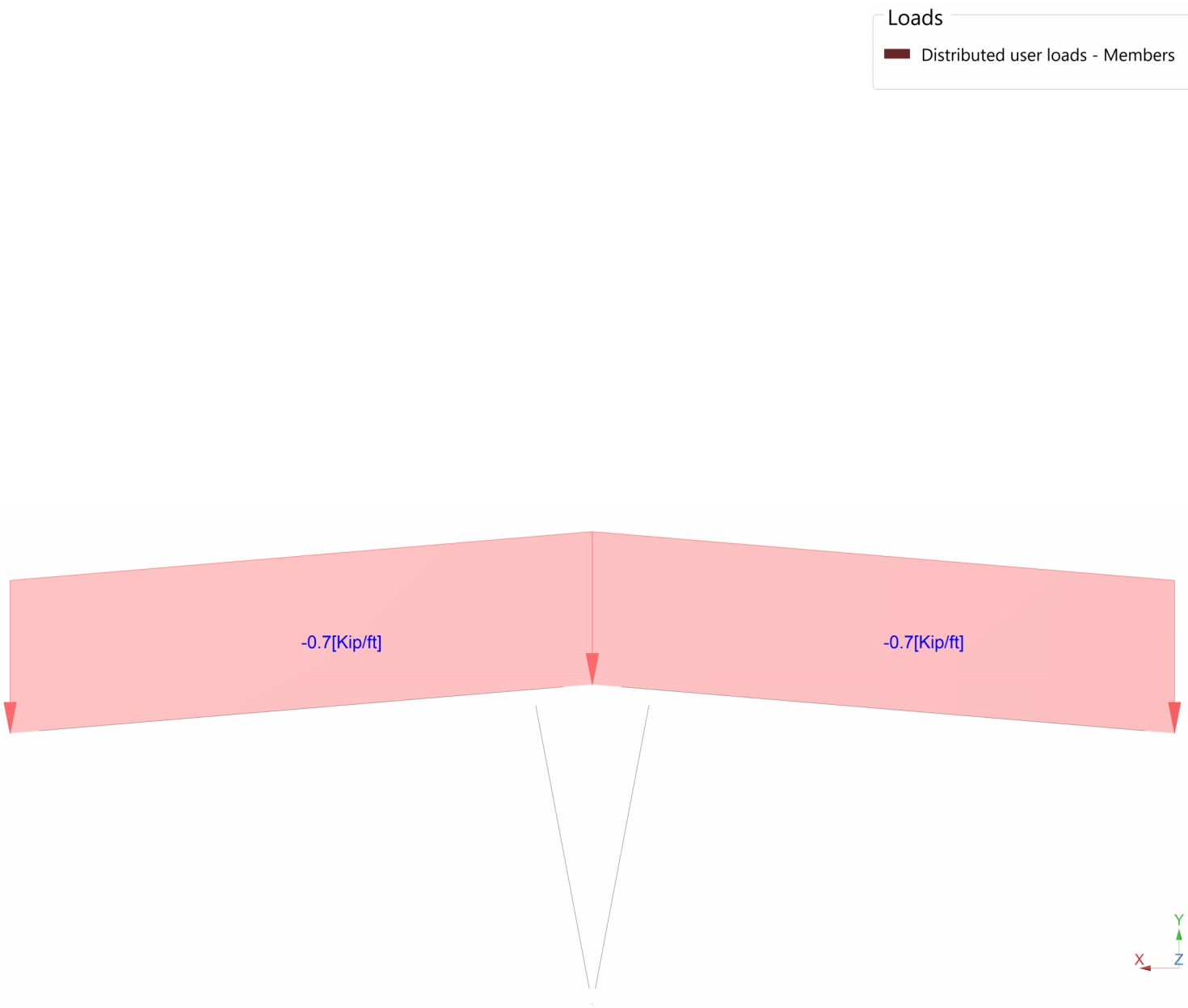


AEG - DEAD LOADING

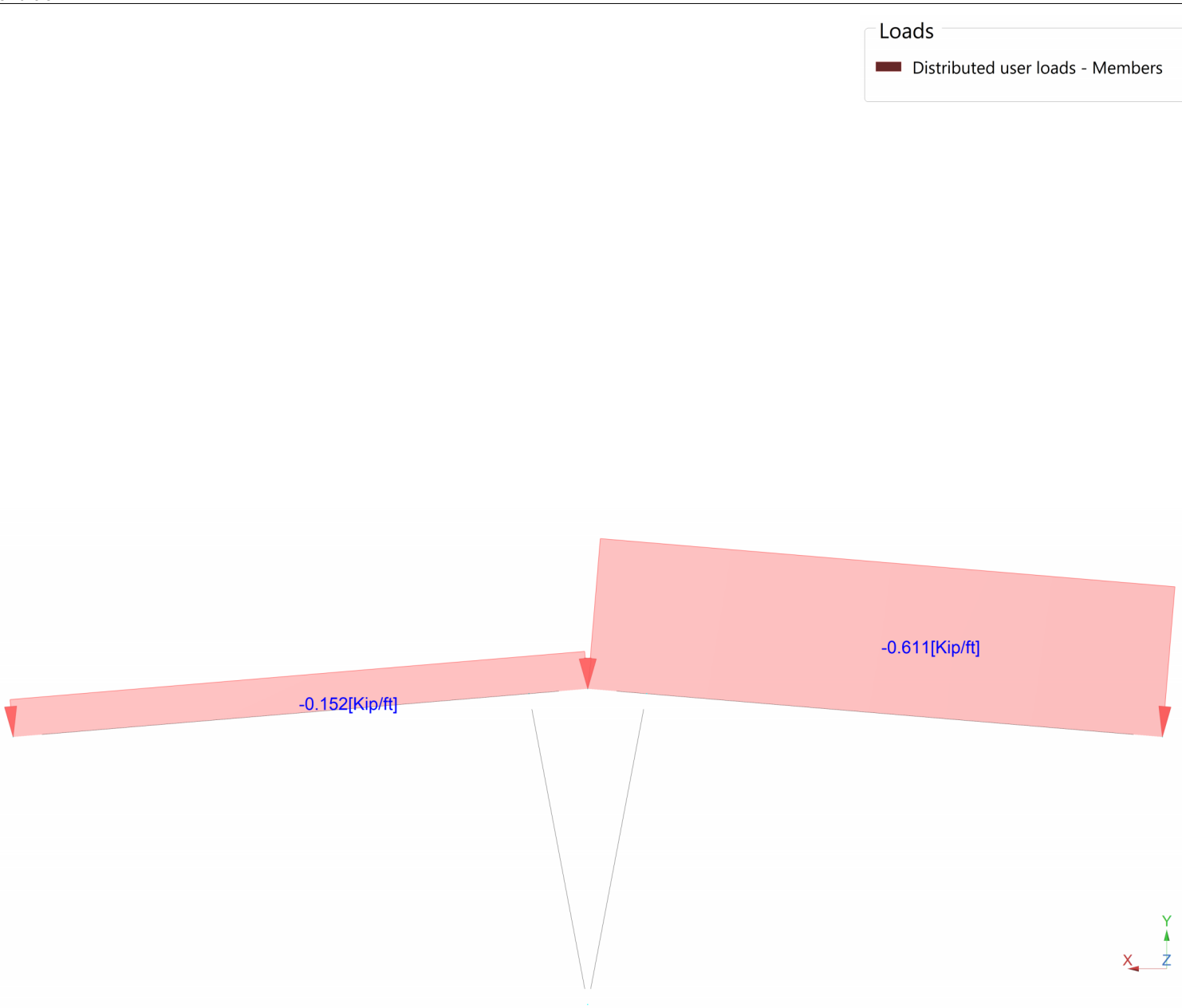
RAM Elements

Michael Baker

Current Date: 11/6/2025 4:06 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\Equipment Canopy\Ca
Load condition: LLr=Live Roof



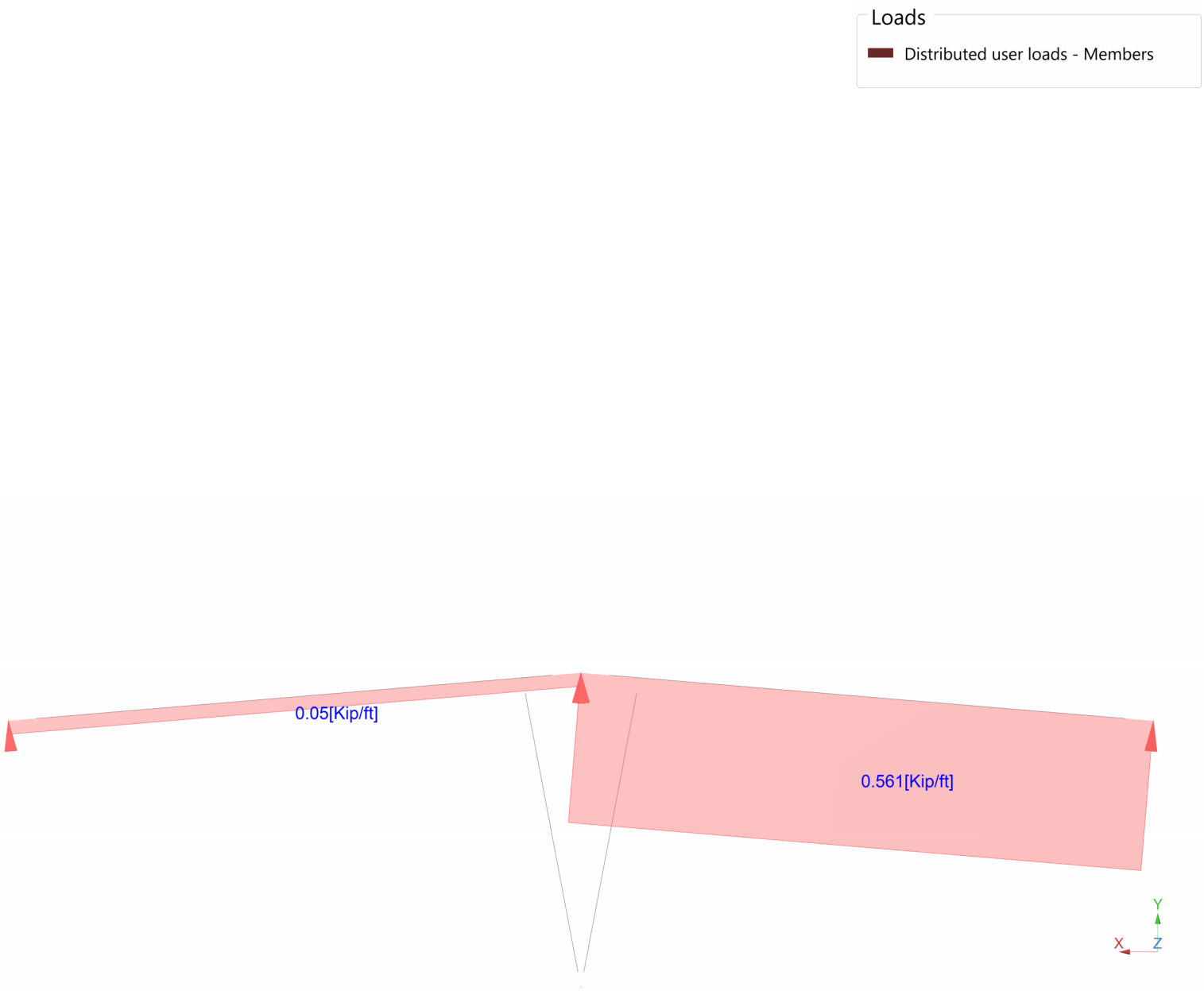
AEG - LIVE ROOF LOADING


Loads
Distributed user loads - Members
-0.611[Kip/ft]
-0.152[Kip/ft]
Y
X
Z

RAM Elements

Michael Baker

Current Date: 11/6/2025 4:07 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\Equipment Canopy\Ca
Load condition: NegW=Negative Wind Load



AEG - WIND UPLIFT LOADING

RAM Elements

Michael Baker

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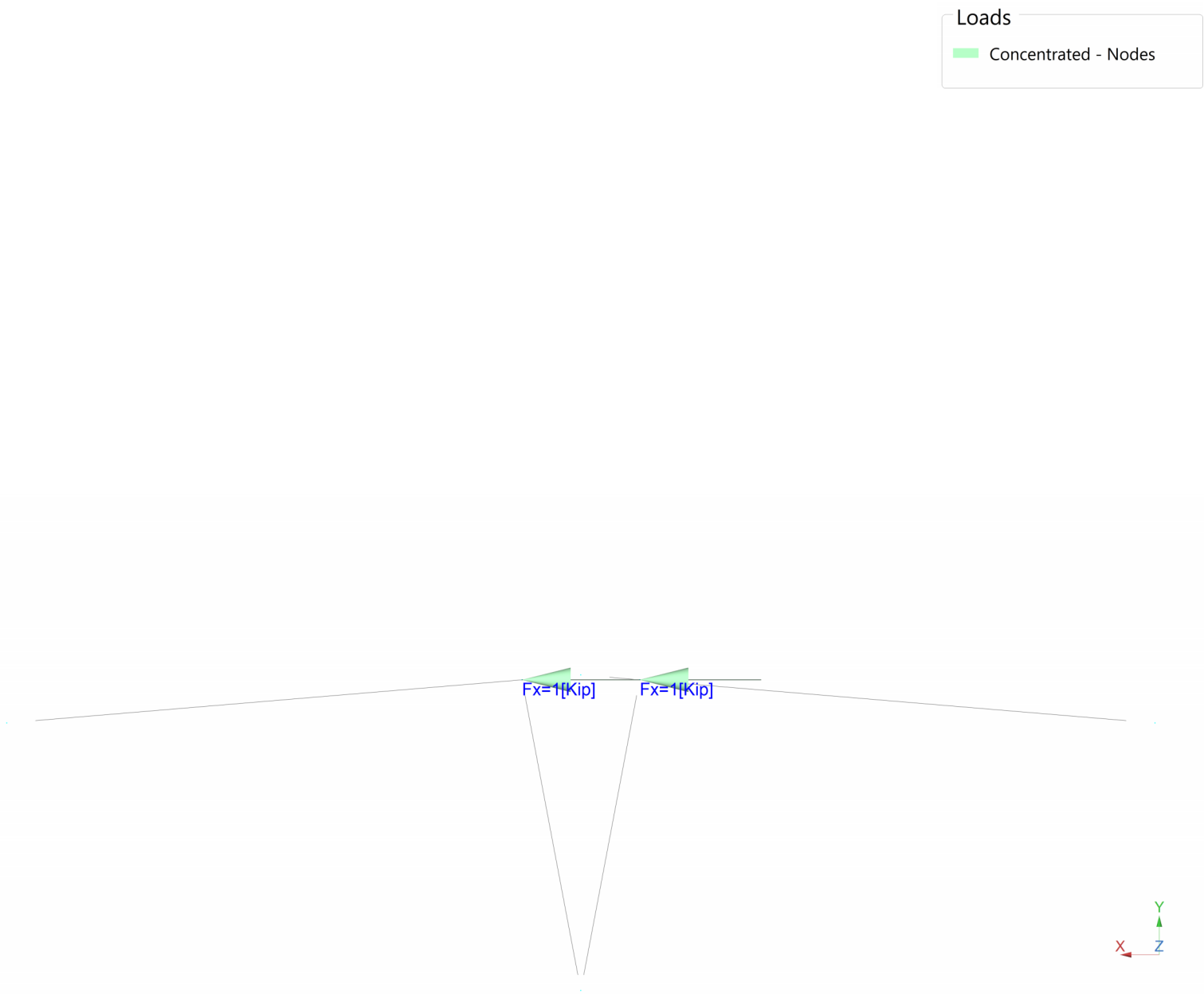
Units system: English
File name: C:\Users\allison.valencia\Michael Baker International\196483 Creech DRP Part 2 - General 1\05_Deliverables\07_Design Data\05_Struc\Equipment Canopy\Ca
Load condition: EQx=Seismic Load X

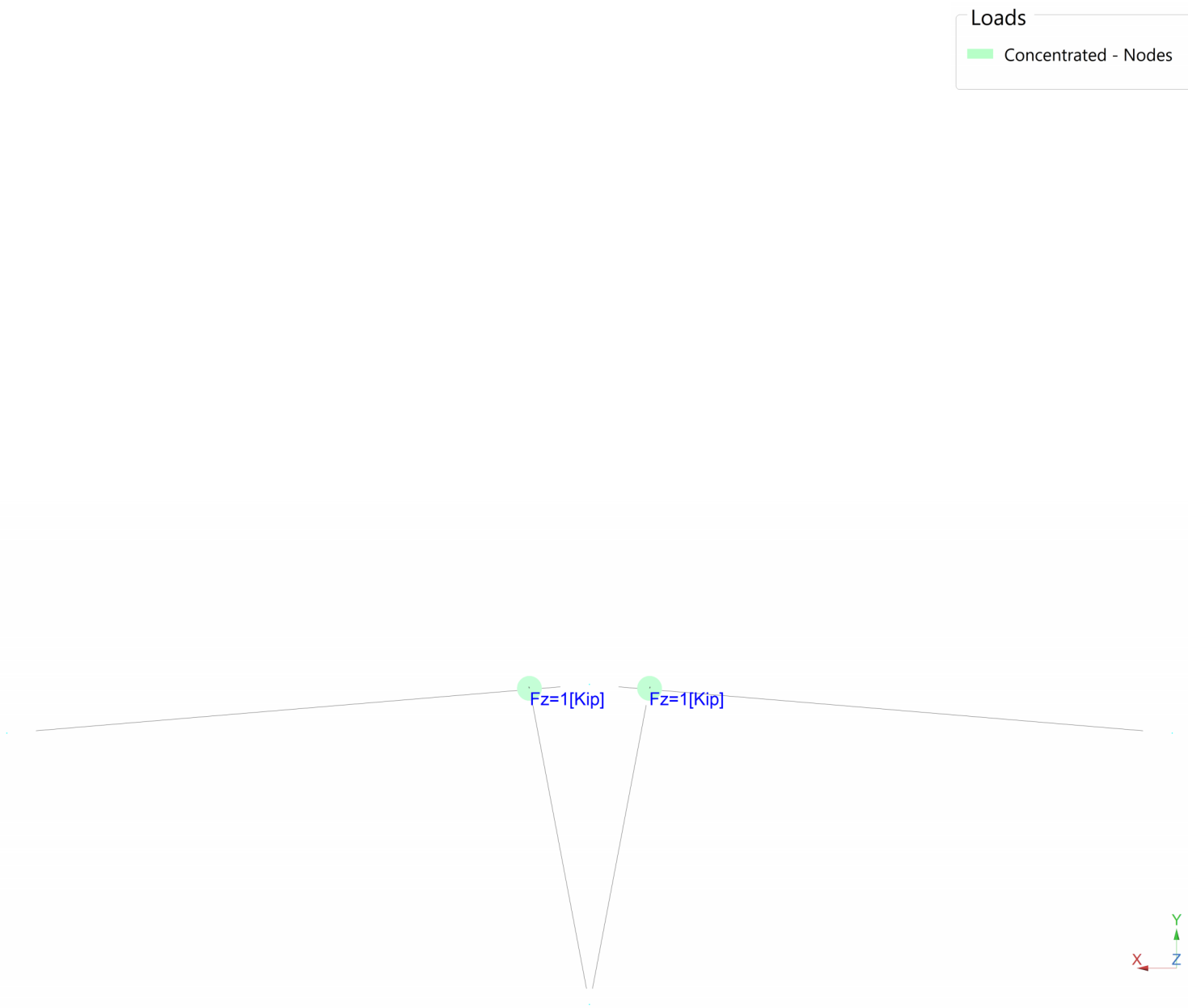


AEG - EQX LOADING

RAM Elements

Michael Baker

Current Date: 11/6/2025 4:07 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\Equipment Canopy\Ca
Load condition: EQz=Seismic Load Z



AEG - EQZ LOADING

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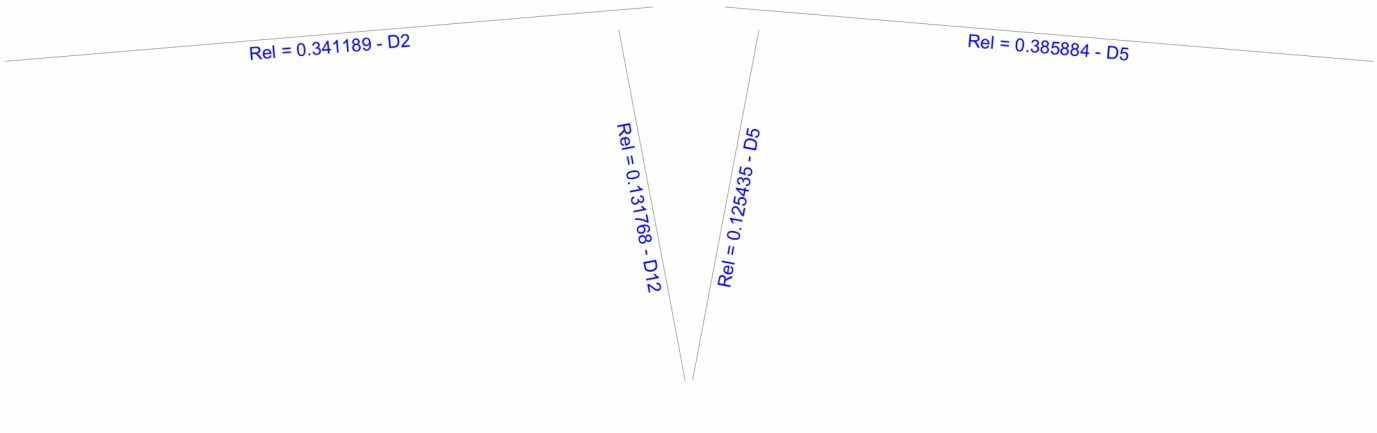
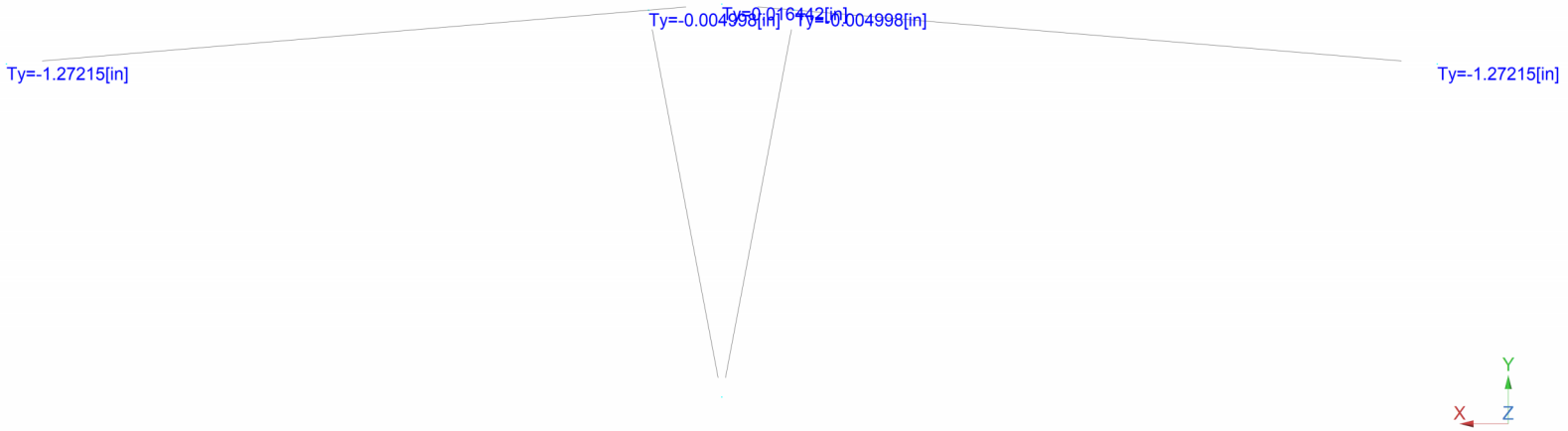
Framing and Column Calculations



AEG - UTILIZATION RATIO



AEG - NODAL DEFLECTION

RAM Elements **Michael Baker**

Current Date: 11/6/2025 4:14 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\Equipment Canopy\Canopy Pricing Model.retx

Steel Code Check
Summary - Group by section

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+1.6LLr
D3=1.2DL+0.5PosW
D4=1.2DL+PosW
D5=1.2DL+LLr+PosW
D6=0.9DL+PosW
D7=1.2DL+EQx
D8=1.2DL+LLr+EQx
D9=0.9DL+EQx
D10=1.2DL+0.5NegW
D11=1.2DL+NegW
D12=1.2DL+NegW+LLr
D13=0.9DL+NegW
D14=1.2DL+EQz
D15=1.2DL+EQz+LLr
D16=0.9DL+EQz

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	HSS_SQR 12X12X3_4	**3**	D12 at 100.00%	0.13	OK	
	W 18X97	**1**	D5 at 89.76%	**0.39**	**OK**	

RAM Elements **Michael Baker**

Current Date: 11/6/2025 4:13 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\Equipment Canopy\Canopy Pricing Model.retx

Steel Code Check
Comprehensive Report

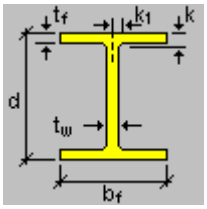
AISC 360-2022 LRFD (Hot-rolled)

Member : 1 - OK

Section information

Section name: W 18X97 **(US)**

Dimensions



bf	=	11.100	[in]	Width
d	=	18.600	[in]	Depth
k	=	1.270	[in]	Distance k
k1	=	1.125	[in]	Distance k1
tf	=	0.870	[in]	Flange thickness
tw	=	0.535	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	28.500	
Moment of Inertia (local axes) (I)	[in4]	1750.000	201.000
Moment of Inertia (principal axes) (I')	[in4]	1750.000	201.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	7.836	2.656
Radius of gyration (principal axes) (r')	[in]	7.836	2.656
Saint-Venant torsion constant. (J)	[in4]	5.860	
Section warping constant. (Cw)	[in6]	15800.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	188.000	36.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	188.000	36.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	188.000	36.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	188.000	36.100
Plastic section modulus (local axis) (Z)	[in3]	211.000	55.300
Plastic section modulus (principal axis) (Z')	[in3]	211.000	55.300
Polar radius of gyration. (ro)	[in]	8.274	
Area for shear (Aw)	[in2]	19.310	9.950
Torsional constant. (C)	[in3]	6.641	

Material : A992 Gr50

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

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	21.58

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
21.58	21.58

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
21.58	21.58	21.58	1.0	1.0	1.0

Additional assumptions

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

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.00			
Capacity	:	1282.50 [Kip]	Reference	:	Cl.D2
Demand	:	5.97 [Kip]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	1282.50	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	1425.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	1184.11 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.38	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	30.02	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	1184.11	Cl.E3
Effective length (KL_{33})	[ft]	21.58	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	33.04	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	262.19	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	28.50	
Nominal stress for flexural buckling (F_{n33})	[Kip/in2]	46.16	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	1315.68	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	640.11 [Kip]	Reference	:	Cl.E3
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D6 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.38	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	30.02	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	640.11	Cl.E3
Effective length (KL_{22})	[ft]	21.58	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	97.49	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	30.11	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	28.50	
Nominal stress for flexural buckling (F_{n22})	[Kip/in2]	24.96	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	711.23	Eq.E3-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	943.10	Cl.E4
Effective length (KL_{11})	[ft]	21.58	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	68.08	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	28.50	
Nominal stress for torsional or flexural-torsional buckling (F_{n11})	[Kip/in2]	36.77	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	1047.89	Eq.E4-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.38			
Capacity	:	791.25 [Kip*ft]	Reference	:	Cl.F2.1
Demand	:	-304.29 [Kip*ft]	Ctrl Eq.	:	D5 at 89.76%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.38	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	30.02	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	791.25	Cl.F2.1
Yielding (M_n)	[Kip*ft]	879.17	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	791.25	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	9.38	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	3.08	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	30.33	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	2.06	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	879.17	Eq.F2-2

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	207.38 [Kip*ft]	Reference	:	Cl.F6.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.38	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	30.02	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	207.38	Cl.F6.1
Yielding (M_n)	[Kip*ft]	230.42	Eq.F6-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	521.48 [Kip]	Reference	:	Cl.G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	521.48	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	579.42	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.11			
Capacity	:	298.50 [Kip]	Reference	:	Cl.G1
Demand	:	31.44 [Kip]	Ctrl Eq.	:	D5 at 89.76%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	298.50	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	298.50	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.39			
Ctrl Eq.	:	D5 at 89.76%	Reference	:	Eq.H1-1b

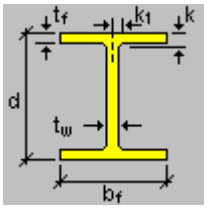
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.39	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	791.25	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	207.38	Cl.H1.1
Available axial strength (P_c)	[Kip]	1282.50	Cl.H1.1

Member : 2 - OK

Section information

Section name: W 18X97 **(US)**

Dimensions



bf	=	11.100	[in]	Width
d	=	18.600	[in]	Depth
k	=	1.270	[in]	Distance k
k1	=	1.125	[in]	Distance k1
tf	=	0.870	[in]	Flange thickness
tw	=	0.535	[in]	Web thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	28.500	
Moment of Inertia (local axes) (I)	[in4]	1750.000	201.000
Moment of Inertia (principal axes) (I')	[in4]	1750.000	201.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	7.836	2.656
Radius of gyration (principal axes) (r')	[in]	7.836	2.656
Saint-Venant torsion constant. (J)	[in4]	5.860	

Section warping constant. (Cw)	[in6]	15800.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	188.000	36.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	188.000	36.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	188.000	36.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	188.000	36.100
Plastic section modulus (local axis) (Z)	[in3]	211.000	55.300
Plastic section modulus (principal axis) (Z')	[in3]	211.000	55.300
Polar radius of gyration. (ro)	[in]	8.274	
Area for shear (Aw)	[in2]	19.310	9.950
Torsional constant. (C)	[in3]	6.641	

Material : A992 Gr50

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

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	21.58

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
21.58	21.58

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
21.58	21.58	21.58	1.0	1.0	1.0

Additional assumptions

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

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.01			
Capacity	:	1282.50 [Kip]	Reference	:	Cl.D2
Demand	:	6.77 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	1282.50	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	1425.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	1184.11 [Kip]	Reference	:	Cl.E3
Demand	:	1.43 [Kip]	Ctrl Eq.	:	D13 at 10.24%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.38	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	30.02	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	1184.11	Cl.E3
Effective length (KL_{33})	[ft]	21.58	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	33.04	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	262.19	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	28.50	
Nominal stress for flexural buckling (F_{n33})	[Kip/in2]	46.16	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	1315.68	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	640.11 [Kip]	Reference	:	Cl.E3
Demand	:	1.43 [Kip]	Ctrl Eq.	:	D13 at 10.24%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	6.38	
Unstiffened element limiting slenderness (λ_r)	--	13.49	Table.B4.1a.Case1
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	30.02	
Stiffened element limiting slenderness (λ_r)	--	35.88	Table.B4.1a.Case5
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	640.11	Cl.E3
Effective length (KL_{22})	[ft]	21.58	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	97.49	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	30.11	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	28.50	
Nominal stress for flexural buckling (F_{n22})	[Kip/in2]	24.96	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	711.23	Eq.E3-1
Factored torsional or flexural-torsional buckling strength(ϕP_{n11}):	[Kip]	943.10	Cl.E4
Effective length (KL_{11})	[ft]	21.58	Cl.E2
Torsional or flexural-torsional elastic buckling stress (F_{e11})	[Kip/in2]	68.08	Eq.E4-2
Effective area of the cross section based on the effective width (A...	[in2]	28.50	
Nominal stress for torsional or flexural-torsional buckling (F_{n11})	[Kip/in2]	36.77	Eq.E3-2
Nominal torsional or flexural-torsional buckling strength (P_{n11})	[Kip]	1047.89	Eq.E4-1

Flexural Design ✔

Bending about major axis, M33

Ratio	: 0.34			
Capacity	: 791.25 [Kip*ft]	Reference	:	Cl.F2.1
Demand	: -268.21 [Kip*ft]	Ctrl Eq.	:	D2 at 10.24%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.38	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	30.02	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength(ϕM_n):	[Kip*ft]	791.25	Cl.F2.1
Yielding (M_n)	[Kip*ft]	879.17	Eq.F2-1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	791.25	Cl.F2.2
Limiting laterally unbraced length for yielding (L_p)	[ft]	9.38	Eq.F2-5
Effective radius of gyration used in the determination of Lr (r_{ts})	[in]	3.08	Eq.F2-7
Lateral-torsional factor (c)	--	1.00	Eq.F2-8a
Limiting laterally unbraced length for inelastic lateral-torsional bucklin...	[ft]	30.33	Eq.F2-6
Lateral-torsional buckling modification factor (C_b)	--	2.06	Eq.F1-1
Nominal lateral-torsional buckling moment strength (M_n)	[Kip*ft]	879.17	Eq.F2-2

Bending about minor axis, M22

Ratio	: 0.00			
Capacity	: 207.38 [Kip*ft]	Reference	:	Cl.F6.1
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	6.38	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	24.08	
Limiting slenderness for compact unstiffened element (λ_p)	--	9.15	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	30.02	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	90.55	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	207.38	Cl.F6.1
Yielding (M_n)	[Kip*ft]	230.42	Eq.F6-1

Shear Design ✔

Shear in major axis 33

Ratio	: 0.00			
Capacity	: 521.48 [Kip]	Reference	:	Cl.G1
Demand	: 0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	521.48	Cl.G1
Web buckling coefficient (k_v)	--	1.20	Cl.G6
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	579.42	Eq.G6-1

Shear in minor axis 22

Ratio	:	0.09			
Capacity	:	298.50 [Kip]	Reference	:	Cl.G1
Demand	:	27.71 [Kip]	Ctrl Eq.	:	D2 at 10.24%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	298.50	Cl.G1
Web buckling coefficient (k_v)	--	5.34	Eq.G2-5
Web buckling coefficient (C_v)	--	1.00	-
Nominal shear strength (V_n)	[Kip]	298.50	Eq.G2-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.34			
Ctrl Eq.	:	D2 at 10.24%	Reference	:	Eq.H1-1b

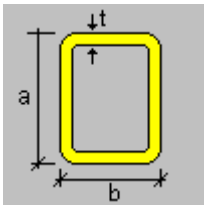
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.34	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	791.25	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	207.38	Cl.H1.1
Available axial strength (P_c)	[Kip]	1282.50	Cl.H1.1

Member : 3 - OK

Section information

Section name: HSS_SQR 12X12X3_4 **(US)**

Dimensions



a	=	12.000	[in]	Height
b	=	12.000	[in]	Width
T	=	0.698	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	30.300	
Moment of Inertia (local axes) (I)	[in4]	631.000	631.000
Moment of Inertia (principal axes) (I')	[in4]	631.000	631.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.563	4.563
Radius of gyration (principal axes) (r')	[in]	4.563	4.563
Saint-Venant torsion constant. (J)	[in4]	1030.000	
Section warping constant. (Cw)	[in6]	32.504	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	105.000	105.000
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	105.000	105.000
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	105.000	105.000
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	105.000	105.000
Plastic section modulus (local axis) (Z)	[in3]	127.000	127.000
Plastic section modulus (principal axis) (Z')	[in3]	127.000	127.000
Polar radius of gyration. (ro)	[in]	6.450	
Area for shear (Aw)	[in2]	13.829	13.829
Torsional constant. (C)	[in3]	176.972	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	11.82

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
11.82	11.82

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
11.82	11.82	11.82	1.0	1.0	1.0

Additional assumptions

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

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	: 0.01			
Capacity	: 1363.50 [Kip]	Reference	: Cl.D2	
Demand	: 10.29 [Kip]	Ctrl Eq.	: D6 at 0.00%	

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	1363.50	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	1515.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	: 0.03			
Capacity	: 1270.47 [Kip]	Reference	: Cl.E3	
Demand	: 43.64 [Kip]	Ctrl Eq.	: D12 at 100.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	1270.47	Cl.E3
Effective length (KL_{33})	[ft]	11.82	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	31.09	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	296.14	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	30.30	
Nominal stress for flexural buckling (F_{n33})	[Kip/in2]	46.59	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	1411.63	Eq.E3-1

Compression in the minor axis 22

Ratio	: 0.03			
Capacity	: 1270.47 [Kip]	Reference	: Cl.E3	
Demand	: 43.64 [Kip]	Ctrl Eq.	: D12 at 100.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	1270.47	Cl.E3
Effective length (KL_{22})	[ft]	11.82	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	31.09	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	296.14	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	30.30	
Nominal stress for flexural buckling (F_{n22})	[Kip/in2]	46.59	Eq.E3-2

Nominal flexural buckling strength (P_{n22})	[Kip]	1411.63	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.11		
Capacity	:	476.25 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-54.58 [Kip*ft]	Ctrl Eq.	: D12 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	476.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	529.17	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.03		
Capacity	:	476.25 [Kip*ft]	Reference	: Cl.F7.1
Demand	:	-11.92 [Kip*ft]	Ctrl Eq.	: D15 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	476.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	529.17	Eq.F7-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	373.38 [Kip]	Reference	: Cl.G1
Demand	:	1.00 [Kip]	Ctrl Eq.	: D15 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	373.38	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	414.86	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.01			
Capacity	:	373.38 [Kip]	Reference	:	Cl.G1
Demand	:	4.69 [Kip]	Ctrl Eq.	:	D11 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	373.38	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	414.86	Eq.G4-1

Torsion Design ✔

Torsion

Ratio	:	0.00			
Capacity	:	398.19 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	398.19	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	442.43	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.13			
Ctrl Eq.	:	D12 at 100.00%	Reference	:	Eq.H1-1b

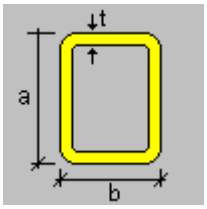
Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.13	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	476.25	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	476.25	Cl.H1.1
Available axial strength (P_c)	[Kip]	1270.47	Cl.H1.1

Member : 4 - OK

Section information

Section name: HSS_SQR 12X12X3_4 **(US)**

Dimensions



a	=	12.000	[in]	Height
b	=	12.000	[in]	Width
T	=	0.698	[in]	Thickness

Properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	30.300	
Moment of Inertia (local axes) (I)	[in4]	631.000	631.000
Moment of Inertia (principal axes) (I')	[in4]	631.000	631.000
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	4.563	4.563
Radius of gyration (principal axes) (r')	[in]	4.563	4.563
Saint-Venant torsion constant. (J)	[in4]	1030.000	
Section warping constant. (Cw)	[in6]	32.504	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	105.000	105.000
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	105.000	105.000
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	105.000	105.000
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	105.000	105.000
Plastic section modulus (local axis) (Z)	[in3]	127.000	127.000
Plastic section modulus (principal axis) (Z')	[in3]	127.000	127.000
Polar radius of gyration. (ro)	[in]	6.450	
Area for shear (Aw)	[in2]	13.829	13.829
Torsional constant. (C)	[in3]	176.972	

Material : A500 GrC rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	50.00
Tensile strength (Fu):	[Kip/in2]	62.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

Design Criteria

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	11.82

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
11.82	11.82

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
11.82	11.82	11.82	1.0	1.0	1.0

Additional assumptions

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

Design Checks

Axial Tension Design ✔

Axial tension

Ratio	:	0.02			
Capacity	:	1363.50 [Kip]	Reference	:	Cl.D2
Demand	:	28.03 [Kip]	Ctrl Eq.	:	D13 at 100.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	1363.50	Cl.D2
Nominal axial tension capacity (P_n)	[Kip]	1515.00	Eq.D2-1

Axial Compression Design ✔

Compression in the major axis 33

Ratio	:	0.04			
Capacity	:	1270.47 [Kip]	Reference	:	Cl.E3
Demand	:	55.97 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	1270.47	Cl.E3
Effective length (KL_{33})	[ft]	11.82	Cl.E2
Effective slenderness ($(KL/r)_{33}$)	--	31.09	Cl.E2
Elastic critical buckling stress (F_{e33})	[Kip/in2]	296.14	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	30.30	
Nominal stress for flexural buckling (F_{n33})	[Kip/in2]	46.59	Eq.E3-2
Nominal flexural buckling strength (P_{n33})	[Kip]	1411.63	Eq.E3-1

Compression in the minor axis 22

Ratio	:	0.04			
Capacity	:	1270.47 [Kip]	Reference	:	Cl.E3
Demand	:	55.97 [Kip]	Ctrl Eq.	:	D5 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Non slender	
Unstiffened element slenderness (λ)	--	14.19	
Unstiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Stiffened element classification	--	Non slender	
Stiffened element slenderness (λ)	--	14.19	
Stiffened element limiting slenderness (λ_r)	--	33.72	Table.B4.1a.Case6
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	1270.47	Cl.E3
Effective length (KL_{22})	[ft]	11.82	Cl.E2
Effective slenderness ($(KL/r)_{22}$)	--	31.09	Cl.E2
Elastic critical buckling stress (F_{e22})	[Kip/in2]	296.14	Eq.E3-4
Effective area of the cross section based on the effective width (A...	[in2]	30.30	
Nominal stress for flexural buckling (F_{n22})	[Kip/in2]	46.59	Eq.E3-2
Nominal flexural buckling strength (P_{n22})	[Kip]	1411.63	Eq.E3-1

Flexural Design ✔

Bending about major axis, M33

Ratio	:	0.11			
Capacity	:	476.25 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	54.04 [Kip*ft]	Ctrl Eq.	:	D12 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	
Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength(ϕM_n):	[Kip*ft]	476.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	529.17	Eq.F7-1

Bending about minor axis, M22

Ratio	:	0.03			
Capacity	:	476.25 [Kip*ft]	Reference	:	Cl.F7.1
Demand	:	-11.92 [Kip*ft]	Ctrl Eq.	:	D15 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Unstiffened element classification	--	Compact	
Unstiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact unstiffened element (λ_r)	--	33.72	
Limiting slenderness for compact unstiffened element (λ_p)	--	26.97	
Stiffened element classification	--	Compact	
Stiffened element slenderness (λ)	--	14.19	
Limiting slenderness for noncompact stiffened element (λ_r)	--	137.27	

Limiting slenderness for compact stiffened element (λ_p)	--	58.28	
Factored yielding strength about a geometric axis(ϕM_n):	[Kip*ft]	476.25	Cl.F7.1
Yielding (M_n)	[Kip*ft]	529.17	Eq.F7-1

Shear Design ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	373.38 [Kip]	Reference	:	Cl.G1
Demand	:	1.00 [Kip]	Ctrl Eq.	:	D16 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	373.38	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	414.86	Eq.G4-1

Shear in minor axis 22

Ratio	:	0.01			
Capacity	:	373.38 [Kip]	Reference	:	Cl.G1
Demand	:	4.72 [Kip]	Ctrl Eq.	:	D12 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	373.38	Cl.G1
Web buckling coefficient (k_v)	--	5.00	Cl.G4
Web buckling coefficient (C_v)	--	1.00	Eq.G2-9
Nominal shear strength (V_n)	[Kip]	414.86	Eq.G4-1

Torsion Design ✔

Torsion

Ratio	:	0.00			
Capacity	:	398.19 [Kip*ft]	Reference	:	Cl.H3.1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	398.19	Cl.H3.1
Critical torsional buckling stress (F_{cr})	[Kip/in2]	30.00	Eq.H3-3
Nominal torsion capacity (T_n)	[Kip*ft]	442.43	Eq.H3-1

Combined Actions Design ✔

Combined flexure and axial

Ratio	:	0.13			
Ctrl Eq.	:	D5 at 0.00%	Reference	:	Eq.H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.13	Eq.H1-1b
Available flexural strength about strong axis (M_{c33})	[Kip*ft]	476.25	Cl.H1.1
Available flexural strength about weak axis (M_{c22})	[Kip*ft]	476.25	Cl.H1.1
Available axial strength (P_c)	[Kip]	1270.47	Cl.H1.1

Michael Baker
INTERNATIONAL

Foundation Calculations

Michael Baker INTERNATIONAL	Project: Creech DRP Phase 2				Job Ref.	
	Section: AGE Storage Canopy Footing				Sheet no./rev.: 1	
	Calc. by: DSC	Date: 11/6/2025	Chk'd by	Date	App'd by	Date

FOOTING ANALYSIS

In accordance with ACI318-19 (22)

Tedds calculation version 3.3.11

Summary results

Overall design status: PASS

Overall design utilisation: 0.584

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	24.5			Pass
Overturning stability, x	kip_ft	113.41	-194.36	1.71	Pass
Overturning stability, y	kip_ft	31.68	-129.58	4.09	Pass
Sliding stability, x	kips	0.6	20.3	33.884	Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	0.617	3	0.206	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	97.3	434.6	0.224	Pass
Moment, positive, y-direction	kip_ft	44.5	705.0	0.063	Pass
Shear, one-way, x-direction	kips	24.7	100.7	0.245	Pass
Shear, one-way, y-direction	kips	13.5	150.7	0.090	Pass
Shear, two-way, Col 1	psi	20.748	189.737	0.122	Pass
Min.area of reinf, bot., x-direction	in^2	4.147	4.800		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	12.7		Pass
Min.area of reinf, bot., y-direction	in^2	6.221	7.800		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	11.4		Pass

Pad footing details

Length of footing: L_x = **12** ft

Width of footing: L_y = **8** ft

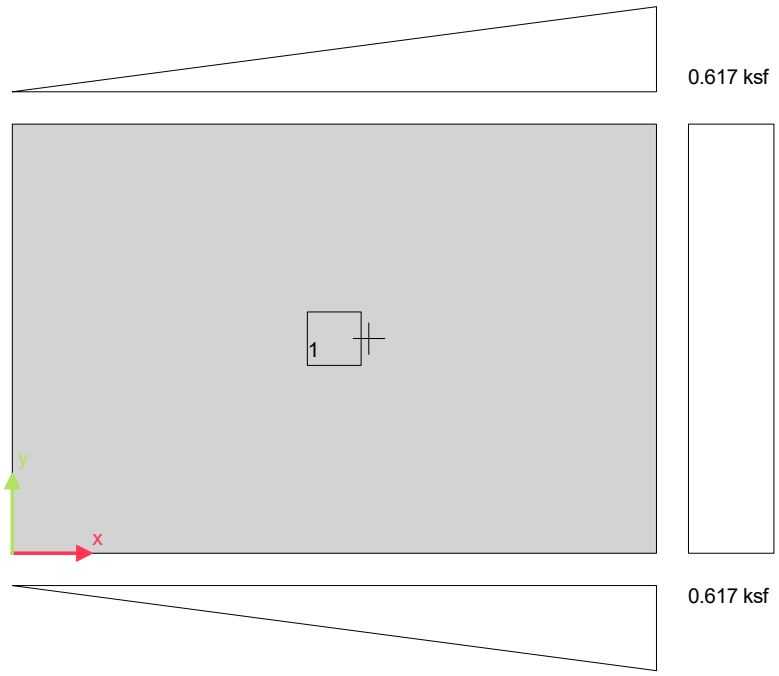
Footing area: $A = L_x \times L_y$ = **96** ft^2

Depth of footing: h = **24** in

Depth of soil over footing: h_{soil} = **12** in

Density of concrete: γ_{conc} = **150.0** lb/ft^3

Michael Baker INTERNATIONAL	Project: Creech DRP Phase 2				Job Ref.	
	Section: AGE Storage Canopy Footing				Sheet no./rev.: 2	
	Calc. by: DSC	Date: 11/6/2025	Chk'd by	Date	App'd by	Date



Column no.1 details

Length of column	l_{x1} = **12.00** in
Width of column	l_{y1} = **12.00** in
position in x-axis	x_1 = **72.00** 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} = **19.8** deg
Coefficient of base friction	tan(δ_{bb}) = **0.360**
Passive pressure coefficient (User defined)	$K_P = K_P$ = **3**
Passive dynamic pressure coefficient (Defined)	$K_{PE} = K_{PE}$ = **2.5**
Self weight	$F_{swt} = h \times \gamma_{conc}$ = **300** psf
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **13.8** kips
Live roof load in z	F_{Lrz1} = **30.2** kips
Wind load in z	F_{Wz1} = **-13.2** kips
Wind load in x	F_{Wx1} = **1.0** kips
Seismic load in x	F_{Ex1} = **2.0** kips
Wind load moment in x	M_{Wx1} = **107.8** kip_ft
Seismic load moment in x	M_{Ex1} = **23.2** kip_ft

Michael Baker INTERNATIONAL	Project: Creech DRP Phase 2				Job Ref.	
	Section: AGE Storage Canopy Footing				Sheet no./rev.: 3	
	Calc. by: DSC	Date: 11/6/2025	Chk'd by	Date	App'd by	Date

Footing analysis for soil and stability

Load combinations per ASCE 7-22

1.0D (0.187)
1.0D + 1.0L (0.187)
1.0D + 1.0Lr (0.292)
1.0D + 0.7S (0.187)
1.0D + 1.0R (0.187)
1.0D + 0.75L + 0.75Lr (0.266)
1.0D + 0.75L + 0.525S (0.187)
1.0D + 0.75L + 0.75R (0.187)
1.0D + 0.6W (0.350)
$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.236)
1.0D + 0.75L + 0.75Lr + 0.45W (0.331)
1.0D + 0.75L + 0.525S + 0.45W (0.263)
1.0D + 0.75L + 0.75R + 0.45W (0.263)
$(1.0 + 0.105 \times S_{DS})D + 0.75L + 0.1S + 0.525E$ (0.224)
0.6D + 0.6W (0.584)
$(0.6 - 0.14 \times S_{DS})D + 0.7E$ (0.130)

Combination 15 results: 0.6D + 0.6W

Forces on footing

Force in x-axis: $F_{dx} = \gamma_W \times F_{Wx1}$ = **0.6** kips

Force in z-axis: $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_W \times F_{Wz1}$ = **24.5** kips

Moments on footing

Moment in x-axis, about x is 0: $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1} + F_{Wx1} \times (h))$ = **212.7** kip_ft

Moment in y-axis, about y is 0: $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **97.9** kip_ft

Uplift verification

Vertical force: F_{dz} = **24.474** kips

PASS - Footing is not subject to uplift

Stability against overturning in x direction, moment about x is 0

Overturning moment: $M_{OTx0} = \gamma_W \times (F_{Wz1} \times x_1)$ = **-47.52** kip_ft

Resisting moment: $M_{Rx0} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_W \times (M_{Wx1} + F_{Wx1} \times (h))$ = **260.26** kip_ft

Factor of safety: $abs(M_{Rx0} / M_{OTx0})$ = **5.477**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in x direction, moment about x is L_x

Overturning moment: $M_{OTxL} = \gamma_W \times (F_{Wz1} \times (x_1 - L_x) + M_{Wx1} + F_{Wx1} \times (h))$ = **113.41** kip_ft

Resisting moment: $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times (x_1 - L_x))$ = **-194.36** kip_ft

Factor of safety: $abs(M_{RxL} / M_{OTxL})$ = **1.714**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is 0

Overturning moment $M_{OTy0} = \gamma_W \times (F_{Wz1} \times y_1)$ = **-31.68** kip_ft

Resisting moment $M_{Ry0} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1)$ = **129.58** kip_ft

Factor of safety $abs(M_{Ry0} / M_{OTy0})$ = **4.090**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against overturning in y direction, moment about y is L_y

Overturning moment $M_{OTyL} = \gamma_W \times (F_{Wz1} \times (y_1 - L_y))$ = **31.68** kip_ft

Resisting moment $M_{RyL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2)) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times (y_1 - L_y))$ = **-129.58** kip_ft

Factor of safety $abs(M_{RyL} / M_{OTyL})$ = **4.090**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction $F_{RFriction} = \max(F_{dz}, 0\ kN) \times \tan(\delta_{bb})$ = **8.811** kips

Stability against sliding in x direction

Resistance from passive soil pressure $F_{RxPass} = 0.5 \times K_P \times (h^2 + 2 \times h \times h_{soil}) \times L_y \times \gamma_{soil}$ = **11.52** kips

Total sliding resistance $F_{Rx} = F_{RFriction} + F_{RxPass}$ = **20.331** kips

Factor of safety $abs(F_{RTx} / F_{dx})$ = **33.88**

PASS - Sliding factor of safety exceeds the minimum of 1.50

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **32.308** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Length of bearing in x-axis $L'_{xd} = \min(L_x, 3 \times (L_x / 2 - abs(e_{dx})))$ = **119.076** in

Pad base pressures

q_1 = **0** ksf

q_2 = **0** ksf

q_3 = **0.617** ksf

q_4 = **0.617** ksf

Minimum base pressure $q_{min} = \min(q_1, q_2, q_3, q_4)$ = **0** ksf

Maximum base pressure $q_{max} = \max(q_1, q_2, q_3, q_4)$ = **0.617** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow_Gross}$ = **3** ksf

q_{max} / q_{allow} = **0.206**

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 = **4000** 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 (0.063)
1.2D + 1.6L + 0.5Lr (0.101)
1.2D + 1.6L + 0.3S (0.061)
1.2D + 1.6L + 0.5R (0.061)
1.2D + 1.0L + 1.6Lr (0.207)
1.2D + 1.0L + 1.0S (0.122)
1.2D + 1.0L + 1.6R (0.122)
1.2D + 1.6Lr + 0.5W (0.245)
1.2D + 1.0S + 0.5W (0.122)
1.2D + 1.6R + 0.5W (0.122)
1.2D + 1.0L + 0.5Lr + 1.0W (0.177)
1.2D + 1.0L + 0.3S + 1.0W (0.129)
1.2D + 1.0L + 0.5R + 1.0W (0.129)
$(1.2 + 0.2 \times S_{DS})D + 1.0L + 0.15S + 1.0E$ (0.122)
0.9D + 1.0W (0.127)
$(0.9 - 0.2 \times S_{DS})D + 1.0E$ (0.122)

Combination 8 results: 1.2D + 1.6Lr + 0.5W

Forces on footing

Ultimate force in x-axis	$F_{ux} = \gamma_W \times F_{Wx1}$ = **0.5** kips
Ultimate force in z-axis	$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1}$ = **106.5** kips

Moments on footing

Ultimate moment in x-axis, about x is 0	$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_{Lr} \times (F_{Lrz1} \times x_1) + \gamma_W \times (F_{Wz1} \times x_1 + M_{Wx1} + F_{Wx1} \times (h))$ = **694.1** kip_ft
Ultimate moment in y-axis, about y is 0	$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - I_{x1} \times I_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_{Lr} \times (F_{Lrz1} \times y_1) + \gamma_W \times (F_{Wz1} \times y_1)$ = **426.1** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis	$e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **6.186** in
Eccentricity of base reaction in y-axis	$e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

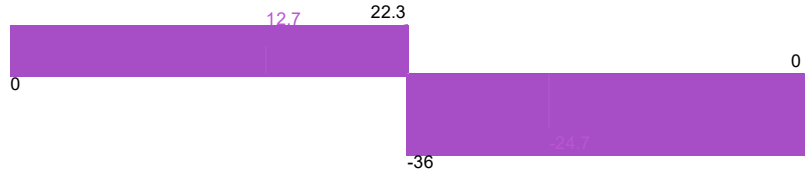
$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.824** ksf
$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.824** ksf
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.396** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.396** ksf

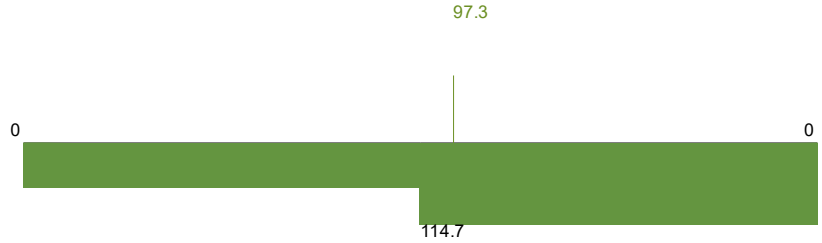
Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **0.824** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **1.396** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max} =$ **97.312** kip_ft

Tension reinforcement provided 8 No.7 bottom bars (12.7 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov} =$ **4.8** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h =$ **4.147** 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 $d = h - c_{nom_b} - \phi_{x.bot} / 2 =$ **20.562** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.882** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **1.038** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.05643**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003 =$ **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **482.912** 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 $\phi M_n = \phi_f \times M_n =$ **434.621** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.224**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

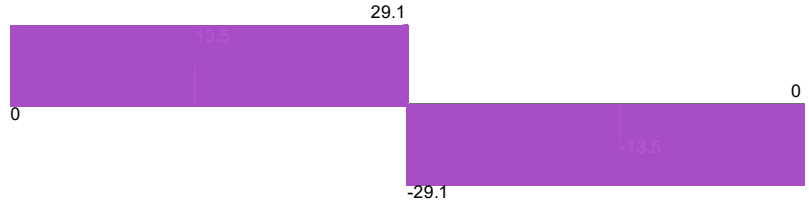
Ultimate shear force $V_{u.x} =$ **24.689** kips

Depth to reinforcement $d_v = \min(h - c_{nom_b} - \phi_{x.bot} / 2, h - c_{nom_t} - \phi_{x.top} / 2) =$ **20.562** in

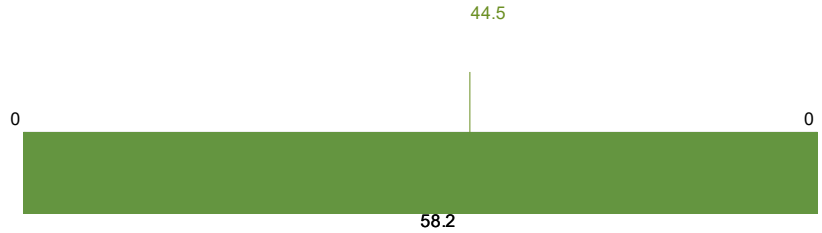
Size effect factor (22.5.5.1.3) $\lambda_s =$ **1**

Ratio of longitudinal reinforcement	$\rho_w = \min(A_{sx.top.prov}, A_{sx.bot.prov}) / (L_y \times d_v)$ = **0.00243**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1\ psi} \times L_y \times d_v)$ = **134.307** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **100.73** kips
	$V_{u.x} / \phi V_n$ = **0.245**
	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	$M_{u.y.max}$ = **44.513** kip_ft
Tension reinforcement provided	13 No.7 bottom bars (11.4 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **7.8** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **6.221** 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	$d = h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **19.687** in
Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.956** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **1.125** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04952**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **749.173** 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	$\phi M_n = \phi_f \times M_n$ = **674.256** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.066**
	PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor (13.3.3.3)	$\beta_f = L_x / L_y =$ **1.500**
Area of reinf. req. for uniform distribution (CRSI)	$A_{sreq} = (M_{u.y.max} / (\phi_f \times f_y \times (d - a / 2))) \times 2 \times \beta_f / (\beta_f + 1) =$ **0.618** in^2
	PASS - Reinforcement can be distributed uniformly

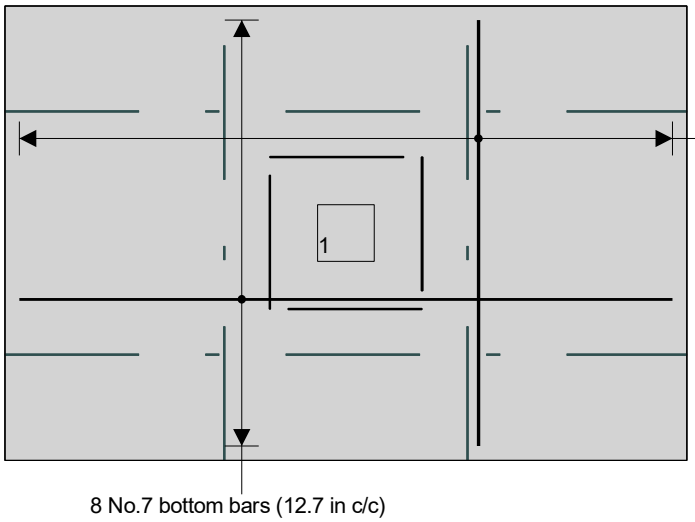
One-way shear design, y direction

Ultimate shear force	$V_{u.y} =$ **13.513** kips
Depth to reinforcement	$d_v = \min(h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom_t} - \phi_{y.top} / 2) =$ **19.687** in
Size effect factor (22.5.5.1.3)	$\lambda_s =$ **1**
Ratio of longitudinal reinforcement	$\rho_w = \min(A_{sy.top.prov}, A_{sy.bot.prov}) / (L_x \times d_v) =$ **0.00275**
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v, 5 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_x \times d_v) =$ **200.996** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n =$ **150.747** kips
	$V_{u.y} / \phi V_n =$ **0.090**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

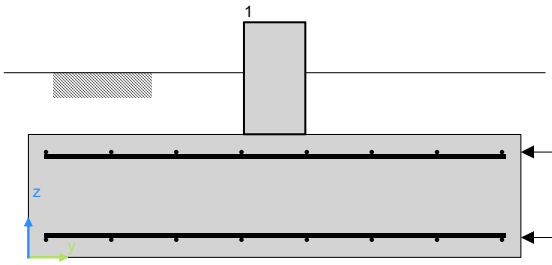
Depth to reinforcement	$d_{v2} =$ **20.125** in
Shear perimeter length (22.6.4)	$l_{xp} =$ **32.125** in
Shear perimeter width (22.6.4)	$l_{yp} =$ **32.125** in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **128.500** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} =$ **1032.016** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **888.016** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **1.110** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_{Lr} \times F_{Lrz1} + \gamma_W \times F_{Wz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **53.656** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) =$ **20.748** 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)	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} =$ **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} =$ **522.699** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} =$ **252.982** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **252.982** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n =$ **189.737** psi
	$v_{ug} / \phi v_n =$ **0.109**
	PASS - Design shear stress capacity exceeds ultimate shear stress load

Michael Baker INTERNATIONAL	Project: Creech DRP Phase 2				Job Ref.	
	Section: AGE Storage Canopy Footing				Sheet no./rev.: 9	
	Calc. by: DSC	Date: 11/6/2025	Chk'd by	Date	App'd by	Date



13 No.7 bottom bars (11.4 in c/c)
13 No.7 top bars (11.4 in c/c)

8 No.7 bottom bars (12.7 in c/c)
8 No.7 top bars (12.7 in c/c)



13 No.7 top bars (11.4 in c/c) (y direction)
8 No.7 top bars (12.7 in c/c) (x direction)

13 No.7 bottom bars (11.4 in c/c) (y direction)
8 No.7 bottom bars (12.7 in c/c) (x direction)