

Project Name: MTSOLAR_ICHI76E1E623 4x5 - V1Jb

Date: Mon May 19 2025

Location: 349 Clancy St, Helena, MT 59601, USA

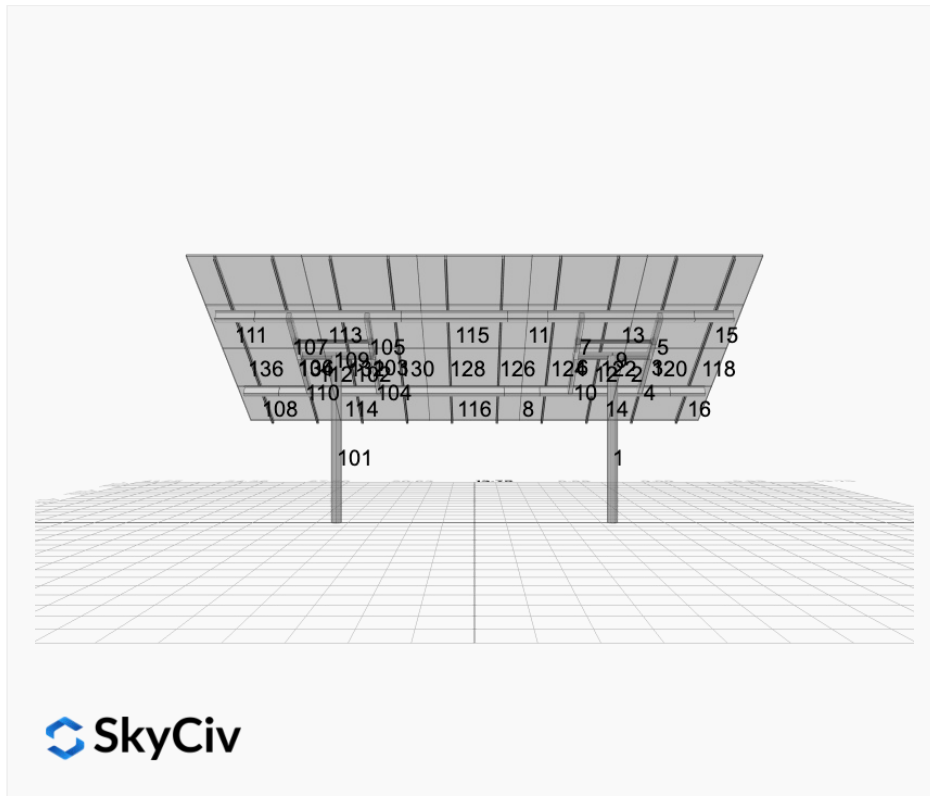
Number of Modules: 20

Unique ID: 2P-15-6TOP-SD-24-L-4Hx5W-C9E0

Number of Poles: 2

Dealer: _____

Date Sold: _____



Array Dimensions N/S	16.17 ft
Array Dimensions E/W	27.50 ft
Winter Tilt Angle	30
Front Edge Clearance	5 ft

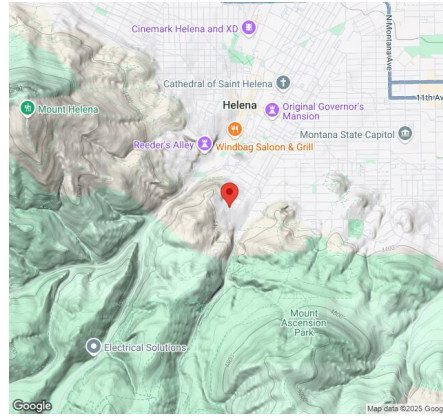
MT Solar Bill of Materials (2P-15-6TOP-SD-24-L-4Hx5W-C9E0)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	2
MTS-HF-SD	H-Frame Assembly-SD	2
MTS-SD-Wing-24	24IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	2
MTS-CLAMP-HOOK-4PK	Hook Clamp	5

Rail Bill of Materials

Part	Qty
Rails (194in)	10
Rail Attachment	20
Module Mid Clamp	30
Module End Clamp	20
Ground Lug	5

Site Details:



Site Address: 349 Clancy St, Helena, MT 59601, USA

Array Specification

Duty Classification:	SD
Module Width:	48.00 in
Module Length:	65.00in
Number of Rows:	4
Number of Columns:	5
Total Number of Modules:	20
Winter Tilt Angle:	30
Front Edge Clearance:	5
Total Array Height at Tilt:	13.08 ft
Total Frame Length:	26.50 ft
Module Info/Notes:	Hyperion 405w
Array Dimensions N/S:	16.17 ft
Array Dimensions E/W:	27.50 ft
Rail Length:	194.00 in
Rail Spacing:	2.75 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.04 ft
Number of Poles:	2
Pole Spacing:	15 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 7.50 ft Pile 2: 7.50 ft
Foundation Volume:	3.927 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	349 Clancy St, Helena, MT 59601, USA
Wind Speed:	90 mph
Snow Load:	30 psf

Design Disclaimer

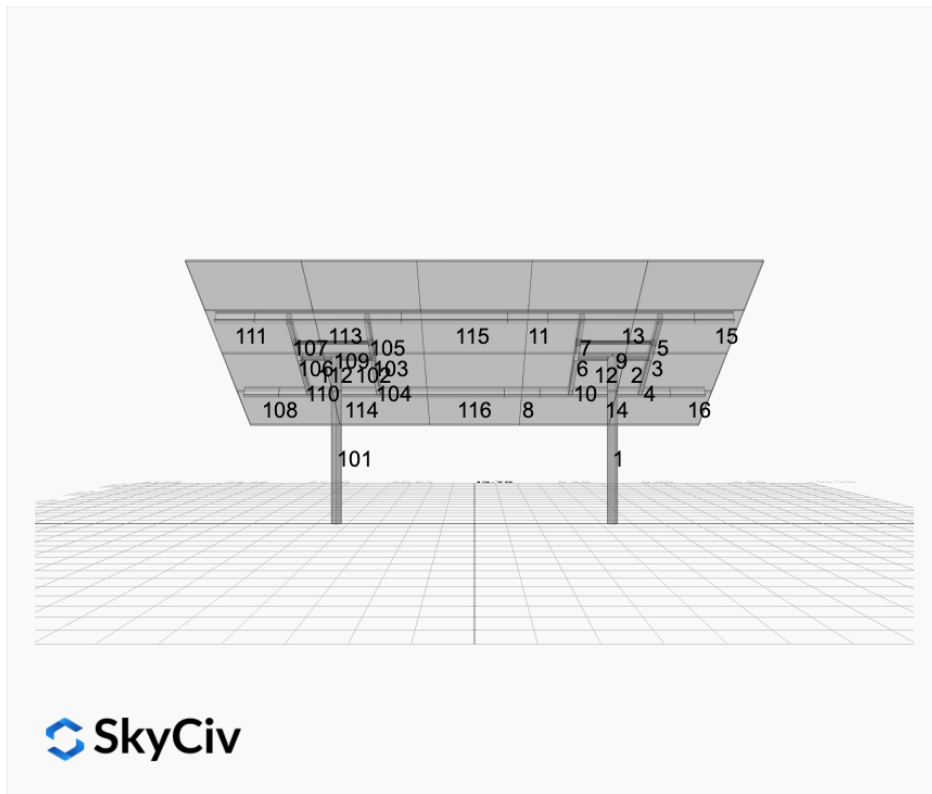
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

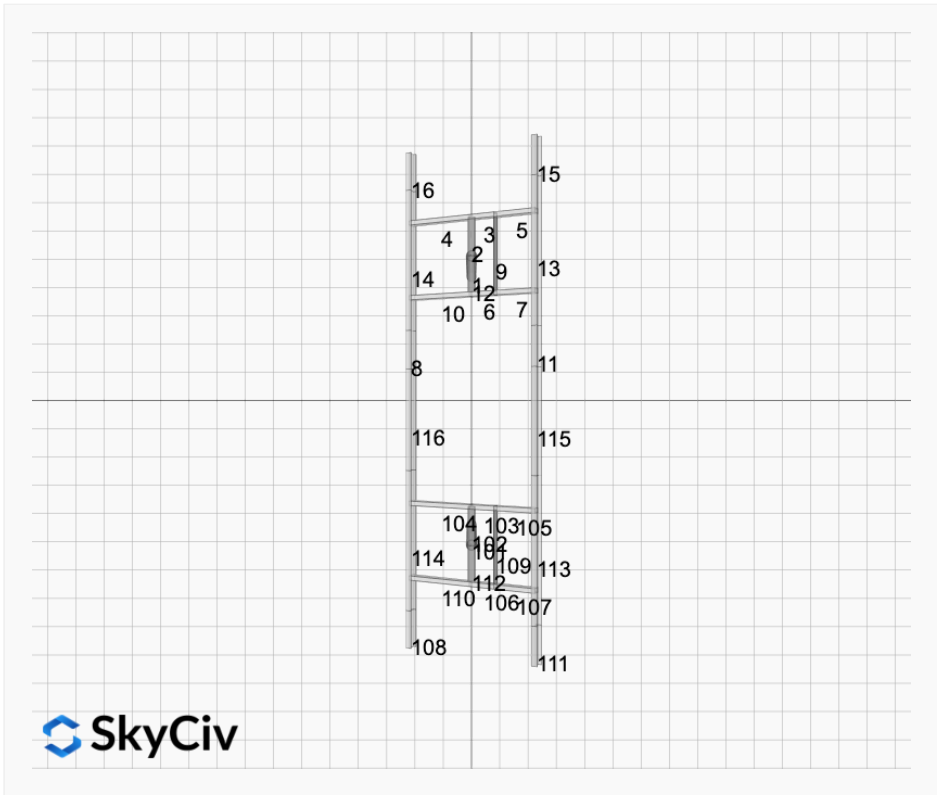
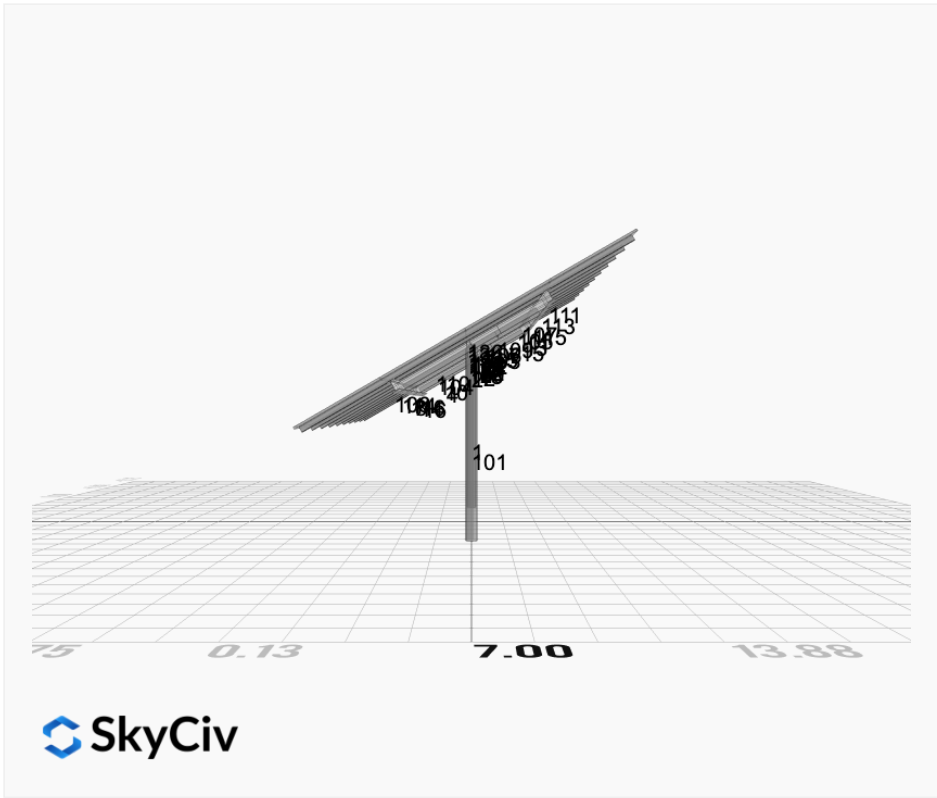
AutoDesigner Input

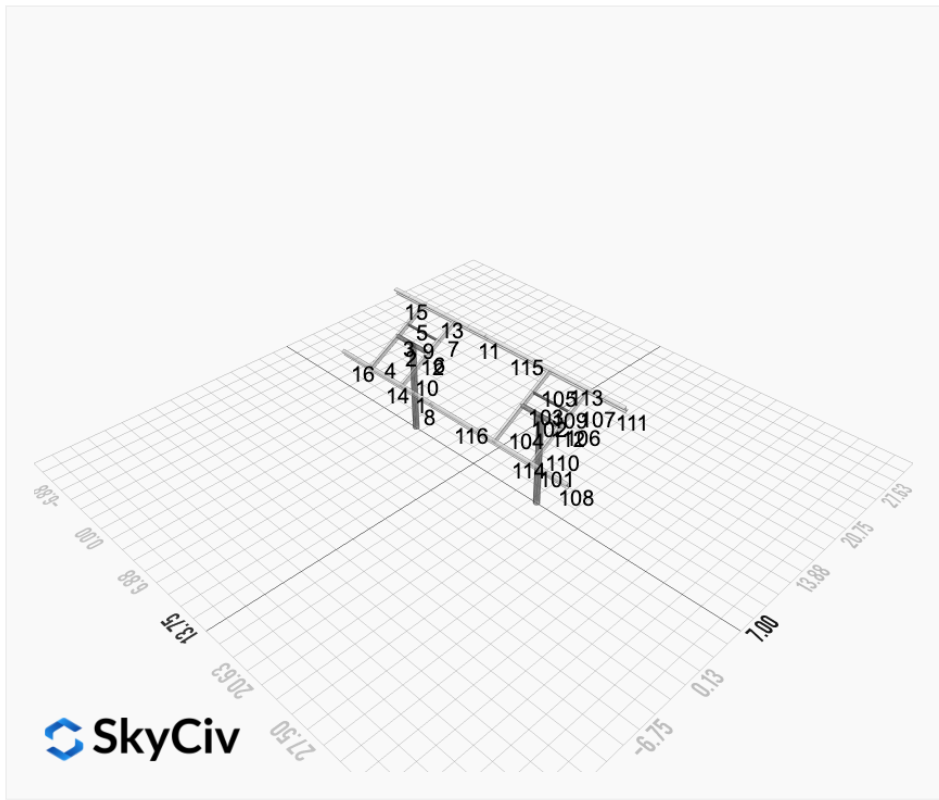
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Design Notes:

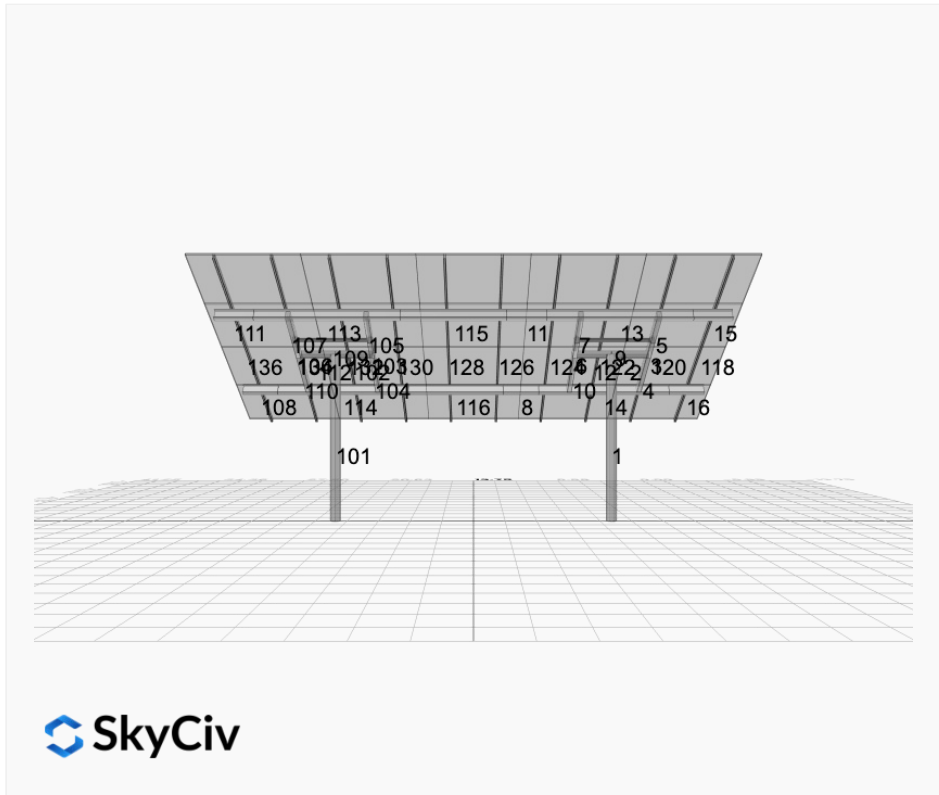
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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





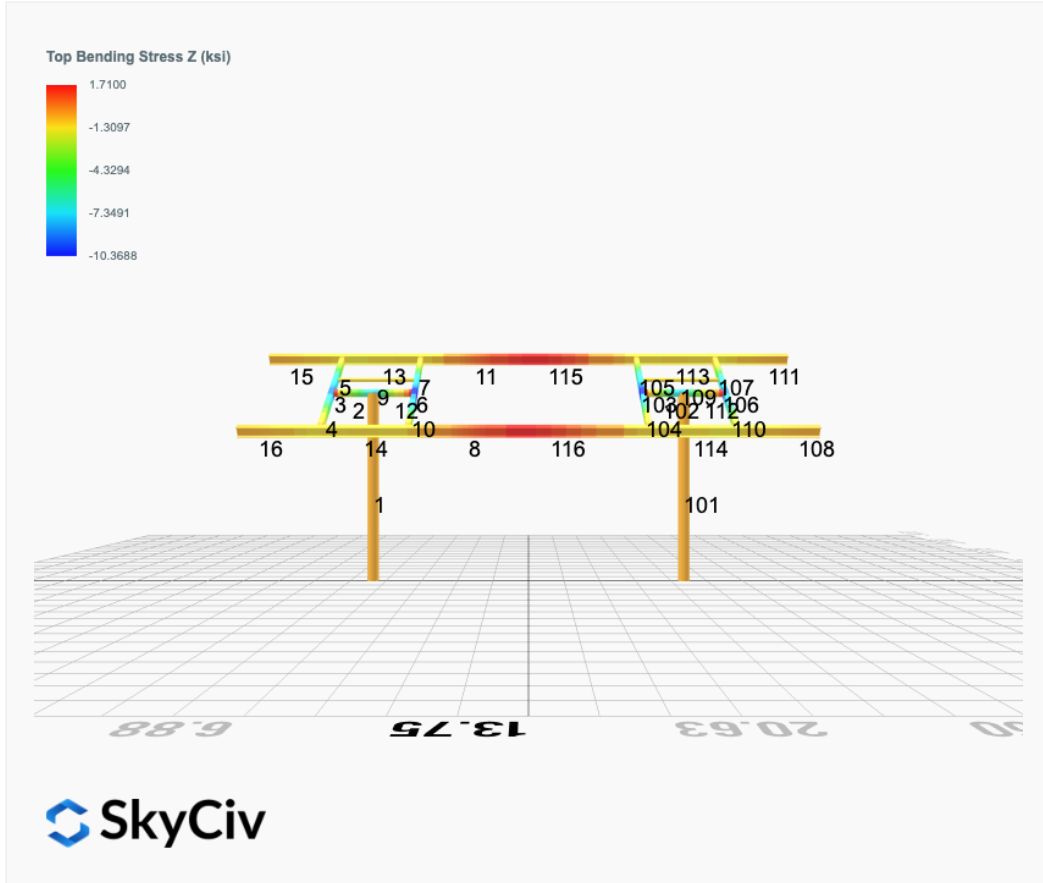
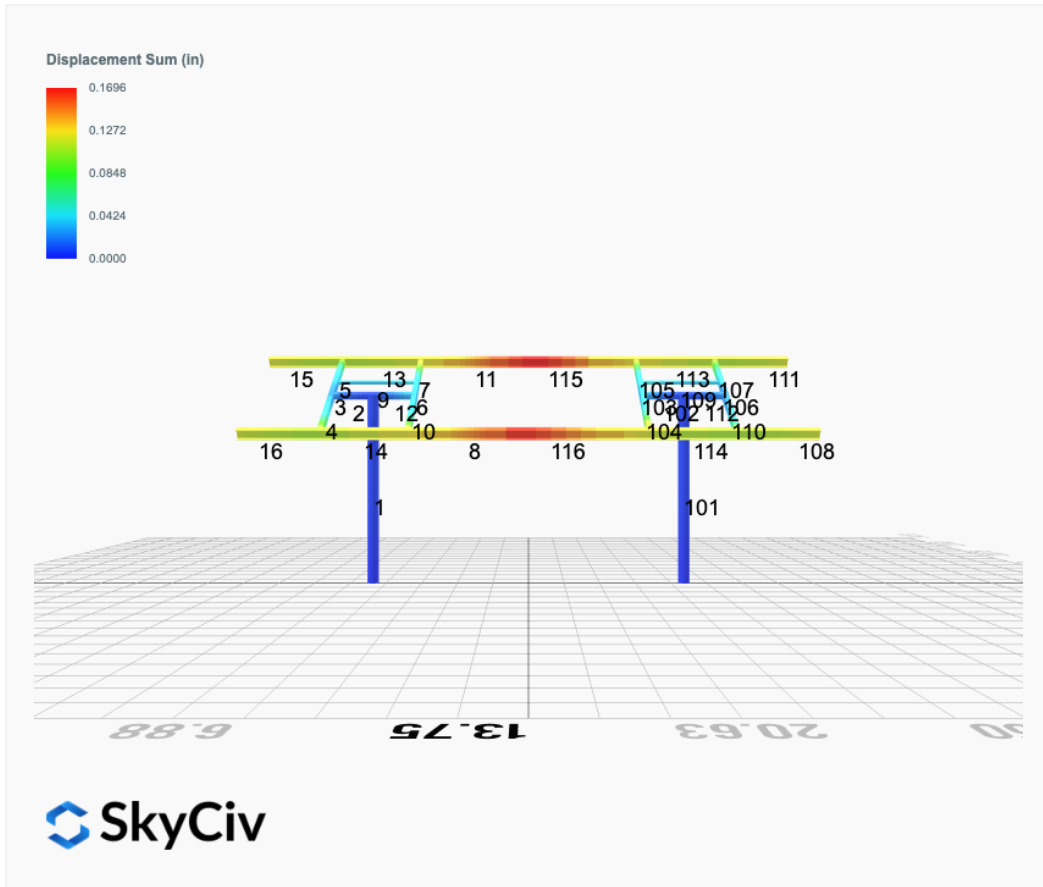


 SkyCiv

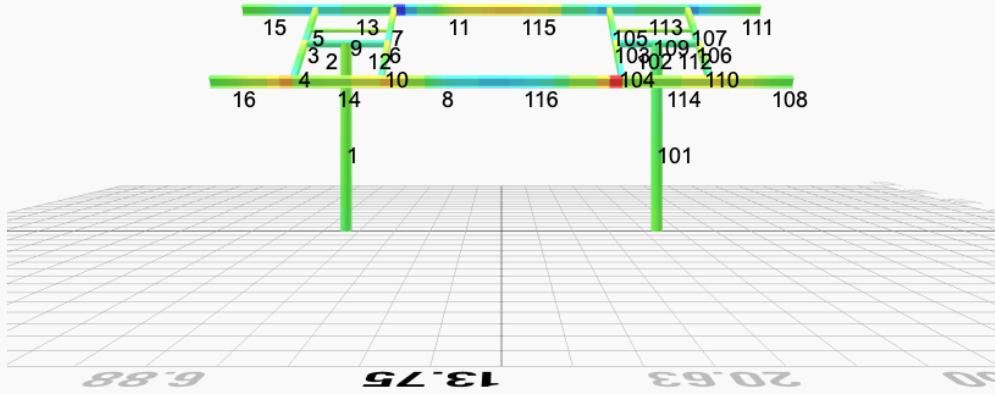
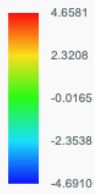


 SkyCiv

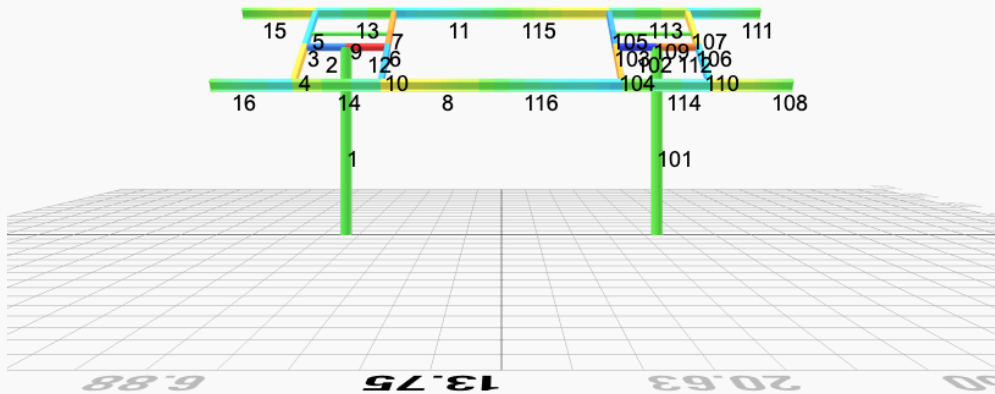
FEM Results (Envelope Worst Case for each member)



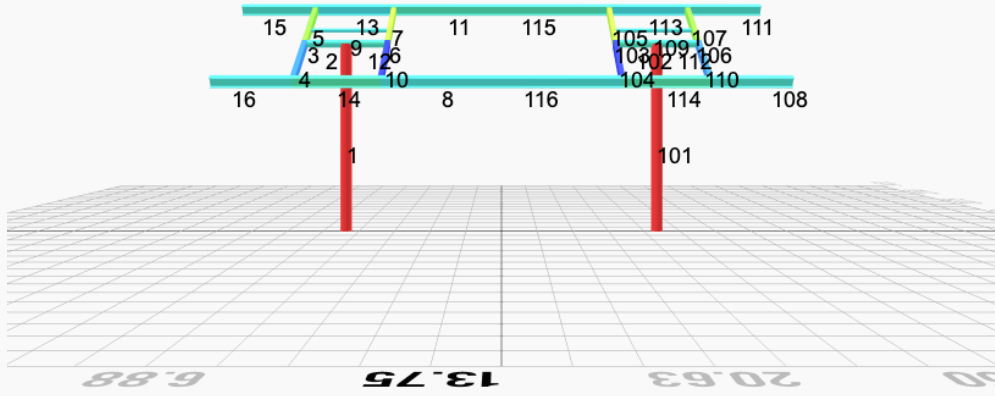
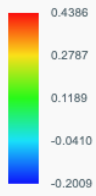
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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	1.6007	0.0245	0.0663	-0.0072	0.0199
ULS: 2. D + L	0.0000	1.6007	0.0245	0.0663	-0.0072	0.0199
ULS: 3. D + (S or Lr or R)	0.0000	4.0486	0.0718	0.1941	-0.0216	0.0295
ULS: 3. D + (S or Lr or R)	0.0000	1.6007	0.0245	0.0663	-0.0072	0.0199
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.4367	0.0600	0.1621	-0.0180	0.0271
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6007	0.0245	0.0663	-0.0072	0.0199
ULS: 5b. D + 0.7E	0.0000	1.6007	0.0245	0.0663	-0.0072	0.0199
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.4367	0.0600	0.1621	-0.0180	0.0271
ULS: 8. 0.6D + 0.7E	0.0000	0.9604	0.0147	0.0398	-0.0043	0.0119
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5241	4.2406	0.0809	0.2128	-0.0954	14.0998
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5241	4.2406	0.0809	0.2128	-0.0954	14.0998
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3064	-0.6620	-0.0236	-0.0584	0.0682	-11.5826
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0887	-0.2849	-0.0160	-0.0387	0.0563	-14.9998
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1431	5.4166	0.1023	0.2721	-0.0841	10.5870
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1431	5.4166	0.1023	0.2721	-0.0841	10.5870
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9798	1.7396	0.0239	0.0686	0.0385	-8.6748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8165	2.0224	0.0296	0.0834	0.0296	-11.2376
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1431	3.5806	0.0668	0.1762	-0.0733	10.5798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1431	3.5806	0.0668	0.1762	-0.0733	10.5798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9798	-0.0964	-0.0116	-0.0272	0.0493	-8.6820
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8165	0.1865	-0.0059	-0.0125	0.0404	-11.2449
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5241	3.6003	0.0711	0.1863	-0.0925	14.0919
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.5241	3.6003	0.0711	0.1863	-0.0925	14.0919
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3064	-1.3023	-0.0334	-0.0849	0.0711	-11.5906
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0887	-0.9252	-0.0258	-0.0652	0.0591	-15.0077

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	8.0374
Shear X	-2.5402
Shear Z	0.1523
Moment X	0.4077
Moment Y (Twist)	0.1626
Moment Z	25.4633

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	5.4166
Shear X	-1.5241
Shear Z	0.1023
Moment X	0.2721
Moment Y (Twist)	0.0954
Moment Z	15.0077

Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.6007	-0.0245	-0.0663	0.0072	0.0199
ULS: 2. D + L	-0.0000	1.6007	-0.0245	-0.0663	0.0072	0.0199
ULS: 3. D + (S or Lr or R)	-0.0000	4.0486	-0.0718	-0.1941	0.0216	0.0295
ULS: 3. D + (S or Lr or R)	-0.0000	1.6007	-0.0245	-0.0663	0.0072	0.0199
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.4367	-0.0600	-0.1621	0.0180	0.0271

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.6007	-0.0245	-0.0663	0.0072	0.0199
ULS: 5b. D + 0.7E	-0.0000	1.6007	-0.0245	-0.0663	0.0072	0.0199
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.4367	-0.0600	-0.1621	0.0180	0.0271
ULS: 8. 0.6D + 0.7E	-0.0000	0.9604	-0.0147	-0.0398	0.0043	0.0119
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5241	4.2406	-0.0809	-0.2128	0.0954	14.0998
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5241	4.2406	-0.0809	-0.2128	0.0954	14.0998
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3064	-0.6620	0.0236	0.0584	-0.0682	-11.5826
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0887	-0.2849	0.0160	0.0387	-0.0563	-14.9998
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1431	5.4166	-0.1023	-0.2721	0.0841	10.5870
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1431	5.4166	-0.1023	-0.2721	0.0841	10.5870
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9798	1.7396	-0.0239	-0.0686	-0.0385	-8.6748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8165	2.0224	-0.0296	-0.0834	-0.0296	-11.2376
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1431	3.5806	-0.0668	-0.1762	0.0733	10.5798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1431	3.5806	-0.0668	-0.1762	0.0733	10.5798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9798	-0.0964	0.0116	0.0273	-0.0493	-8.6820
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8165	0.1865	0.0059	0.0125	-0.0404	-11.2449
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5241	3.6003	-0.0711	-0.1863	0.0925	14.0919
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.5241	3.6003	-0.0711	-0.1863	0.0925	14.0919
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3064	-1.3023	0.0334	0.0849	-0.0711	-11.5906
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0887	-0.9252	0.0258	0.0652	-0.0591	-15.0077

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.0375
Shear X	-2.5402
Shear Z	-0.1523
Moment X	-0.4076
Moment Y (Twist)	0.1625
Moment Z	25.4639

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	5.4166
Shear X	-1.5241
Shear Z	-0.1023
Moment X	-0.2721
Moment Y (Twist)	0.0954
Moment Z	15.0077

Project Details

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

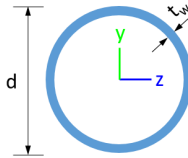


Design Input Information

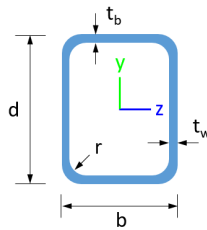
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
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Design Materials			
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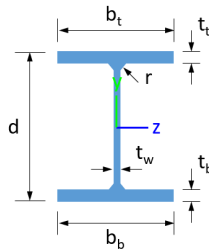
Section Dimensions



ID	Name	d (in)	t_w (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

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)
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5	0.008	0.382	0.065	0.062	0.011	0.385	#13	0.073	Not Required	Pass
6	0.010	0.675	0.110	0.068	0.026	0.773	#21	0.044	Not Required	Pass
7	0.010	0.418	0.113	0.067	0.022	0.435	#21	0.073	Not Required	Pass
8	0.001	0.104	0.069	0.035	0.008	0.169	#21	0.088	Not Required	Pass
9	0.002	0.056	0.054	0.001	0.002	0.103	#21	0.132	Not Required	Pass
10	0.010	0.672	0.105	0.068	0.020	0.739	#21	0.078	Not Required	Pass
11	0.002	0.103	0.068	0.035	0.008	0.168	#21	0.088	Not Required	Pass
12	0.001	0.435	0.164	0.097	0.030	0.582	#13	0.052	Not Required	Pass
13	0.002	0.109	0.151	0.052	0.012	0.218	#21	0.265	Not Required	Pass
14	0.003	0.112	0.150	0.052	0.012	0.215	#21	0.177	Not Required	Pass
15	0.000	0.024	0.032	0.019	0.004	0.055	#21	Not Required	Not Required	Pass
16	0.000	0.024	0.032	0.019	0.004	0.055	#21	Not Required	Not Required	Pass
101	0.068	0.602	0.023	0.034	0.002	0.606	#13	0.507	Not Required	Pass
102	0.001	0.435	0.164	0.097	0.030	0.582	#13	0.052	Not Required	Pass
103	0.010	0.675	0.110	0.068	0.026	0.773	#21	0.044	Not Required	Pass
104	0.010	0.672	0.105	0.068	0.020	0.739	#21	0.078	Not Required	Pass
105	0.010	0.418	0.113	0.067	0.022	0.435	#21	0.073	Not Required	Pass
106	0.009	0.617	0.068	0.062	0.016	0.668	#21	0.044	Not Required	Pass
107	0.008	0.382	0.065	0.062	0.011	0.385	#13	0.073	Not Required	Pass
108	0.000	0.024	0.032	0.019	0.004	0.055	#21	Not Required	Not Required	Pass
109	0.002	0.056	0.054	0.001	0.002	0.103	#21	0.132	Not Required	Pass
110	0.007	0.614	0.080	0.062	0.016	0.677	#21	0.078	Not Required	Pass
111	0.000	0.024	0.032	0.019	0.004	0.055	#21	Not Required	Not Required	Pass
112	0.000	0.375	0.154	0.086	0.030	0.514	#13	0.167	Not Required	Pass
113	0.002	0.109	0.151	0.052	0.012	0.218	#21	0.177	Not Required	Pass
114	0.003	0.112	0.150	0.052	0.012	0.215	#21	0.265	Not Required	Pass
115	0.002	0.126	0.090	0.035	0.008	0.213	#21	0.235	Not Required	Pass
116	0.001	0.127	0.091	0.035	0.008	0.214	#21	0.235	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/compression)
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
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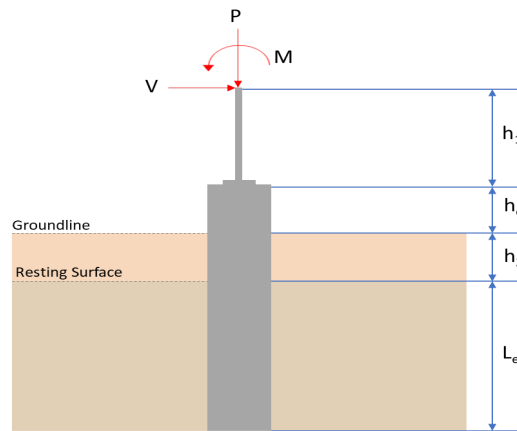
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 7.5$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ 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	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.417	8.037
V_x (kip)	-1.524	-2.540
V_z (kip)	0.102	0.152
M_x (kipft)	0.272	0.408
M_z (kipft)	15.008	25.463

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.524 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.508 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.008 \text{ kipft}) + ((-1.524 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.0027 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.7385 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.102 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.034 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z H)}{D}$$

$$M_o = \frac{(0.272 \text{ kipft}) + ((0.102 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.090667 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.7194 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.7385 \text{ ft}), (2.7194 \text{ ft})]$$

$$L_{e,req} = 6.739 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(6.739 \text{ ft})}{(7.5 \text{ ft})}$$

$$Ratio = 0.89853$$

Status: **PASS**
Ratio: **0.900**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(5.417 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.76635 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.76635 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.38317$$

Status: **PASS**
Ratio: **0.380**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.508 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.0027 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0027 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (5.0027 \text{ kipft/ft})) + (4 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.2105 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (5.0027 \text{ kipft/ft})) + (3 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (5.0027 \text{ kipft/ft})) + (2 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.20871 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (5.0027 \text{ kipft/ft})) + ((-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0381 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.2105 \text{ ft})}{2}$$

$$p_a = 0.39079 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20871 \text{ kip/ft}^2)}{(0.39079 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.53407$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0381 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92272$$

Status: **PASS**
Ratio: **0.530**

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = 0.034 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.090667 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.090667 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.034 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.090667 \text{ kipft/ft})) + (4 \times (0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.4076 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.090667 \text{ kipft/ft})) + (3 \times (0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.090667 \text{ kipft/ft})) + (2 \times (0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.034055 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.090667 \text{ kipft/ft})) + ((0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.07311 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.4076 \text{ ft})}{2}$$

$$p_a = 0.40557 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.034055 \text{ kip/ft}^2)}{(0.40557 \text{ kip/ft}^2)}$$

$$(0.07311 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.083967$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

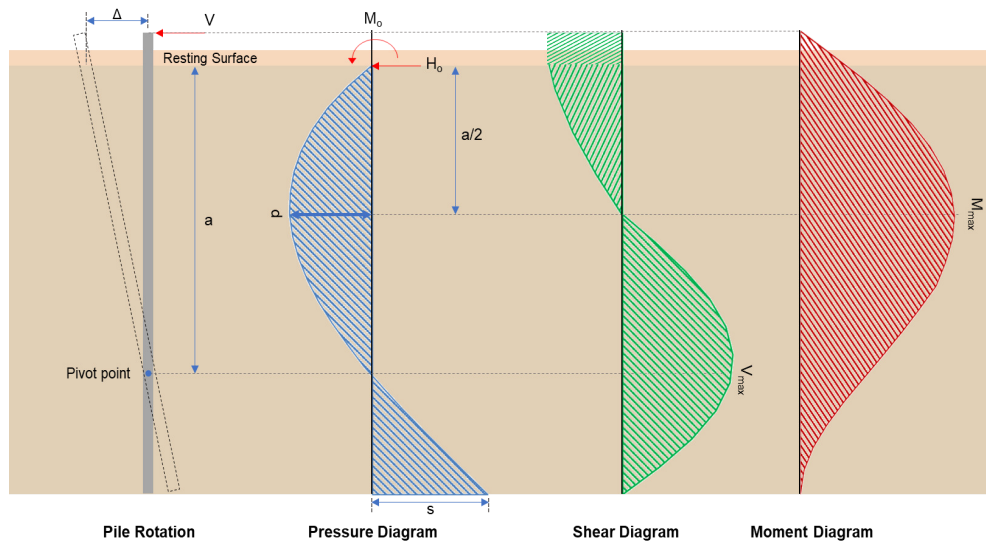
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.07311 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.064987$$

Status: **PASS**
Ratio: **0.080**

Status: **PASS**
Ratio: **0.060**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.54 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.84667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.463 \text{ kipft}) + ((-2.54 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.4877 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.4877 \text{ kipft/ft})}{(-0.84667 \text{ kip/ft})}$$

$$E = 10.025 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.4877 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.84667 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (8.4877 \text{ kipft/ft})) + (4 \times (-0.84667 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.208 \text{ ft}$$

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

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

$$V_{max} = ((-0.84667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.208 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.208 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.6825 \text{ kip}$$

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

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

$$M_{max} = ((-0.84667 \text{ kip/ft}) \times (36 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(10.025 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.208 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.208 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.208 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.086 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.152 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.050667 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z H)}{D}$$

$$M_o = \frac{(0.408 \text{ kipft}) + ((0.152 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.136 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.136 \text{ kipft/ft})}{(0.050667 \text{ kip/ft})}$$

$$E = 2.6842 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.136 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.050667 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.136 \text{ kipft/ft})) + (4 \times (0.050667 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.4067 \text{ ft}$$

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

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

$$[\setminus L_e \quad / \setminus L_e /]]$$

$$V_{max} = ((0.050667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.4067 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.4067 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.19806 \text{ kip}$$

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

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

$$M_{max} = ((0.050667 \text{ kip/ft}) \times (36 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(2.6842 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.4067 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.4067 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.4067 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.64147 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.037 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.122 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.122 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</p> <p>$s_{rebar} = Max [1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.037 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0064095$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.037 \text{ kip} \rightarrow 8037 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8037 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.802 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.802 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.802 \text{ kip}$$

22.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{s,a}$ - Shear strength of steel (a)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.802 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.082 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.6825 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.6825 \text{ kip})}{(74.082 \text{ kip})}$$

$$Ratio = 0.1037$$

Status: **PASS**
Ratio: **0.100**

Considering z-direction:

$V_{max} = 0.19806 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.19806 \text{ kip})}{(74.082 \text{ kip})}$$

$$Ratio = 0.0026735$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

$$\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

$$\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 27.086 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$Ratio = \frac{(27.086 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.43668$$

Status: **PASS**
Ratio: **0.440**

Considering z-direction:

$M_{max} = 0.64147 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.64147 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.010342$$

Status: **PASS**
Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS
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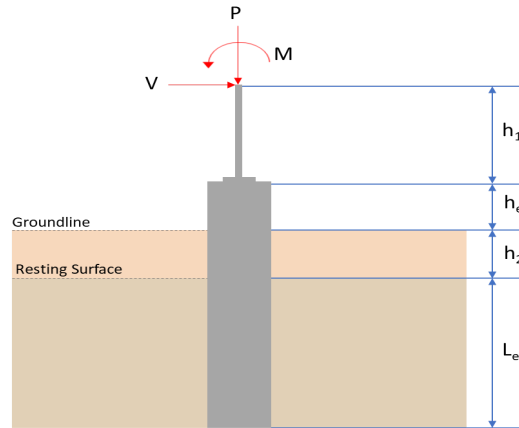
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 7.5$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ 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	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.417	8.037
V_x (kip)	-1.524	-2.540
V_z (kip)	-0.102	-0.152
M_x (kipft)	-0.272	-0.408
M_z (kipft)	15.008	25.464

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.524 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.508 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.008 \text{ kipft}) + ((-1.524 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.0027 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.7385 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.102 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.034 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z H)}{D}$$

$$M_o = \frac{(0.272 \text{ kipft}) + ((-0.102 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.090667 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.7839 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.7385 \text{ ft}), (1.7839 \text{ ft})]$$

$$L_{e,req} = 6.739 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(6.739 \text{ ft})}{(7.5 \text{ ft})}$$

$$Ratio = 0.89853$$

Status: **PASS**
Ratio: **0.900**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(5.417 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.76635 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.76635 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.38317$$

Status: **PASS**
Ratio: **0.380**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.508 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.0027 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0027 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (5.0027 \text{ kipft/ft})) + (4 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.2105 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (5.0027 \text{ kipft/ft})) + (3 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (5.0027 \text{ kipft/ft})) + (2 \times (-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.20871 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (5.0027 \text{ kipft/ft})) + ((-0.508 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0381 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.2105 \text{ ft})}{2}$$

$$p_a = 0.39079 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20871 \text{ kip/ft}^2)}{(0.39079 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.53407$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0381 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92272$$

Status: **PASS**
Ratio: **0.530**

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = -0.034 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.090667 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.090667 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.034 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.090667 \text{ kipft/ft})) + (4 \times (-0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.4076 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.090667 \text{ kipft/ft})) + (3 \times (-0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.090667 \text{ kipft/ft})) + (2 \times (-0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = -0.014244 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.090667 \text{ kipft/ft})) + ((-0.034 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = -0.012343 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.4076 \text{ ft})}{2}$$

$$p_a = 0.40557 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.014244 \text{ kip/ft}^2)}{(0.40557 \text{ kip/ft}^2)}$$

$$(0.0001 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.03512$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

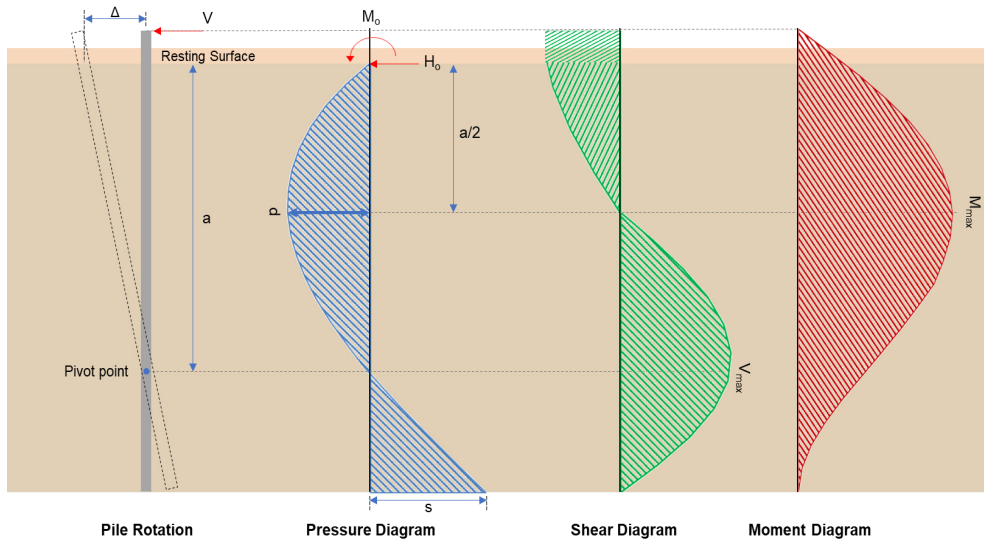
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.012343 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.010972$$

Status: **PASS**
Ratio: **-0.040**

Status: **PASS**
Ratio: **-0.010**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.54 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.84667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.464 \text{ kipft}) + ((-2.54 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.488 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.488 \text{ kipft/ft})}{(-0.84667 \text{ kip/ft})}$$

$$E = 10.025 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.488 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.84667 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (8.488 \text{ kipft/ft})) + (4 \times (-0.84667 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.208 \text{ ft}$$

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

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

$$V_{max} = ((-0.84667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.208 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.208 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.6828 \text{ kip}$$

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

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

$$M_{max} = ((-0.84667 \text{ kip/ft}) \times (36 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(10.025 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.208 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.208 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.025 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.208 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.087 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.152 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.050667 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z H)}{D}$$

$$M_o = \frac{(0.408 \text{ kipft}) + ((-0.152 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.136 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.136 \text{ kipft/ft})}{(-0.050667 \text{ kip/ft})}$$

$$E = 2.6842 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.136 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.050667 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.136 \text{ kipft/ft})) + (4 \times (-0.050667 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.4067 \text{ ft}$$

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

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

$$[\setminus L_e / \setminus L_e /]]$$

$$V_{max} = ((-0.050667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.4067 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.4067 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.19806 \text{ kip}$$

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

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

$$M_{max} = ((-0.050667 \text{ kip/ft}) \times (36 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(2.6842 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.4067 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.4067 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6842 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.4067 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.64147 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.037 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.122 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.122 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</p> <p>$s_{rebar} = Max [1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.037 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0064095$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.037 \text{ kip} \rightarrow 8037 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8037 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.802 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.802 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.802 \text{ kip}$$

22.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{s,a}$ - Shear strength of steel (a)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.802 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.082 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.6828 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.6828 \text{ kip})}{(74.082 \text{ kip})}$$

$$Ratio = 0.10371$$

Status: **PASS**
Ratio: **0.100**

Considering z-direction:

$V_{max} = 0.19806 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.19806 \text{ kip})}{(74.082 \text{ kip})}$$

$$Ratio = 0.0026735$$

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

Flexural Strength (ACI 318-19, LFRD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

$$\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

$$\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 27.087 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$Ratio = \frac{(27.087 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.4367$$

Status: **PASS**
Ratio: **0.440**

Considering z-direction:

$M_{max} = 0.64147 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.64147 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.010342$$

Status: **PASS**
Ratio: **0.010**