Project Details

Project Name: Thu Gbury Pole - V1Jb

Location: 216 Thompson St, South Glastonbury, CT 06073, USA

Unique ID: 1P-0-8TOP-XD-84-L-5Hx4W-STRUTS-241H

Dealer: ______________________________

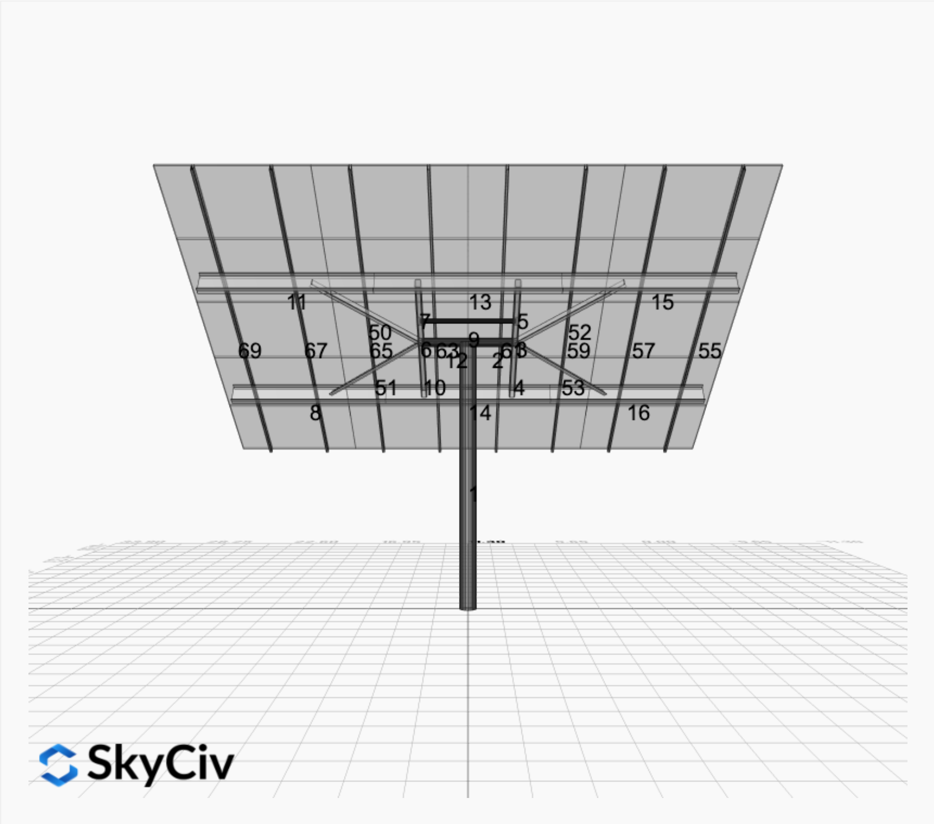
Date: Fri Aug 02 2024

Number of Modules: 20

Number of Poles: 1

Date Sold:

____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	22.93 ft
Winter Tilt Angle	35
Front Edge Clearance	6 ft

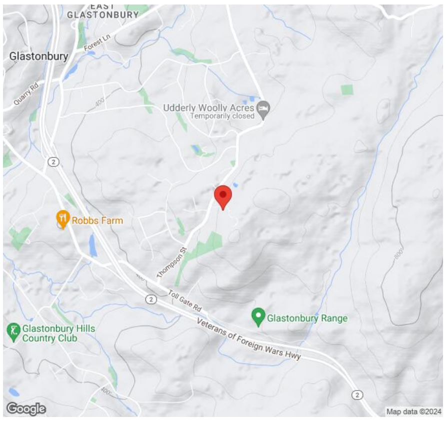
MT Solar Bill of Materials (1P-0-8TOP-XD-84-L-5Hx4W-STRUTS-241H)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-XD	H-Frame Assembly-XD	1
MTS-XD-Wing-84	84IN XD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	4

Rail Bill of Materials

Part	Qty
Rails (223in)	8
Rail Attachment	32
Module Mid Clamp	32
Module End Clamp	16
Ground Lug	4

Site Details:



Site Address: 216 Thompson St, South Glastonbury, CT 06073, USA

Array Specification

Duty Classification:	XD
Module Width:	44.65 in
Module Length:	67.80in
Number of Rows:	5
Number of Columns:	4
Total Number of Modules:	20
Winter Tilt Angle:	35
Front Edge Clearance:	6
Total Array Height at Tilt:	16.73 ft
Total Frame Length:	21.50 ft
Frame Weight:	1773 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	22.93 ft
Rail Length:	225.75 in
Rail Spacing:	2.82 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	11.40 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	60 x 60 in
Foundation Depth (below grade):	Pile 1: 7.00 ft
Foundation Volume:	6.481 y^3

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sandy_gravel
Site Location:	216 Thompson St, South Glastonbury, CT 06073, USA
Wind Speed:	125 mph
Snow Load:	30 psf

Design Disclaimer

This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input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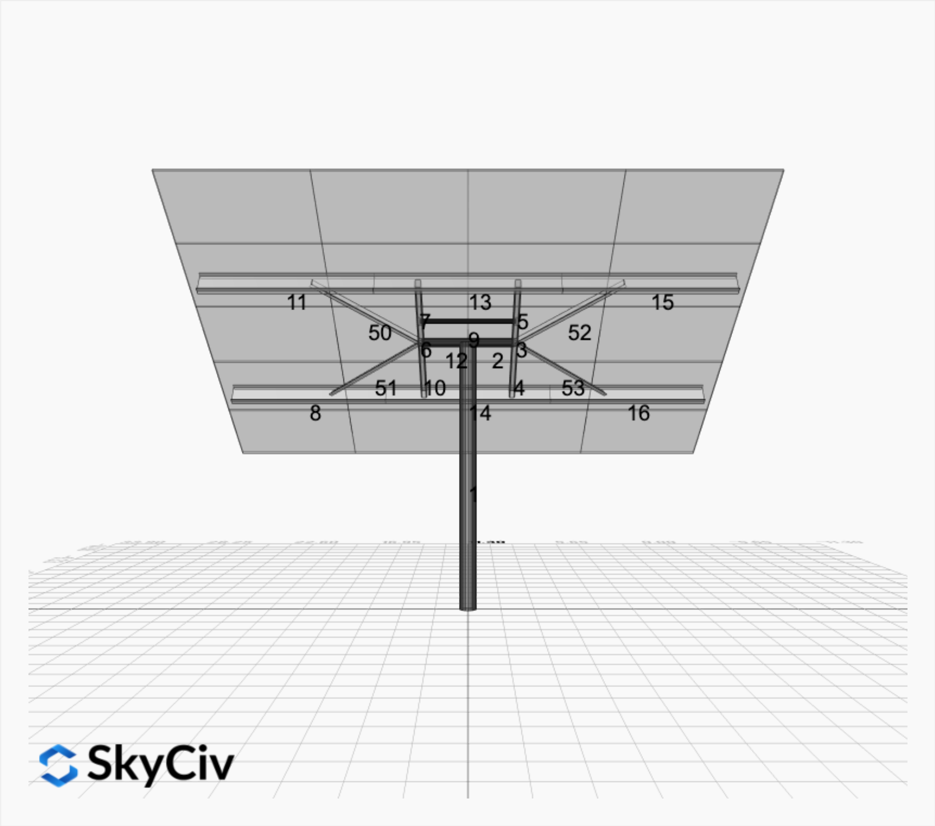
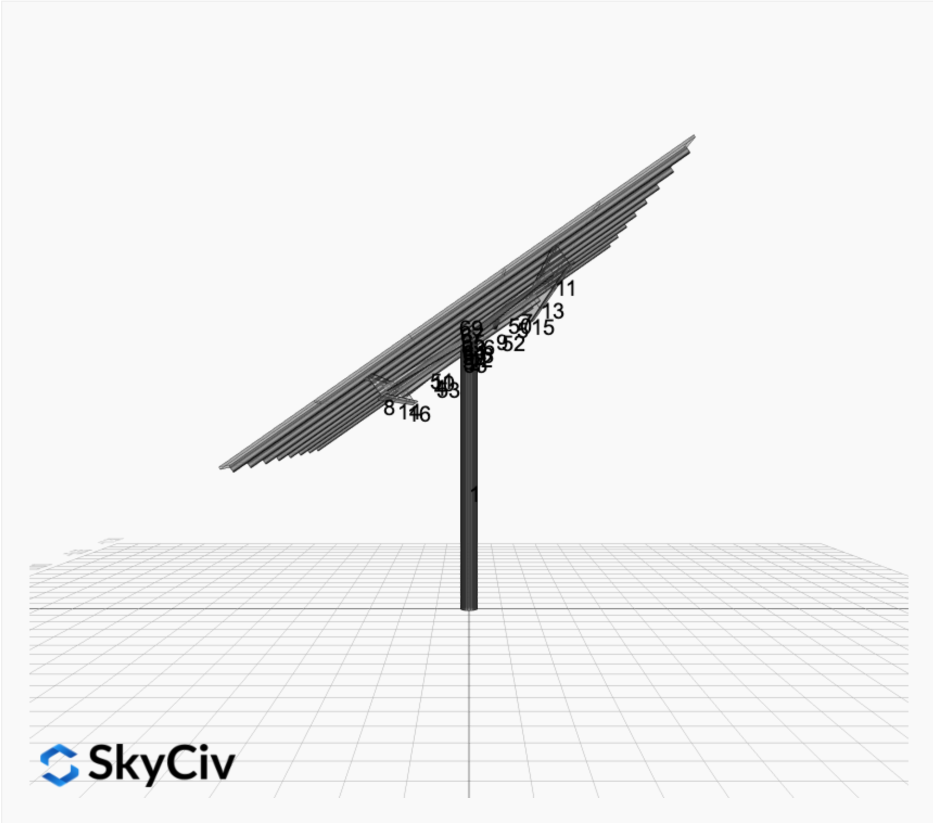
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)

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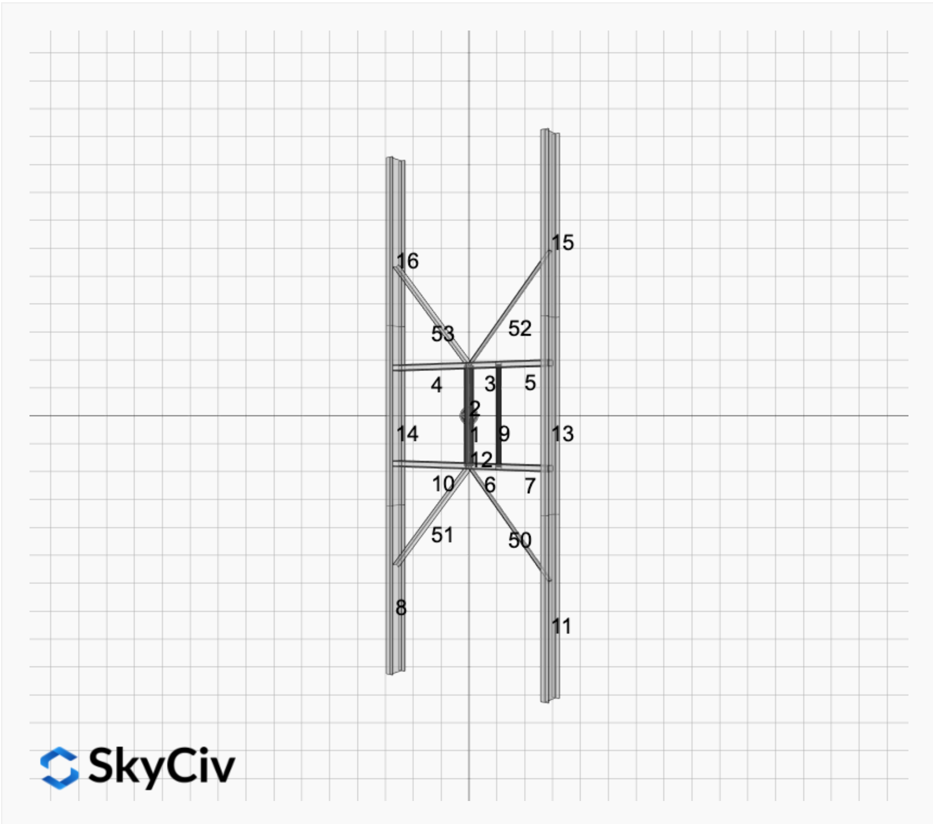
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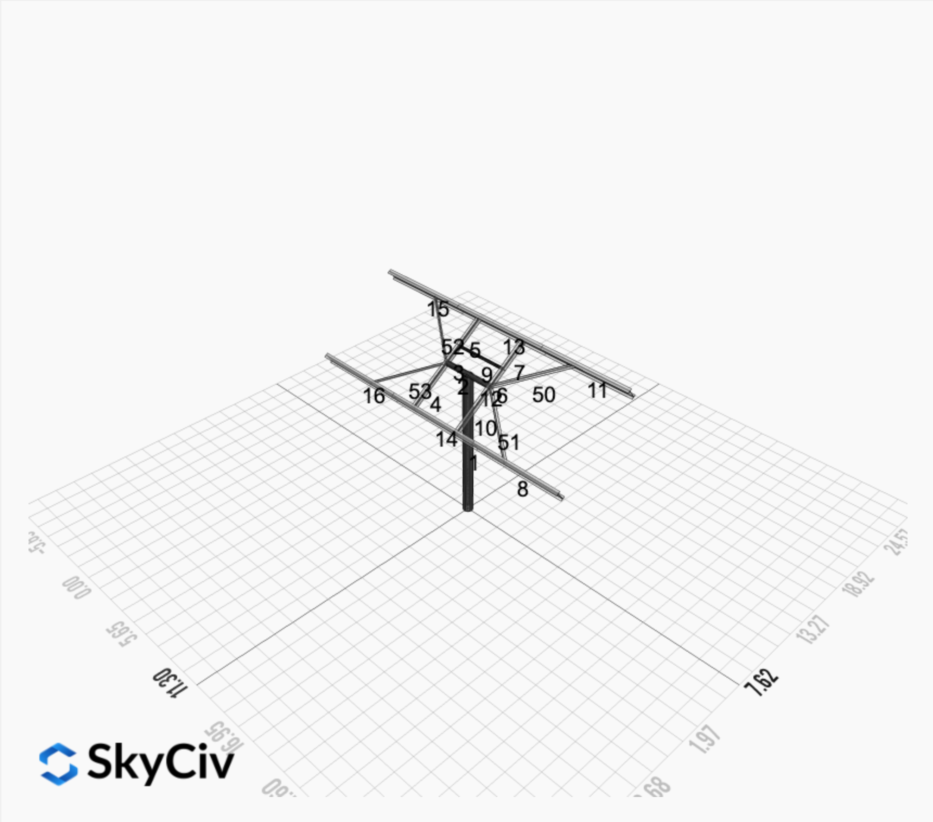
SkyCiv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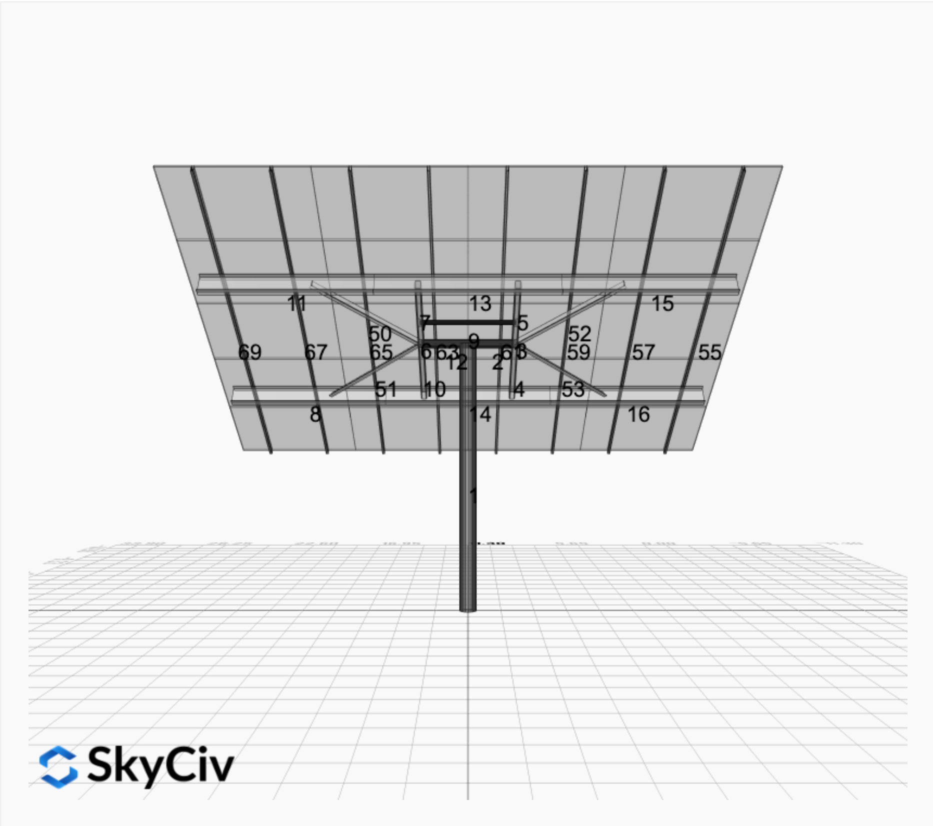
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SkyCiv

Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	3.3200	0.0000	-0.0000	-0.0000	0.0327
ULS: 2. D + L	0.0000	3.3200	0.0000	-0.0000	-0.0000	0.0327
ULS: 3. D + (S or Lr or R)	0.0000	7.1455	0.0000	-0.0000	-0.0000	0.0544
ULS: 3. D + (S or Lr or R)	0.0000	3.3200	0.0000	-0.0000	-0.0000	0.0327
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.1891	0.0000	-0.0000	-0.0000	0.0490
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.3200	0.0000	-0.0000	-0.0000	0.0327
ULS: 5b. D + 0.7E	0.0000	3.3200	0.0000	-0.0000	-0.0000	0.0327
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.1891	0.0000	-0.0000	-0.0000	0.0490
ULS: 8. 0.6D + 0.7E	0.0000	1.9920	0.0000	-0.0000	-0.0000	0.0196
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.1364	10.6555	0.0000	-0.0000	-0.0000	60.6077
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.1364	10.6555	0.0000	-0.0000	-0.0000	60.6077
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.3339	-2.8694	0.0000	-0.0000	-0.0000	-48.3422
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.6115	-1.8378	0.0000	-0.0000	-0.0000	-54.7321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8523	11.6908	0.0000	-0.0000	-0.0000	45.4802
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.8523	11.6908	0.0000	-0.0000	-0.0000	45.4802
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2504	1.5471	0.0000	-0.0000	-0.0000	-36.2322
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7087	2.3207	0.0000	-0.0000	-0.0000	-41.0247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8523	8.8217	0.0000	-0.0000	-0.0000	45.4640
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.8523	8.8217	0.0000	-0.0000	-0.0000	45.4640
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2504	-1.3221	0.0000	-0.0000	-0.0000	-36.2485
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7087	-0.5484	0.0000	-0.0000	-0.0000	-41.0409
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.1364	9.3276	0.0000	-0.0000	-0.0000	60.5946
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.1364	9.3276	0.0000	-0.0000	-0.0000	60.5946
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.3339	-4.1974	0.0000	-0.0000	-0.0000	-48.3553
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.6115	-3.1658	0.0000	-0.0000	-0.0000	-54.7452

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	18.1227
Shear X	-8.5607
Shear Z	0.0000
Moment X	0.0014
Moment Y (Twist)	0.0017
Moment Z	102.4513

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	11.6908
Shear X	-5.1364
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	60.6077

REFERENCES	CALCULATIONS	RESULTS

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : Thu Gbury Pole - V1Jb
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **216 Thompson St, South Glastonbury, CT 06073, USA** categorized as **Exposure B** (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Main Wind Force Resisting System (MWFRS) - is based on the Directional Procedure (Chapter 27) of ASCE 7. Moreover, the structure is classified as **Risk Category I**. The location is elevated at **662.04 ft** above mean sea level.

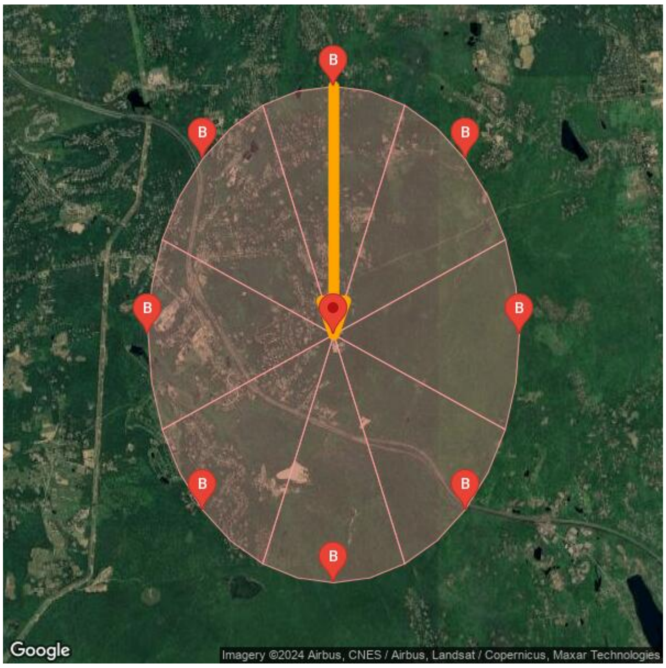


Figure 1. Site location.

Parameter	Value
Building Length, L	15.24 ft
Building Width, B	22.60 ft
Mean Roof Height, h	11.39 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	35.00°
Structure Type	Main Wind Force Resisting System (MWFRS)
Wind Blockage	Empty Under

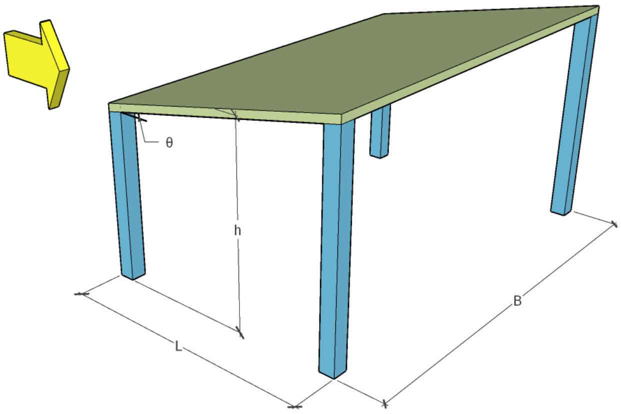


Figure 2. Building parameters.

Figure 26.5-1

Basic Wind Speed, V

Wind speed for the address is **125 mph (defined by the user)** for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.

$V = 125$ mph **(defined by the user)**

Wind Directionality Factor, K_d

Table 26.6-1 $K_d = 0.85$ - Wind Directionality Factor
For buildings

$K_d = 0.85$

Topographic Factor, K_{zt}

Section 26.8.1 $K_{zt} = 1$ - Topographic Factor
For the selected wind source direction, either the terrain is relatively a flat surface or the structure is outside the local topographic zones.

For calculating the topographic factor, the detected topography for the selected wind source direction is Flat.

$K_{zt} = 1$

Ground Elevation Factor, K_e

Section 26.9 K_e - Ground Elevation Factor

$$K_e = e^{-0.0000362\ E}$$

$$K_e = 0.97632$$

$$\text{Where } E = \text{Site Elevation} = 662.04\ \text{ft}$$

Section 26.9 $K_e = 0.97632$ - Ground Elevation Factor

$K_e = 0.97632$

Velocity Pressure Exposure Coefficient, K_z

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $z < 15ft$

$$K_z = 2.01 \times (15/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $15ft \le z \le z_g$

$$K_z = 2.01 \times (z/z_g)^{2/\alpha}$$

Table 26.11-1 $\alpha = 7$
Table 26.11-1 $z_g = 1200$ ft
Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $z < 15ft$

$$K_z = 2.01\ (15/z_g)^{2/\alpha}$$

$$K_z = 0.57472$$

$$\text{Where } z_g = 1200\ \text{ft}$$
$$\alpha = 7$$

Level	Elevation (ft)	K_z
h	11.39	0.575

$\alpha = 7$
$z_g = 1200$ ft

Velocity Pressure, q_h

For the selected wind source direction.

Section 26.10.2 q_h - Velocity Pressure at h

$$q_h = 0.00256\ K_z\ K_{zt}\ K_d\ K_e\ V^2$$

$$q_h = 19.078\ \text{psf}$$

$$\text{Where } K_z = \text{Velocity Pressure Exposure Coefficient} = 0.57472$$
$$K_{zt} = \text{Topographic Factor} = 1$$
$$K_d = \text{Wind Directionality Factor} = 0.85$$
$$V = \text{Basic Wind Speed} = 125\ \text{mi/h}$$
$$K_e = \text{Ground Elevation Factor} = 0.97632$$

Velocity Pressure for All Directions

Section 26.8 K_{zt} - Topographic Factor

$$K_{zt} = (1 + K_1 \times K_2 \times K_3)^2 \ge 1.0$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $15ft \le z \le z_g$

$$K_z = 2.01 \times (z/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $z < 15ft$

$$K_z = 2.01 \times (15/z_g)^{2/\alpha}$$

Section 26.10.2 q_h - Velocity Pressure at h

$$q_h = 0.00256 \times K_{z,h} \times K_{zt} \times K_d \times K_e \times V^2$$

Direction	Exposure Category	K_z @ $h = 11.395ft$	K_{zt}	K_d	K_e	V (mph)	q_h (psf)
N	B	0.575	1.000	0.850	0.976	125.000	19.078
NE	B	0.575	1.000	0.850	0.976	125.000	19.078
E	B	0.575	1.000	0.850	0.976	125.000	19.078
SE	B	0.575	1.000	0.850	0.976	125.000	19.078
S	B	0.575	1.000	0.850	0.976	125.000	19.078
SW	B	0.575	1.000	0.850	0.976	125.000	19.078

W	B	0.575	1.000	0.850	0.976	125.000	19.078
NW	B	0.575	1.000	0.850	0.976	125.000	19.078

Net Pressure Coefficients, C_N

Figure 27.3-4 to 27.3-7

The net pressure coefficients, C_N, are calculated using Figures 27.3-4 to 27.3-7 of ASCE 7-16 - Clear Wind Flow - as shown in Table below.

Direction	Surface	C_N Case A	C_N Case B
0	Windward	-1.800	-2.433
	Leeward		-0.567
180	Windward	2.100	2.667
	Leeward	2.167	1.067
90	≤ h from windward edge	-0.800	0.800
	h to 2h from windward edge	-0.600	0.500
	> 2h from windward edge	-0.300	0.300

Gust Effect Factor, G

Section 26.11.1

$G = 0.85$ - Gust Effect Factor

The structure is assumed to be rigid.

$G = 0.85$

Design Wind Pressures (MWFRS)

Section 27.3.2

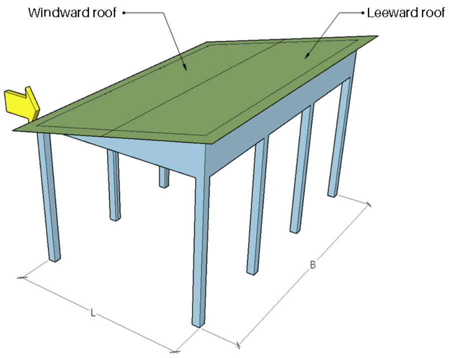
p - Design Wind Pressure

For open buildings

$$p = q_h \times G \times C_N$$

For Wind Pressure - 0°

Direction	Surface	q_h (psf)	G	C_N Case A	C_N Case B	p Case A (psf)	p Case B (psf)
0	Windward	19.078	0.850	-1.800	-2.433	-29.189	-39.459
	Leeward	19.078	0.850		-0.567	-29.189	-9.189
180	Windward	19.078	0.850	2.100	2.667	34.054	43.243
	Leeward	19.078	0.850	2.167	1.067	35.135	17.297



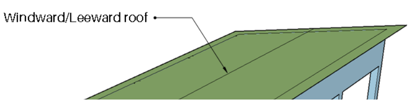
Wind along L - 0°

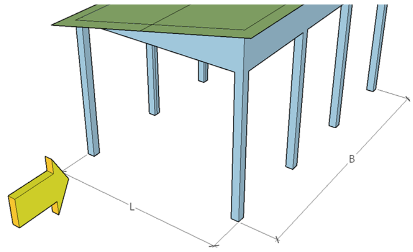
Service Wind Pressure - 0°/180°

Direction	Surface	p Case A (psf)	p Case B (psf)
0	Windward	-17.513	-23.675
	Leeward	-17.513	-5.513
180	Windward	20.432	25.946
	Leeward	21.081	10.378

For Wind Pressure - 90°

Direction	Surface	q_h (psf)	G	C_N Case A	C_N Case B	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	19.078	0.850	-0.800	0.800	-12.973	12.973
	h to 2h from windward edge	19.078	0.850	-0.600	0.500	-9.730	8.108
	> 2h from windward edge	19.078	0.850	-0.300	0.300	-4.865	4.865





Wind along B - 90°

Service Wind Pressure - 90°

Direction	Surface	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	-7.784	7.784
	h to 2h from windward edge	-5.838	4.865
	> 2h from windward edge	-2.919	2.919

Section 27.3.2
Section 28.3.5

In addition to the roof pressures for 90°, an additional horizontal wind load on open building should be calculated for wind pressures parallel to the ridge in accordance with Section 28.3.5. We will assume $K_S = 1.0$ and should be adjusted and be reduced based on the actual solidity ratio ϕ and number of frames n - See Figure 28.3-2.

Section 27.3.2
Section 28.3.5

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

$$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \times K_B \times K_S$$

Section 27.3.2
Section 28.3.5

K_B - Frame Width Factor

For $L < 100ft$, $K_B = 1.8 - 0.01L$. *Otherwise,* $K_B = 0.8$.

$$K_B = 1.8 - 0.01 * L \leq 0.8$$

$$K_B = 1.6476$$

$$\text{Where } L = \text{Building Length} = 15.24 \text{ ft}$$

Section 28.3.5

K_S - Shielding Factor

$$K_S = 0.6 + 0.073 \times (n-1) + \left(1.25 \times \phi^{1.8}\right)$$

Section 28.3.5

$K_S = 1$ - Shielding Factor — $K_S = 1$

Assumed to be equal to 1.0 and should be adjusted based on the actual wall solidity ratio ϕ and number of frames n.

Figure 28.3-1
Section 28.3.5

$(GC_{pf})_{windward} = 0.4$ — $(GC_{pf})_{windward} = 0.4$

Using Zone 5 from Figure 28.3-1

Figure 28.3-1
Section 28.3.5

$(GC_{pf})_{leeward} = -0.29$ — $(GC_{pf})_{leeward} = -0.29$

Using Zone 6 from Figure 28.3-1

Section 27.3.2
Section 28.3.5

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

REFERENCES	CALCULATIONS	RESULTS

Snow Load Calculations based on ASCE 7-16

Design Information :

Project Name : Thu Gbury Pole - V1Jb
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **216 Thompson St, South Glastonbury, CT 06073, USA** categorized as **Risk Category I**. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at **662 ft** above mean sea level.

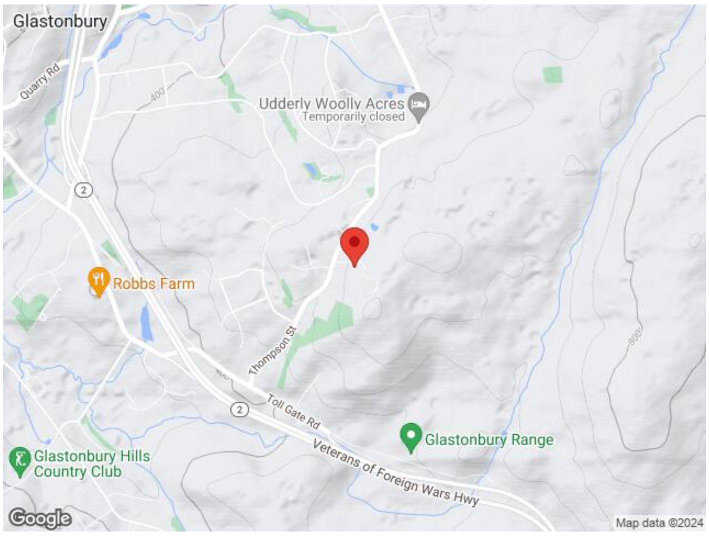


Figure 1. Site location.

Parameter	Value
Building Length, L	15.240 ft
Building Width, B	22.600 ft
Mean Roof Height, h	11.395 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	35.000°

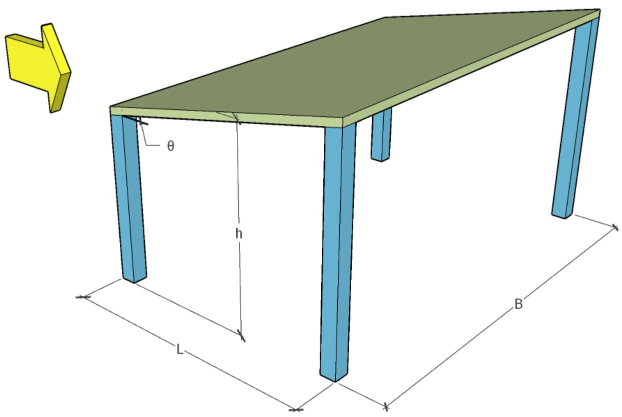


Figure 2. Building parameters.

Figure 7.6-2 S

Where roof slope is equal to $\frac{1}{S}$

$$S = \frac{1}{tan\theta}$$

$$S = 1.4281$$

$$\text{Where } \theta = \text{Angle of slope of roof} = 35\ ^{\circ}$$

Ground Snow Load, p_g

From Fig. 7.2-1 and Table 7.2-1 ATC/ASCE 7 Hazard Tools *Google Maps*	$p_g = 30$ psf - Ground Snow Load *User-defined value* $E = 662.04$ ft - Ground Elevation above mean sea level	$p_g = 30$ psf $E = 662.04$ ft
Table 7.3-1 of ASCE 7-16	**Exposure Factor, C_e** $C_e = 0.9$ - Exposure Factor *For Terrain Category **Surface Roughness B** with exposure condition specified as Fully Exposed.*	$C_e = 0.9$
Table 7.3-2 of ASCE 7-16	**Thermal Factor, C_t** $C_t = 1.2$ - Thermal Factor *For Thermal Condition equal to **Unheated and open air structures**.*	$C_t = 1.2$
Table 1.5-1 of ASCE 7-16	**Importance Factor, I_s** $I_s = 0.8$ - Importance Factor *For Risk Category I.*	$I_s = 0.8$
Section 7.3 Equation 7.3-1	**Flat Roof Snow Load, p_f** From Section 7.3 - Equation 7.3-1, the flat roof snow load p_f can be calculated as follows: p_f - Flat Roof Snow Load $$p_f = 0.7\, C_e\, C_t\, I_s\, p_g$$ $$p_f = 18.144 \text{ psf}$$ Where C_e = Exposure Factor = 0.9 C_t = Thermal Factor = 1.2 I_s = Importance Factor = 0.8 p_g = Ground Snow Load = 30 psf	
Section 7.10	**Rain-on-snow Surcharge Load, p_r** p_r - Rain-on-snow Surcharge Load *For $0 < p_g \leq 20$ and $\theta < \frac{W}{50}$, $p_r = 5$ psf. Otherwise, $p_r = 0$ psf. This applies only to sloped roof (balanced) snow load case.* $$p_r = 5 : 0 < p_g \leq 20, \theta < \frac{W}{50}$$ $$0 : allcases$$ $$p_r = 0 \text{ psf}$$ Where p_g = Ground Snow Load = 30 psf θ = Angle of slope of roof = 35 $^\circ$ W = Horizontal distance from eave to ridge - equal to [mathin] L [mathin] for monoslope/monopitch roof. = 15.24 ft	
Equation 7.7-1	**Snow Density, γ** γ - Snow Density $$\gamma = 0.13\, p_g + 14 \leq 30$$ $$\gamma = 17.9 \text{ lbf/ft}^3$$ Where p_g = Ground Snow Load = 30 psf	
Table 7.3-2 of ASCE 7-16 *Figure 7-2c*	**Roof Slope Factor (Balanced), C_s** $C_t = 1.2$ - Thermal Factor *For Thermal Condition equal to **Unheated and open air structures**.* $\theta = 35\ ^\circ$ - Angle of slope of roof $C_s = 0.63636$ - Roof Slope Factor *For roof pitch angle equal to 35.000° and Thermal Condition equal to **Unheated and open air structures**.*	$C_t = 1.2$ $\theta = 35\ ^\circ$ $C_s = 0.63636$
Section 7.4 *Section 7.4*	**Sloped Roof Snow Load, p_s** p_s - Sloped Roof Snow Load *Assumed to act on the horizontal projection of the surface* $$p_s = C_s\, p_f$$ $$p_s = 11.546 \text{ psf}$$ Where p_f = Flat Roof Snow Load = 18.144 psf C_s = Roof Slope Factor = 0.63636 p_s - Sloped Roof Snow Load with Rain Load *Assumed to act on the horizontal projection of the surface* $$p_s = C_s\, p_f + p_r$$ $$p_s = 11.546 \text{ psf}$$ Where p_f = Flat Roof Snow Load = 18.144 psf C_s = Roof Slope Factor = 0.63636 p_r = Rain-on-snow Surcharge Load = 0 psf	

Section 7.3.4	**Minimum Roof Snow Load, p_m** $p_m = 0 \text{ psf}$ - Minimum Roof Snow Load *For monoslope, gable, or hip roof with pitch angle $\theta \geq 15^\circ$.*	$p_m = 0 \text{ psf}$

Project Details

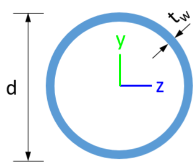
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Project Name:	Thu Gbury Pole - V1Jb
Unit System:	imperial

Design Input Information

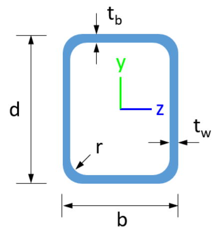
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

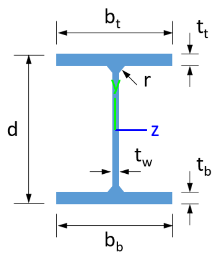
Section Dimensions



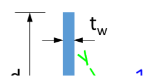
ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
10	8in Pipe Sch 80	8.63	0.50					

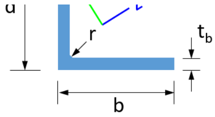


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30





ID	Name	d (in)	t_w (in)	b (in)	t_b (in)	r (in)		
34	L3x2x3/16	3.00	0.19	2.00	0.19	0.31		

Section Properties

ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60
34	L3x2x3/16	0.92	0.01	0.17	0.98	0.01	0.33	0.82

Member Properties

Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	10	23.93	23.93	11.40	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,2.77,1.67,1.67,1.66,1.64,1.67,1.67,1.70,1.67,1.67,1.67,1.66,1.74,1.67,1.67,1.66,1.65	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.69,1.64,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.69,1.64,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67	300	200	1
8	20	7.00	7.00	7.00	2.32,2.32,2.32,2.31,2.32,2.32,2.32,2.32,2.31,3.18,2.31,2.31,2.31,2.33,2.32,2.32,2.30,2.30,2.32,2.32,2.31,2.31,2.31,2.31,2.31,2.32	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,2.89,1.67,1.67,1.66,1.64,1.67,1.67,1.70,1.67,1.67,1.67,1.66,1.74,1.67,1.67,1.66,1.65	300	200	1
11	20	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.32,2.33,2.32,2.32,2.31,2.32,2.32,2.32,2.31,2.31,2.32,2.32,2.30,2.37,2.32,2.32,2.31,2.31,2.32,2.32,2.31,2.31	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.22,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
14	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.00,1.02,1.02,1.02,1.01,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.75,1.02,1.02,1.02,1.01	300	200	1
15	20	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.32,2.33,2.32,2.32,2.31,2.32,2.32,2.32,2.31,2.31,2.32,2.32,2.30,2.37,2.32,2.32,2.31,2.31,2.32,2.32,2.31,2.31	300	200	1
16	20	7.00	7.00	7.00	2.32,2.32,2.32,2.31,2.32,2.32,2.32,2.32,2.31,3.20,2.31,2.31,2.31,2.33,2.32,2.32,2.30,2.30,2.32,2.32,2.31,2.31,2.31,2.31,2.31,2.32	300	200	1
50	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
51	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
52	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
53	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	574.32	277.36	123.94	123.94	172.30	172.30
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	68.92	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	68.92	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	31.06	6.46	56.26	44.91
14	159.30	97.43	30.62	6.46	56.26	44.91
15	159.30	68.92	46.90	6.46	56.26	44.91
16	159.30	68.92	46.90	6.46	56.26	44.91
50	41.27	8.45	1.63	0.88	15.23	10.15
51	41.27	8.45	1.63	0.88	15.23	10.15
52	41.27	8.45	1.63	0.88	15.23	10.15
53	41.27	8.45	1.63	0.88	15.23	10.15

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P, M_z, M_y)	Worst LC	KL/r	δ	Status
1	0.065	0.827	0.000	0.050	0.000	0.859	#13	0.499	Not Required	Pass
2	0.005	0.565	0.307	0.116	0.057	0.873	#13	0.054	Not Required	Pass
3	0.001	0.903	0.104	0.090	0.044	0.961	#13	0.046	Not Required	Warn
4	0.001	0.876	0.031	0.087	0.002	0.894	#13	0.082	Not Required	Pass
5	0.001	0.560	0.025	0.090	0.010	0.573	#13	0.076	Not Required	Pass
6	0.001	0.903	0.104	0.090	0.044	0.961	#13	0.046	Not Required	Warn
7	0.001	0.560	0.025	0.090	0.010	0.574	#13	0.076	Not Required	Pass
8	0.019	0.228	0.190	0.054	0.013	0.274	#21	0.535	Not Required	Pass
9	0.021	0.091	0.064	0.001	0.000	0.161	#13	0.137	Not Required	Pass
10	0.001	0.876	0.031	0.087	0.002	0.894	#13	0.082	Not Required	Pass
11	0.008	0.235	0.190	0.056	0.013	0.270	#21	0.357	Not Required	Pass
12	0.005	0.565	0.307	0.116	0.057	0.873	#13	0.054	Not Required	Pass
13	0.008	0.574	0.037	0.070	0.007	0.586	#13	0.204	Not Required	Pass
14	0.014	0.565	0.026	0.068	0.006	0.572	#13	0.306	Not Required	Pass
15	0.008	0.235	0.190	0.056	0.013	0.270	#21	0.357	Not Required	Pass
16	0.019	0.228	0.190	0.054	0.013	0.274	#21	0.357	Not Required	Pass
50	0.207	0.010	0.001	0.003	0.001	0.216	#21	0.783	Not Required	Pass
51	0.042	0.006	0.013	0.002	0.002	0.036	#21	0.522	Not Required	Pass
52	0.207	0.010	0.001	0.003	0.001	0.216	#24	0.783	Not Required	Pass
53	0.042	0.006	0.013	0.002	0.002	0.036	#24	0.522	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comresssion)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

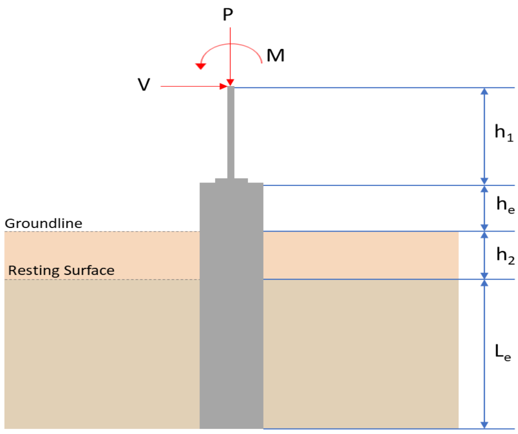
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 60\ \mathrm{in}$ - Pile width
$D = 60\ \mathrm{in}$ - Pile depth
$L = 7\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sandy gravel and/or gravel	3000.000	200.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	11.691	18.123
V_x (kip)	-5.136	-8.561
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.001
M_z (kipft)	60.608	102.451

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_x}{1.57\ D}$$

$$H_o = \frac{(-5.136\ \mathrm{kip})}{1.57 \times (60\ \mathrm{in})}$$

$$H_o = -0.65427\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_z + (V_x\,H)}{1.57\,D}$$

$$M_o = \frac{(60.608\text{ kipft}) + ((-5.136\text{ kip}) \times (0\text{ ft}))}{1.57 \times (60\text{ in})}$$

$$M_o = 7.7208\text{ kipft/ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:
$L_{e,x} = 6.4823\text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0\text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

$$L_{e,req} = MAX[L_{e,x}, L_{e,z}]$$

$$L_{e,req} = MAX[(6.4823\text{ ft}), (0\text{ ft})]$$

$$L_{e,req} = 6.482\text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7\text{ ft}) - (0\text{ ft}) - (0\text{ ft})$$

$$L_e = 7\text{ ft}$$

$Ratio$ - Embedded depth

$$Ratio = \frac{L_{e,req}}{L_e}$$

$$Ratio = \frac{(6.482\text{ ft})}{(7\text{ ft})}$$

$$Ratio = 0.926$$

Status: **PASS**
Ratio: **0.930**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

$$A = (60\text{ in}) \times (60\text{ in})$$

$$A = 25\text{ ft}^2$$

q - End-bearing pressure

$$q = \frac{P_v}{A}$$

$$q = \frac{(11.691\text{ kip})}{(25\text{ ft}^2)}$$

$$q = 0.46764\text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

$$Ratio = \frac{q}{q_a}$$

$$Ratio = \frac{(0.46764\text{ kip/ft}^2)}{(3000\text{ psf})}$$

$$Ratio = 0.15588$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

$$L/D = \frac{L}{D}$$

$$L/D = \frac{(7\ \mathrm{ft})}{(60\ \mathrm{in})}$$

$$L/D = 1.4$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.65427\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 7.7208\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

$$a = \frac{(4\,M_o\,L_e) + \left(3\,H_o\,L_e^2\right)}{(6\,M_o) + (4\,H_o\,L_e)}$$

$$a = \frac{(4 \times (7.7208\ \mathrm{kipft/ft}) \times (7\ \mathrm{ft})) + \left(3 \times (-0.65427\ \mathrm{kip/ft}) \times (7\ \mathrm{ft})^2\right)}{(6 \times (7.7208\ \mathrm{kipft/ft})) + (4 \times (-0.65427\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))}$$

$$a = 4.832\ \mathrm{ft}$$

p - Earth pressure against the pile at distance a/2 from resting surface,

$$p = \frac{0.75\ [(4\,M_o) + (3\,H_o\,L_e)]^2}{L_e^2\ [(3\,M_o) + (2\,H_o\,L_e)]}$$

$$p = \frac{0.75 \times [(4 \times (7.7208\ \mathrm{kipft/ft})) + (3 \times (-0.65427\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]^{\ 2}}{(7\ \mathrm{ft})^2 \times [(3 \times (7.7208\ \mathrm{kipft/ft})) + (2 \times (-0.65427\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]}$$

$$p = 0.32126\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{6\ [(2\,M_o) + (H_o\,L_e)]}{L_e^2}$$

$$s = \frac{6 \times [(2 \times (7.7208\ \mathrm{kipft/ft})) + ((-0.65427\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]}{(7\ \mathrm{ft})^2}$$

$$s = 1.33\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

p_a - Allowable lateral soil pressure at depth a/2,

$$p_a = R\,\frac{a}{2}$$

$$p_a = (200\ \mathrm{psf/ft}) \times \frac{(4.832\ \mathrm{ft})}{2}$$

$$p_a = 0.4832\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

$$Ratio = \frac{p}{p_a}$$

$$Ratio = \frac{\left(0.32126\ \mathrm{kip/ft^2}\right)}{\left(0.4832\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.66486$$

Status: **PASS**
Ratio: **0.660**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (200\ \mathrm{psf/ft}) \times (7\ \mathrm{ft})$$

$$p_s = 1.4\ \mathrm{kip/ft^2}$$

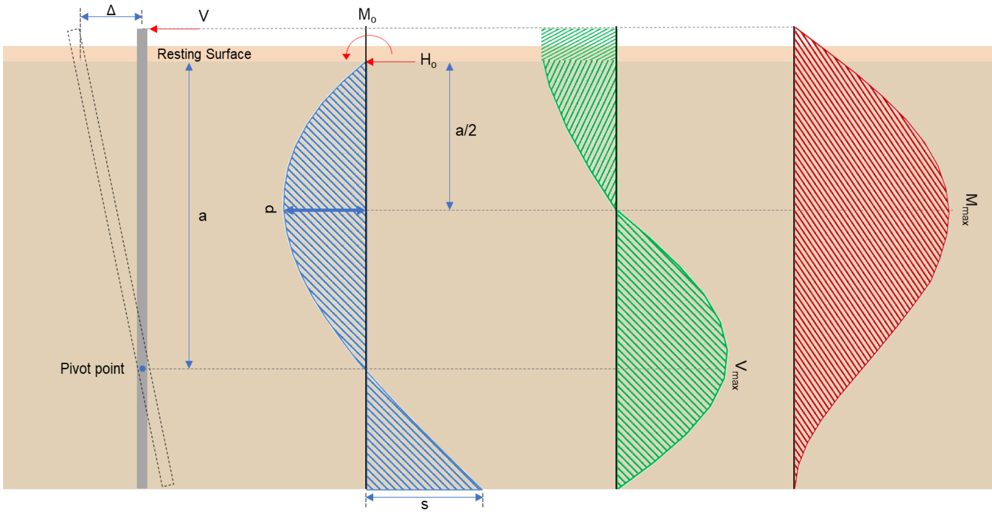
$Ratio$ - Lateral soil capacity

$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(1.33\ \mathrm{kip/ft^2}\right)}{\left(1.4\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.95$$

Status: PASS
Ratio: 0.950



Pile Rotation | Pressure Diagram | Shear Diagram | Moment Diagram

Shear force and Bending moment (x-direction, LRFD)

H_o - Lateral force per length of pile,

$$H_o = \frac{V_x}{1.57\,D}$$

$$H_o = \frac{(-8.561\text{ kip})}{1.57 \times (60\text{ in})}$$

$$H_o = -1.0906\text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_z + (V_x\,H)}{1.57\,D}$$

$$M_o = \frac{(102.45\text{ kipft}) + ((-8.561\text{ kip}) \times (0\text{ ft}))}{1.57 \times (60\text{ in})}$$

$$M_o = 13.051\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

$$E = \frac{M_o}{H_o}$$

$$E = \frac{(13.051\text{ kipft/ft})}{(-1.0906\text{ kip/ft})}$$

$$E = 11.967\text{ ft}$$

a - Distance from resting surface to pivot point,

$$a = \frac{(4\,M_o\,L_e) + \left(3\,H_o\,L_e^2\right)}{(6\,M_o) + (4\,H_o\,L_e)}$$

$$a = \frac{(4 \times (13.051\text{ kipft/ft}) \times (7\text{ ft})) + \left(3 \times (-1.0906\text{ kip/ft}) \times (7\text{ ft})^2\right)}{(6 \times (13.051\text{ kipft/ft})) + (4 \times (-1.0906\text{ kip/ft}) \times (7\text{ ft}))}$$

$$a = 4.8303\text{ ft}$$

V_{max} - Max shear force located at depth a,

$$V_{max} = (H_o\,D)\left[1 - \left[3\left(\frac{4\,E}{L_e} + 3\right)\left(\frac{a}{L_e}\right)^2\right] + \left[4\left(\frac{3\,E}{L_e} + 2\right)\left(\frac{a}{L_e}\right)^3\right]\right]$$

$$V_{max} = ((-1.0906\text{ kip/ft}) \times (60\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.967\text{ ft})}{(7\text{ ft})} + 3\right) \times \left(\frac{(4.8303\text{ ft})}{(7\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.967\text{ ft})}{(7\text{ ft})} + 2\right) \times \left(\frac{(4.8303\text{ ft})}{(7\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 20.092\text{ kip}$$

$$V_{max} = 20.092 \text{ kip}$$

M_{max} - Max bending moment located at depth a/2,

$$M_{max} = (H_o \, D \, L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 \, L_e} \right) - \left[\left(\frac{4 \, E}{L_e} + 3 \right) \left(\frac{a}{2 \, L_e} \right)^3 \right] + \left[\left(\frac{3 \, E}{L_e} + 2 \right) \left(\frac{a}{2 \, L_e} \right)^4 \right] \right]$$

$$M_{max} = ((-1.0906 \text{ kip/ft}) \times (60 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(11.967 \text{ ft})}{(7 \text{ ft})} + \frac{(4.8303 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.967 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.8303 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.967 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.8303 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 66.857 \text{ kipft}$$

Shear force and Bending moment (z-direction, LRFD)

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57 \, b}$$

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z \, H)}{1.57 \, b}$$

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.00012739 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

$$a = \frac{(4 \, M_o \, L_e) + \left(3 \, H_o \, L_e^2\right)}{(6 \, M_o) + (4 \, H_o \, L_e)}$$

$$a = \frac{(4 \times (0.00012739 \text{ kipft/ft}) \times (7 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (7 \text{ ft})^2\right)}{(6 \times (0.00012739 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.6667 \text{ ft}$$

V_{max} - Max shear force located at depth a,

$$V_{max} = 12 \left(\frac{M_o \, b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00012739 \text{ kipft/ft}) \times (60 \text{ in})}{(7 \text{ ft})} \right) \times \left(\frac{(4.6667 \text{ ft})}{(7 \text{ ft})} - 1 \right) \times \left(\frac{(4.6667 \text{ ft})}{(7 \text{ ft})} \right)^2$$

$$V_{max} = 0.00016176 \text{ kip}$$

M_{max} - Max bending moment at depth a/2,

$$M_{max} = (M_o \, b) \left[1 - \left(4 \, \frac{a}{2 \, L_e}^3 \right) + \left(3 \, \frac{a}{2 \, L_e}^4 \right) \right]$$

$$M_{max} = ((0.00012739 \text{ kipft/ft}) \times (60 \text{ in})) \times \left[1 - \left(4 \times \frac{(4.6667 \text{ ft})}{2 \times (7 \text{ ft})}^3 \right) + \left(3 \times \frac{(4.6667 \text{ ft})}{2 \times (7 \text{ ft})}^4 \right) \right]$$

$$M_{max} = 0.00056617 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1 $A_{st,required}$

$$A_{st,required} = Min\left[\frac{\frac{P}{\phi\,\alpha} - (0.85\,f'_{ck}\,A_g)}{f_{yk} - (0.85\,f'_{ck})}, (0.08\,A_g)\right]$$

$$A_{st,required} = Min\left[\frac{\frac{(18.123\text{ kip})}{(0.65)\times(0.8)} - \left(0.85\times(2.5\text{ ksi})\times\left(3600\text{ in}^2\right)\right)}{(60\text{ ksi}) - (0.85\times(2.5\text{ ksi}))}, \left(0.08\times\left(3600\text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -131.58\text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

$$A_{min} = Max\left[A_{st,required}, (0.0018\,A_g)\right]$$

$$A_{min} = Max\left[\left(-131.58\text{ in}^2\right), \left(0.0018\times\left(3600\text{ in}^2\right)\right)\right]$$

$$A_{min} = 6.48\text{ in}^2$$

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(6.48\text{ in}^2\right)}{\left(0.3068\text{ in}^2\right)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar}\,\frac{\pi\,d_{bar}^2}{4}$$

$$A_{st} = (22)\times\frac{\pi\times(0.625\text{ in})^2}{4}$$

$$A_{st} = 6.7495\text{ in}^2$$

$Ratio$ - Capacity

$$Ratio = \frac{A_{min}}{A_{st}}$$

$$Ratio = \frac{\left(6.48\text{ in}^2\right)}{\left(6.7495\text{ in}^2\right)}$$

$$Ratio = 0.96007$$

Status: **PASS**
Ratio: **0.960**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

$$s_{rebar} = Max\left[1.5, (1.5\,d_{bar})\right]$$

$$s_{rebar} = Max\left[1.5, (1.5\times(0.625\text{ in}))\right]$$

$$s_{rebar} = 1.5\text{ in}$$

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

$$s_{ties} = Min\left[(16\,d_{bar}), (48\,d_{ties}), Min\,(D, b)\right]$$

$$s_{ties} = Min\left[(16\times(0.625\text{ in})), (48\times(0.375\text{ in})), Min\,((60\text{ in}), (60\text{ in}))\right]$$

$$s_{ties} = 10\text{ in}$$

Summary:

Main reinforcement: **22 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

$$\phi P_N = \phi\,0.80\,\left[\left(0.85\,f'_{ck}\,[A_g - A_{st}]\right) + (f_{yk}\,A_{st})\right]$$

$$\phi P_N = (0.65)\times 0.80\times\left[\left(0.85\times(2.5\text{ ksi})\times\left[\left(3600\text{ in}^2\right) - \left(6.7495\text{ in}^2\right)\right]\right) + \left((60\text{ ksi})\times\left(6.7495\text{ in}^2\right)\right)\right]$$

$$\phi P_N = 4181.1\text{ kip}$$

$Ratio$ - Capacity

$$Ratio = \frac{P}{\phi P_N}$$

$$Ratio = \frac{(18.123 \text{ kip})}{(4181.1 \text{ kip})}$$

$$Ratio = 0.0043345$$

Status: **PASS**
Ratio: **0.000**

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 60$ in - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

$$d = 0.80 \times (60 \text{ in})$$

$$d = 48 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(48 \text{ in})}{10}}}, 1\right]$$

$$\lambda_s = 0.58722$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

$$V_{c,max} = 5\ \lambda_s \sqrt{f'_{ck}}\ b_w\ d$$

$$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,max} = 422.8 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}, P = 18.123 \text{ kip} \rightarrow 18123 \text{ lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

$$V_{c,a} = \left[2\ \lambda_s \sqrt{f'_{ck}} + \frac{P}{6\ A_g}\right]\ b_w\ d$$

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + \frac{(18123 \text{ lbf})}{6 \times (3600 \text{ in}^2)}\right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.54 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

$$V_{c,b} = \left[2\ \lambda_s \sqrt{f'_{ck}} + (0.05\ f'_{ck})\right]\ b_w\ d$$

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi}))\right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

$$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$

$$V_c = Min\left[(422.8 \text{ kip}), (171.54 \text{ kip}), (529.12 \text{ kip})\right]$$

$$V_c = 171.54 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

$$V_{s,a} = 8 \sqrt{f'_{ck}}\ b_w\ d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152\ \text{kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi\, d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375\ \text{in})^2}{4}$$

$$A_v = 0.11045\ \text{in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times \left(0.11045\ \text{in}^2\right) \times (60\ \text{ksi}) \times (48\ \text{in})}{(10\ \text{in})}$$

$$V_{s,b} = 63.617\ \text{kip}$$

V_s - Governing shear strength of steel

$$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$

$$V_s = MIN\left[(1152\ \text{kip}), (63.617\ \text{kip})\right]$$

$$V_s = 63.617\ \text{kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\ (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((171.54\ \text{kip}) + (63.617\ \text{kip}))$$

$$\phi V_n = 152.85\ \text{kip}$$

Considering x-direction:

$V_{max} = 20.092\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(20.092\ \text{kip})}{(152.85\ \text{kip})}$$

$$Ratio = 0.13145$$

Status: **PASS**
Ratio: **0.130**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b\, D^2}{6}$$

$$S_m = \frac{(60\ \text{in}) \times (60\ \text{in})^2}{6}$$

$$S_m = 36000\ \text{in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 ksi)} \times 35999.999 in^3$$

$$\phi M_{n,1} = 487.500 kipft$$

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi\ 0.85\ f_{ck}'\ S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \text{ksi}) \times \left(36000\ \text{in}^3\right)$$

$$\phi M_{n,2} = 4143.7\ \text{kipft}$$

	Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$ $$\phi M_n = 487.5 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 66.857 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(66.857 \text{ kipft})}{(487.5 \text{ kipft})}$$ $$Ratio = 0.13714$$	Status: **PASS** Ratio: **0.140**
	Considering z-direction: $M_{max} = 0.00056617 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.00056617 \text{ kipft})}{(487.5 \text{ kipft})}$$ $$Ratio = 1.1614 \times 10^{-6}$$	Status: **PASS** Ratio: **0.000**