

Project Details



Project Name: Libby Residence - V1Jb

Date: Fri Aug 09 2024

Location: 7641 Snowmass Creek Rd, Snowmass, CO
81654, USA

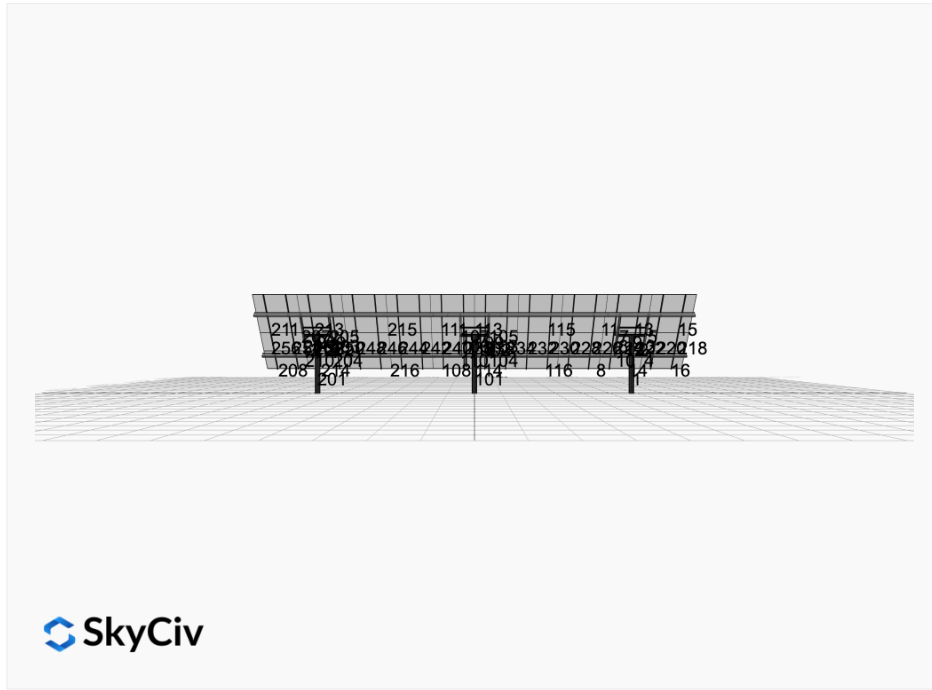
Number of Modules: 40

Number of Poles: 3

Unique ID: 3P-22.5-8TOP-HD-57-L-4Hx10W-BA5J

Date Sold:

Dealer: _____



Array Dimensions N/S	13.87 ft
Array Dimensions E/W	62.50 ft
Winter Tilt Angle	50
Front Edge Clearance	3 ft

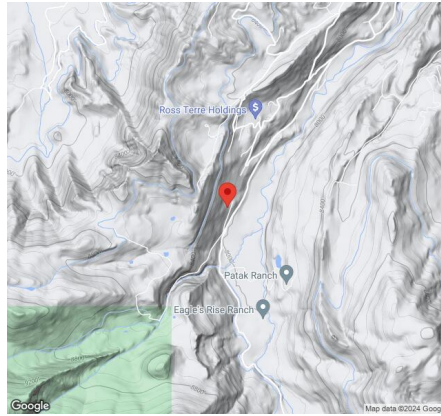
MT Solar Bill of Materials (3P-22.5-8TOP-HD-57-L-4Hx10W-BA5J)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-57	57IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	10

Rail Bill of Materials

Part	Qty
Rails (164in)	20
Rail Attachment	40
Module Mid Clamp	60
Module End Clamp	40
Ground Lug	10

Site Details:



Site Address: 7641 Snowmass Creek Rd, Snowmass, CO 81654, USA

Array Specification

Duty Classification:	HD
Module Width:	41.10 in
Module Length:	74.00in
Number of Rows:	4
Number of Columns:	10
Total Number of Modules:	40
Winter Tilt Angle:	50
Front Edge Clearance:	3
Total Array Height at Tilt:	13.56 ft
Total Frame Length:	62.00 ft
Frame Weight:	3369 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	62.50 ft
Rail Length:	166.40 in
Rail Spacing:	3.08 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.31 ft
Number of Poles:	3
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 6.00 ft Pile 3: 5.75 ft
Foundation Volume:	10.370 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	7641 Snowmass Creek Rd, Snowmass, CO 81654, USA
Wind Speed:	115 mph

Snow Load:

90 psf

Design Disclaimer

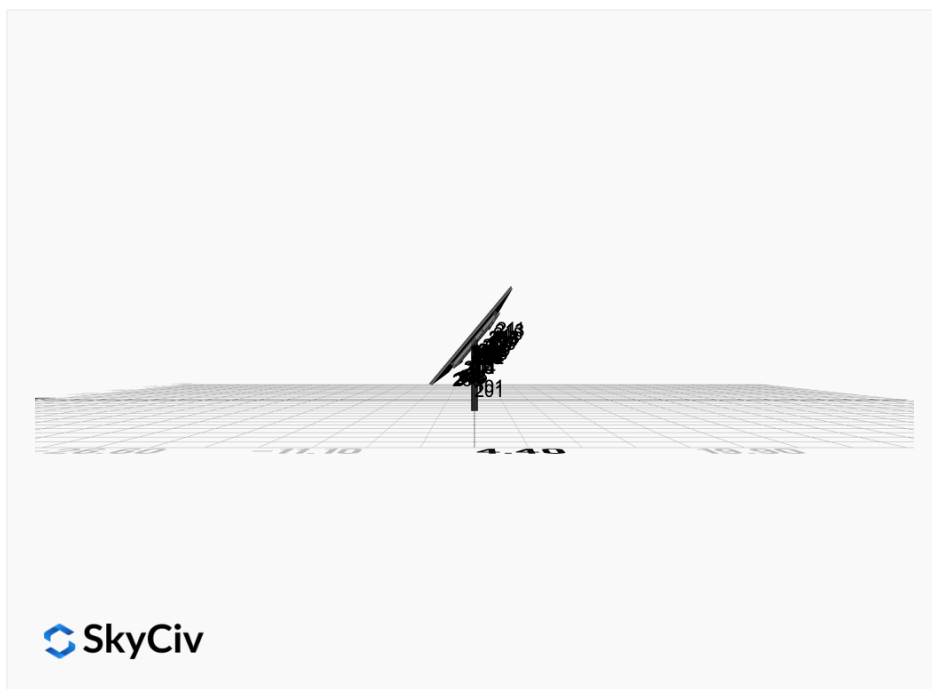
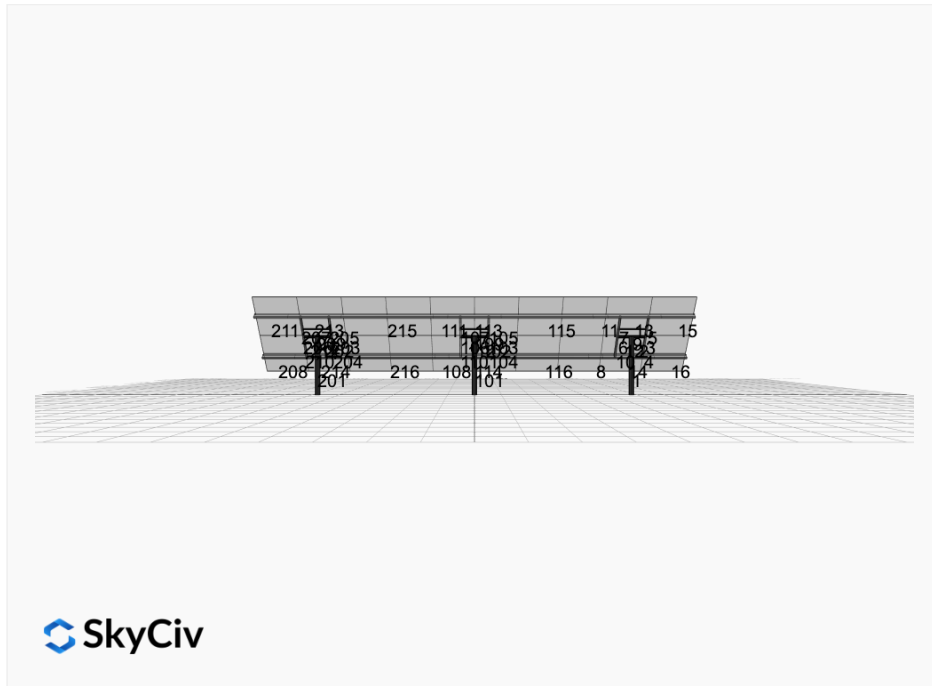
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

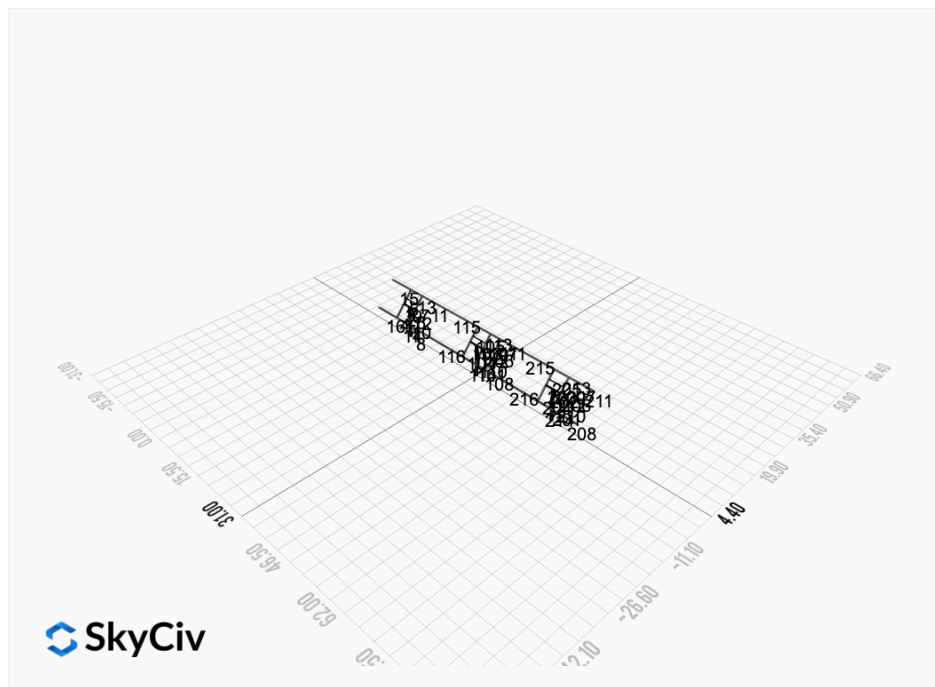
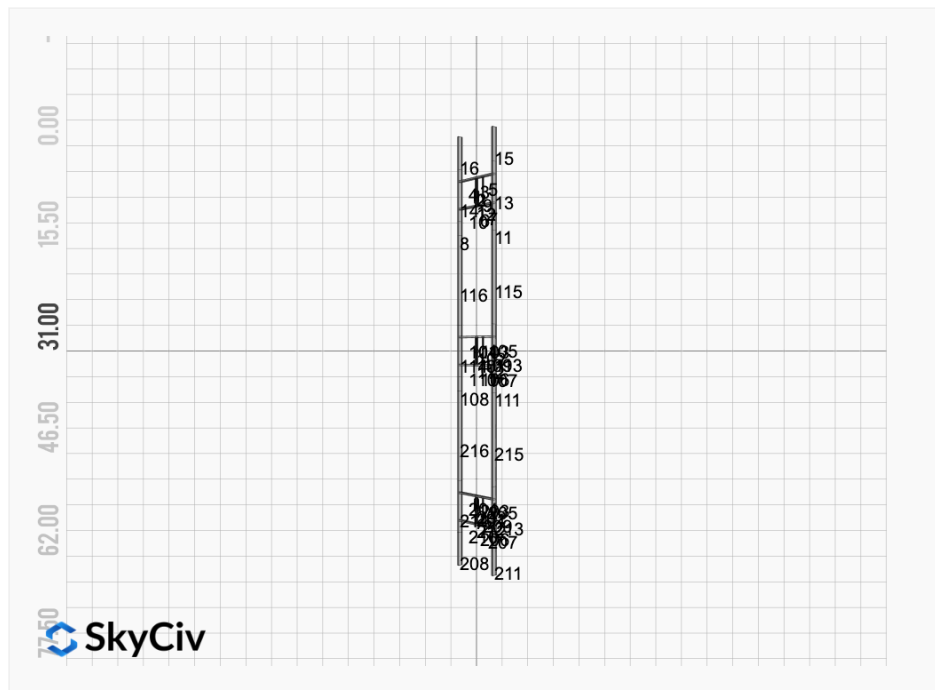
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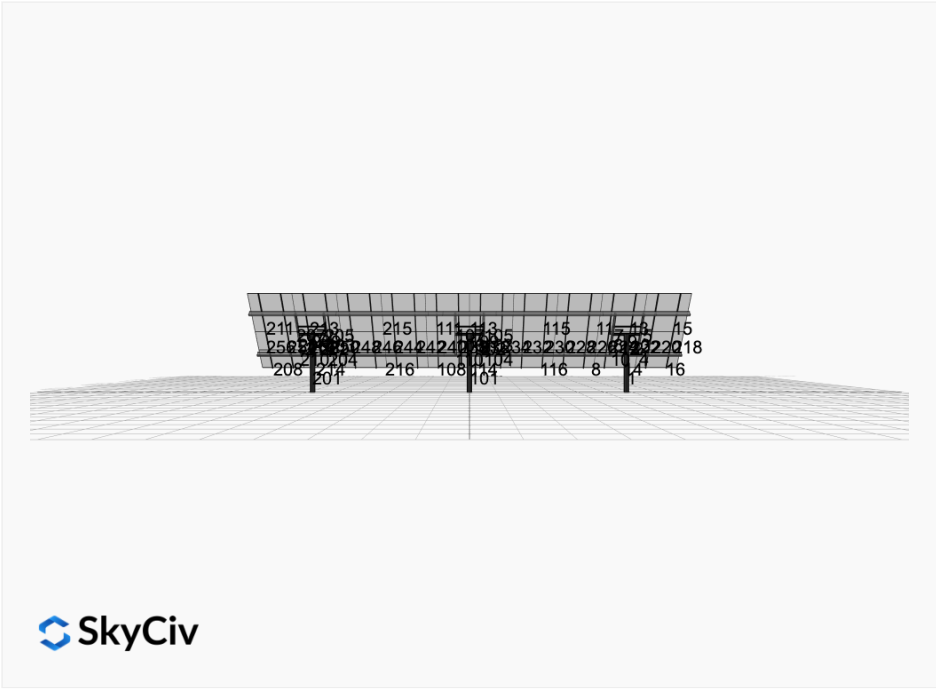
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Design Notes:

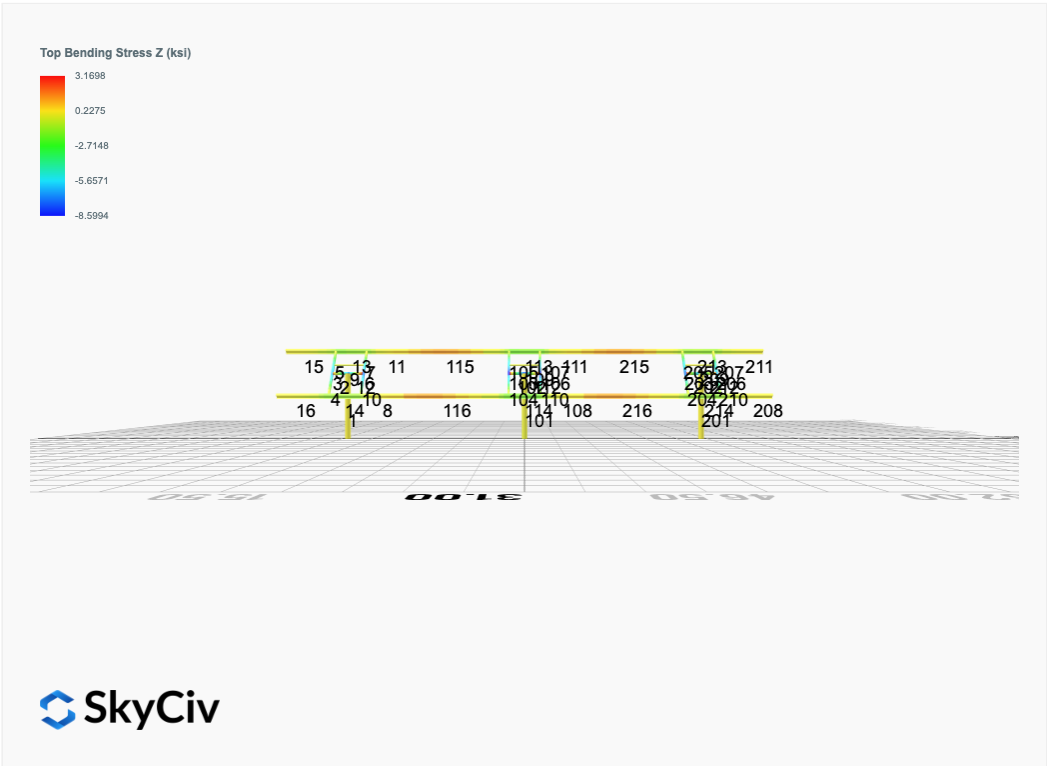
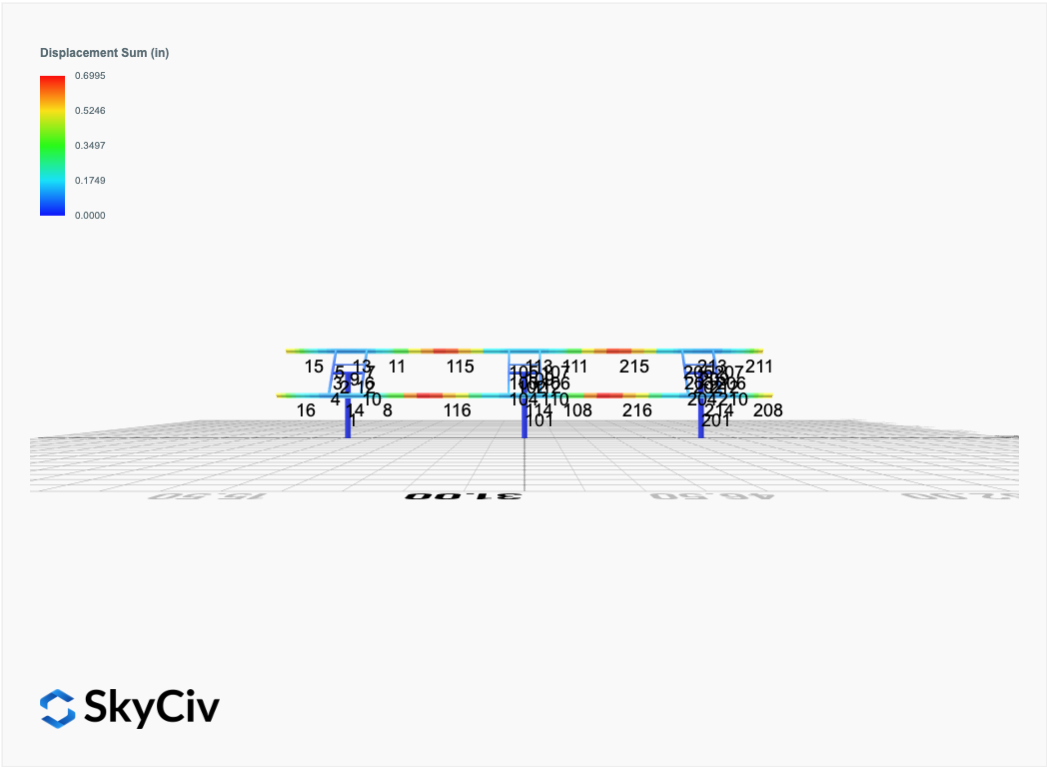
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



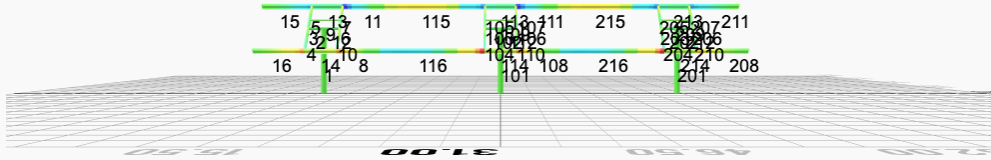
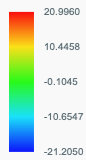




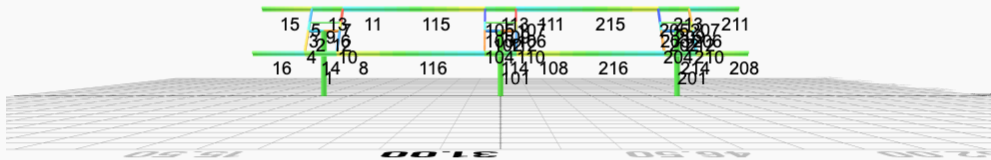
FEM Results (Envelope Worst Case for each member)

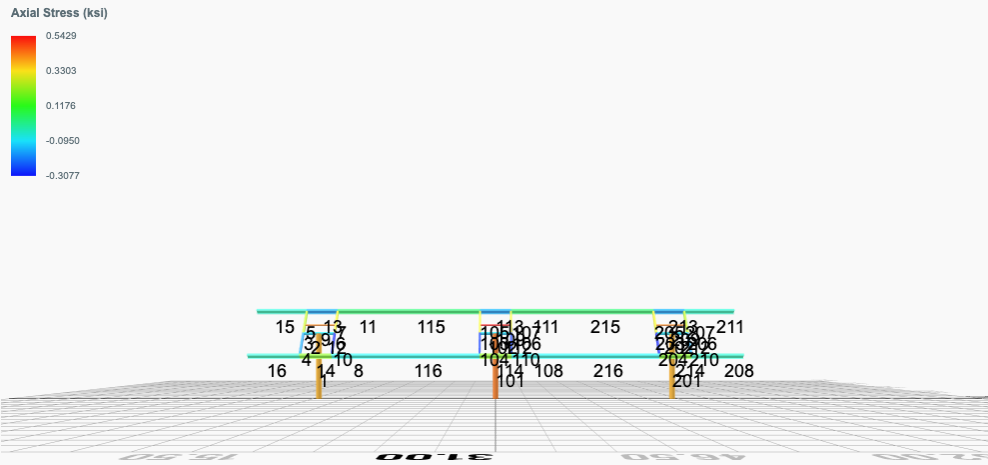


Top Bending Stress Y (ksi)



Shear Stress Y (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	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 2. D + L	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 3. D + (S or Lr or R)	0.0310	5.5996	0.1303	0.2703	-0.0521	-0.2064
ULS: 3. D + (S or Lr or R)	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0258	4.7331	0.1083	0.2246	-0.0432	-0.1694
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 5b. D + 0.7E	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0258	4.7331	0.1083	0.2246	-0.0432	-0.1694
ULS: 8. 0.6D + 0.7E	0.0061	1.2802	0.0254	0.0525	-0.0100	-0.0350
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.4891	4.2254	0.1190	0.2169	-0.3783	20.8982
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5090	0.0422	-0.0335	-0.0406	0.3420	-20.7800
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8486	6.3019	0.1658	0.3217	-0.3144	15.5480
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0258	4.7331	0.1083	0.2246	-0.0432	-0.1694
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8999	3.1645	0.0514	0.1285	0.2258	-15.7106
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0258	4.7331	0.1083	0.2246	-0.0432	-0.1694
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8643	3.7025	0.0999	0.1846	-0.2879	15.6591
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8842	0.5651	-0.0145	-0.0086	0.2523	-15.5995
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0101	2.1337	0.0424	0.0875	-0.0167	-0.0583
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.4932	3.3719	0.1021	0.1819	-0.3716	20.9215
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0061	1.2802	0.0254	0.0525	-0.0100	-0.0350
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5049	-0.8113	-0.0505	-0.0756	0.3487	-20.7567
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0061	1.2802	0.0254	0.0525	-0.0100	-0.0350

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	9.8493
Shear X	-4.1877
Shear Z	0.2575
Moment X	0.5136
Moment Y (Twist)	0.6422
Moment Z	35.0803

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	6.3019
Shear X	-2.5090
Shear Z	0.1658
Moment X	0.3217
Moment Y (Twist)	0.3783
Moment Z	20.9215

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	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 2. D + L	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 3. D + (S or Lr or R)	-0.0620	6.4028	0.0000	-0.0000	0.0001	0.5272
ULS: 3. D + (S or Lr or R)	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0516	5.4012	0.0000	-0.0000	0.0001	0.4403

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 5b. D + 0.7E	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0516	5.4012	0.0000	-0.0000	0.0001	0.4403
ULS: 8. 0.6D + 0.7E	-0.0121	1.4378	0.0000	0.0000	0.0000	0.1077
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9274	4.8465	0.0000	-0.0000	0.0000	24.3588
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8877	-0.0544	0.0000	-0.0000	0.0000	-23.7015
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2320	7.2388	0.0000	-0.0000	0.0001	18.5748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0516	5.4012	0.0000	-0.0000	0.0001	0.4403
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1294	3.5632	0.0000	-0.0000	0.0001	-17.4705
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0516	5.4012	0.0000	-0.0000	0.0001	0.4403
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2006	4.2339	0.0000	-0.0000	0.0000	18.3140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1607	0.5583	0.0000	-0.0000	0.0000	-17.7313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0202	2.3963	0.0000	0.0000	0.0000	0.1795
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9193	3.8880	0.0000	-0.0000	0.0000	24.2870
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0121	1.4378	0.0000	0.0000	0.0000	0.1077
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8958	-1.0129	0.0000	-0.0000	0.0000	-23.7734
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0121	1.4378	0.0000	0.0000	0.0000	0.1077

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	11.3273
Shear X	-4.8896
Shear Z	0.0001
Moment X	-0.0002
Moment Y (Twist)	0.0007
Moment Z	41.0613

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	7.2388
Shear X	-2.9274
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0001
Moment Z	24.3588

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 2. D + L	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 3. D + (S or Lr or R)	0.0310	5.5996	-0.1303	-0.2707	0.0522	-0.2063
ULS: 3. D + (S or Lr or R)	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0258	4.7331	-0.1083	-0.2249	0.0434	-0.1693
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 5b. D + 0.7E	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0258	4.7331	-0.1083	-0.2249	0.0434	-0.1693
ULS: 8. 0.6D + 0.7E	0.0061	1.2802	-0.0254	-0.0525	0.0101	-0.0350
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.4891	4.2254	-0.1190	-0.2170	0.3784	20.8982
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5089	0.0422	0.0335	0.0406	-0.3420	-20.7799
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8486	6.3019	-0.1658	-0.3219	0.3146	15.5481
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0258	4.7331	-0.1083	-0.2249	0.0434	-0.1693
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8999	3.1645	-0.0514	-0.1288	-0.2257	-15.7106
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0258	4.7331	-0.1083	-0.2249	0.0434	-0.1693
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8643	3.7024	-0.0999	-0.1846	0.2880	15.6591
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8842	0.5651	0.0145	0.0085	-0.2523	-15.5995
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0101	2.1337	-0.0424	-0.0876	0.0168	-0.0583
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.4932	3.3719	-0.1021	-0.1819	0.3717	20.9215
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0061	1.2802	-0.0254	-0.0525	0.0101	-0.0350
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5049	-0.8113	0.0505	0.0756	-0.3487	-20.7566
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0061	1.2802	-0.0254	-0.0525	0.0101	-0.0350

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	9.8492
Shear X	-4.1877
Shear Z	-0.2575
Moment X	-0.5152
Moment Y (Twist)	0.6428
Moment Z	35.0809

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	6.3019
Shear X	-2.5089
Shear Z	-0.1658
Moment X	-0.3219
Moment Y (Twist)	0.3784
Moment Z	20.9215

Project Details

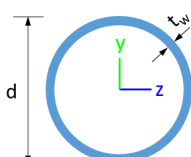
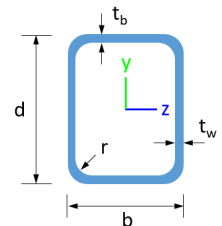
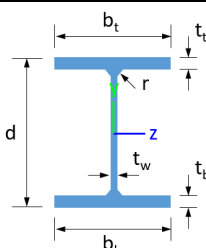
Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: Libby Residence - V1Jb
 Unit System: imperial



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)
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Section Dimensions								
								
ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
9	8in Pipe Sch 40	8.63	0.32					
								
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		
								
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	LS T	LS C	L D	
1	9	17.45	17.45	8.31	-	300	200	1	
2	5	1.30	1.30	2.00	-	300	200	1	
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	300	200	1	
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.68,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1	
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1	
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
8	19	1.33	1.33	2.05	2.10,2.10,2.10,2.10,2.10,2.10,2.10,2.10,2.09,2.10,2.10,2.10,2.09,2.10,2.10,2.10,2.12,2.10,2.10,2.10,2.09,2.10,2.10,2.10,2.09,2.10	300	200	1	
9	2	2.60	2.60	4.00	-	300	200	1	
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
11	19	1.33	1.33	2.05	2.11,2.11,2.11,2.11,2.11,2.11,2.12,2.11,2.12,2.11,2.12,2.11,2.12,2.11,2.11,2.11,2.10,2.11,2.12,2.11,2.13,2.11,2.12,2.11,2.12,2.11	300	200	1	
12	5	1.30	1.30	2.00	-	300	200	1	
13	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.07,1.08,1.08,1.08,1.07,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.07,1.08,1.08,1.08,1.08,1.08	300	200	1	
14	19	4.88	4.00	7.50	1.08,1.08	300	200	1	
15	19	9.97	9.97	4.75	2.33,2.33	300	200	1	
16	19	9.97	9.97	4.75	2.33,2.33	300	200	1	
101	9	17.45	17.45	8.31	-	300	200	1	
102	5	1.30	1.30	2.00	-	300	200	1	
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1	
104	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1	
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
108	19	1.33	1.33	2.05	2.27,2.27,2.27,2.27,2.27,2.27,2.27,2.27,2.27,2.28,2.27,2.27,2.27,2.28,2.27,2.27,2.27,2.26,2.27,2.27,2.27,2.28,2.27,2.27,2.27,2.28	300	200	1	
109	2	2.60	2.60	4.00	-	300	200	1	
110	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
111	19	1.33	1.33	2.05	2.29,2.29,2.29,2.29,2.29,2.29,2.33,2.29,2.10,2.29,2.25,2.29,2.13,2.29,2.33,2.29,2.14,2.29,2.36,2.29,2.10,2.29,2.22,2.29,2.13,2.29	300	200	1	
112	5	1.30	1.30	2.00	-	300	200	1	

101	377.97	260.65	83.29	83.29	113.39	113.39
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.46	6.12	40.24	43.62
114	133.20	85.85	23.49	6.12	40.24	43.62
115	133.20	46.28	12.30	6.12	40.24	43.62
116	133.20	46.28	12.71	6.12	40.24	43.62
201	377.97	260.65	83.29	83.29	113.39	113.39
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	32.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	32.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	24.60	6.12	40.24	43.62
214	133.20	85.85	24.64	6.12	40.24	43.62
215	133.20	46.28	12.32	6.12	40.24	43.62
216	133.20	46.28	12.29	6.12	40.24	43.62

Design Ratio

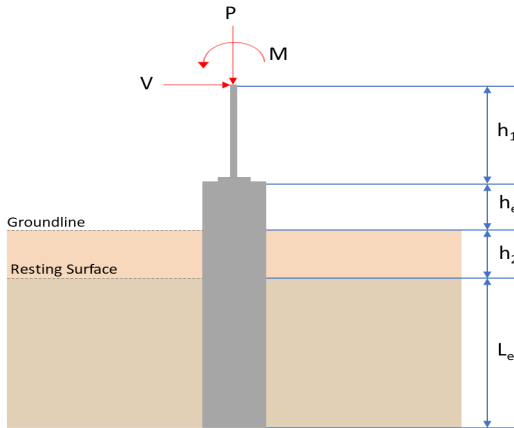
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.038	0.421	0.019	0.037	0.002	0.441	#13	0.356	Not Required	Pass
2	0.006	0.289	0.181	0.071	0.034	0.422	#13	0.035	Not Required	Pass
3	0.011	0.440	0.061	0.044	0.008	0.466	#21	0.045	Not Required	Pass
4	0.011	0.438	0.222	0.044	0.047	0.571	#21	0.080	Not Required	Pass
5	0.011	0.273	0.221	0.044	0.057	0.324	#21	0.074	Not Required	Pass
6	0.015	0.507	0.095	0.051	0.012	0.590	#21	0.045	Not Required	Pass
7	0.015	0.314	0.297	0.050	0.076	0.384	#21	0.074	Not Required	Pass
8	0.002	0.048	0.307	0.036	0.025	0.333	#21	0.095	Not Required	Pass
9	0.025	0.035	0.073	0.002	0.002	0.095	#13	0.204	Not Required	Pass
10	0.015	0.495	0.279	0.050	0.058	0.650	#21	0.080	Not Required	Pass
11	0.004	0.045	0.317	0.037	0.025	0.348	#21	0.095	Not Required	Pass
12	0.005	0.363	0.210	0.089	0.039	0.517	#13	0.035	Not Required	Pass
13	0.010	0.188	0.672	0.045	0.031	0.826	#21	0.286	Not Required	Pass
14	0.013	0.187	0.660	0.045	0.031	0.805	#21	0.190	Not Required	Pass
15	0.000	0.060	0.272	0.024	0.016	0.228	#21	Not Required	Not Required	Pass

15	0.000	0.009	0.272	0.024	0.010	0.338	#21	Not Required	Not Required	Pass
16	0.000	0.069	0.272	0.024	0.016	0.338	#21	Not Required	Not Required	Pass
101	0.043	0.493	0.000	0.043	0.000	0.510	#13	0.356	Not Required	Pass
102	0.007	0.377	0.230	0.093	0.041	0.548	#13	0.035	Not Required	Pass
103	0.015	0.547	0.073	0.055	0.002	0.612	#21	0.045	Not Required	Pass
104	0.015	0.552	0.283	0.055	0.058	0.716	#21	0.080	Not Required	Pass
105	0.015	0.339	0.297	0.054	0.076	0.412	#21	0.074	Not Required	Pass
106	0.015	0.547	0.073	0.055	0.002	0.612	#21	0.045	Not Required	Pass
107	0.015	0.339	0.298	0.054	0.076	0.412	#21	0.074	Not Required	Pass
108	0.002	0.055	0.311	0.039	0.025	0.365	#21	0.095	Not Required	Pass
109	0.030	0.033	0.056	0.001	0.000	0.096	#21	0.204	Not Required	Pass
110	0.015	0.552	0.283	0.055	0.058	0.716	#21	0.080	Not Required	Pass
111	0.004	0.046	0.319	0.038	0.025	0.367	#21	0.095	Not Required	Pass
112	0.007	0.377	0.230	0.093	0.041	0.548	#13	0.035	Not Required	Pass
113	0.010	0.202	0.674	0.047	0.031	0.864	#21	0.286	Not Required	Pass
114	0.014	0.223	0.665	0.047	0.031	0.864	#21	0.286	Not Required	Pass
115	0.010	0.341	0.366	0.038	0.025	0.699	#21	0.601	Not Required	Pass
116	0.002	0.333	0.364	0.039	0.025	0.684	#21	0.601	Not Required	Pass
201	0.038	0.421	0.019	0.037	0.002	0.441	#13	0.356	Not Required	Pass
202	0.005	0.363	0.210	0.089	0.039	0.517	#13	0.035	Not Required	Pass
203	0.015	0.507	0.095	0.051	0.012	0.590	#21	0.045	Not Required	Pass
204	0.015	0.495	0.279	0.050	0.058	0.650	#21	0.080	Not Required	Pass
205	0.015	0.314	0.297	0.050	0.076	0.384	#21	0.074	Not Required	Pass
206	0.011	0.440	0.061	0.044	0.008	0.466	#21	0.045	Not Required	Pass
207	0.011	0.273	0.221	0.044	0.057	0.324	#21	0.074	Not Required	Pass
208	0.000	0.069	0.272	0.024	0.016	0.338	#21	Not Required	Not Required	Pass
209	0.025	0.035	0.073	0.002	0.002	0.095	#13	0.204	Not Required	Pass
210	0.011	0.438	0.222	0.044	0.047	0.571	#21	0.080	Not Required	Pass
211	0.000	0.069	0.272	0.024	0.016	0.338	#21	Not Required	Not Required	Pass
212	0.006	0.289	0.181	0.071	0.034	0.422	#13	0.035	Not Required	Pass
213	0.010	0.188	0.671	0.045	0.031	0.825	#21	0.190	Not Required	Pass
214	0.013	0.187	0.660	0.045	0.031	0.806	#21	0.286	Not Required	Pass
215	0.010	0.349	0.366	0.037	0.025	0.702	#21	0.601	Not Required	Pass
216	0.002	0.340	0.363	0.036	0.025	0.689	#21	0.601	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																											
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																												
	Pile Input  Geometry Pile shape: rectangular b = 48 in - Pile width D = 48 in - Pile depth L = 5.75 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>6.302</td><td>9.849</td></tr><tr><td>Vx (kip)</td><td>-2.509</td><td>-4.188</td></tr><tr><td>Vz (kip)</td><td>0.166</td><td>0.257</td></tr><tr><td>Mx (kipft)</td><td>0.322</td><td>0.514</td></tr><tr><td>Mz (kipft)</td><td>20.922</td><td>35.080</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.302	9.849	Vx (kip)	-2.509	-4.188	Vz (kip)	0.166	0.257	Mx (kipft)	0.322	0.514	Mz (kipft)	20.922	35.080	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: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 \text{ } D}$ $H_o = \frac{(-2.509 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.39952 \text{ kip/ft}$	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																										
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																										
Load Component	ASD	LRFD																											
P (kip)	6.302	9.849																											
Vx (kip)	-2.509	-4.188																											
Vz (kip)	0.166	0.257																											
Mx (kipft)	0.322	0.514																											
Mz (kipft)	20.922	35.080																											

	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(20.922 \text{ kipft}) + ((-2.509 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.3315 \text{ kipft/ft}$ 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: $L_{e,x} = 5.212 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.166 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.026433 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.322 \text{ kipft}) + ((0.166 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.051274 \text{ kipft/ft}$ 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: $L_{e,z} = 1.9272 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.212 \text{ ft}), (1.9272 \text{ ft})]$ $L_{e,req} = 5.212 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.75 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.212 \text{ ft})}{(5.75 \text{ ft})}$ $\text{Ratio} = 0.90643$	Status: PASS Ratio: 0.910
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure	

	$q = \frac{P_v}{A}$ $q = \frac{(6.302 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.39387 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.39387 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.19694$	Status: PASS Ratio: 0.200
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.4375$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.39952 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 3.3315 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (3.3315 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.3315 \text{ kipft/ft})) + (4 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 3.9842 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (3.3315 \text{ kipft/ft})) + (3 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.3315 \text{ kipft/ft})) + (2 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.17391 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (3.3315 \text{ kipft/ft})) + ((-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.79228 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.9842 \text{ ft})}{2}$ $p_a = 0.29882 \text{ kip/ft}^2$ Ratio - Lateral soil capacity	

	$Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.17391 \text{ kip/ft}^2)}{(0.29882 \text{ kip/ft}^2)}$ $Ratio = 0.58201$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{s}{p_s}$ $Ratio = \frac{(0.79228 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $Ratio = 0.91859$	Status: PASS Ratio: 0.580 Status: PASS Ratio: 0.920
	Considering z-direction: $H_o = 0.026433 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.051274 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.051274 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.026433 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.051274 \text{ kipft/ft})) + (4 \times (0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.1515 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.051274 \text{ kipft/ft})) + (3 \times (0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.051274 \text{ kipft/ft})) + (2 \times (0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.021654 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.051274 \text{ kipft/ft})) + ((0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.046192 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.1515 \text{ ft})}{2}$ $p_a = 0.31136 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.021654 \text{ kip/ft}^2)}{(0.31136 \text{ kip/ft}^2)}$	

$$Ratio = 0.069546$$

Status: **PASS**
Ratio: **0.070**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

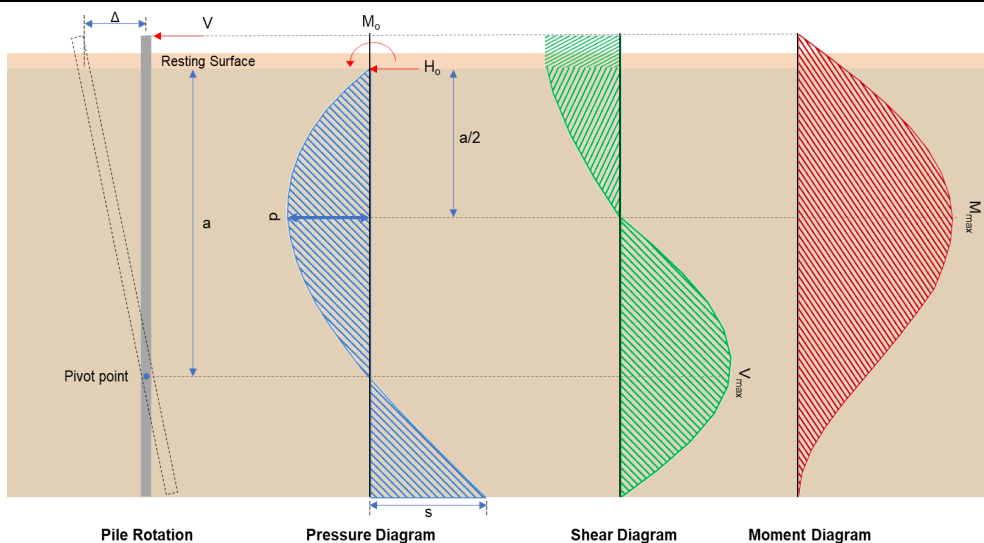
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.046192 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.053556$$

Status: **PASS**
Ratio: **0.050**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.188 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.66688 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.08 \text{ kipft}) + ((-4.188 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.586 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.586 \text{ kipft/ft})}{(-0.66688 \text{ kip/ft})}$$

$$E = 8.3763 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.586 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.66688 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.586 \text{ kipft/ft})) + (4 \times (-0.66688 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.66688 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.586 \text{ kip/ft})) + (4 \times (-0.66688 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9838 \text{ ft}$$

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

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

$$V_{max} = ((-0.66688 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.3763 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9838 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.3763 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9838 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.635 \text{ kip}$$

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

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

$$M_{max} = ((-0.66688 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(8.3763 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9838 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.3763 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9838 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.3763 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9838 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.436 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.257 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.040924 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.514 \text{ kipft}) + ((0.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.081847 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.081847 \text{ kipft/ft})}{(0.040924 \text{ kip/ft})}$$

$$E = 2 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.081847 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.040924 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.081847 \text{ kipft/ft})) + (4 \times (0.040924 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1482 \text{ ft}$$

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

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

$$V_{max} = ((0.040924 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1482 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1482 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21043 \text{ kip}$$

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

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

$$M_{max} = ((0.040924 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1482 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1482 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1482 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.52141 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(9.849 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.269 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max[(-84.269 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

$$Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

25.2.3	$Ratio = 0.96556$ s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ 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, $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	Status: PASS Ratio: 0.970
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ $Ratio$ - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.849 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0036816$	Status: PASS Ratio: 0.000
22.5.2.2 22.5.1.3 22.5.1.1	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.849 \text{ kip} \rightarrow 9849 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9849 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.8 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.8 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.8 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.8 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.95 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.635 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

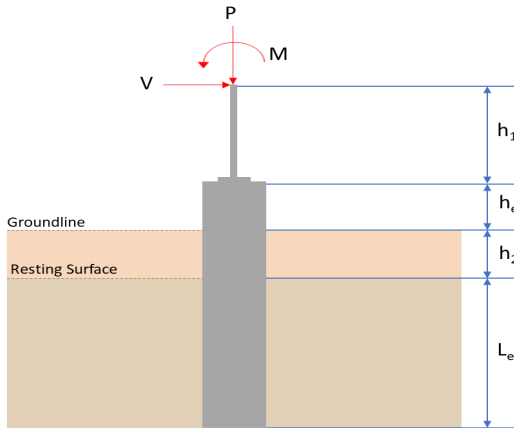
$$Ratio = \frac{V_{max}}{\phi V_n}$$

	$Ratio = \frac{(8.635 \text{ kip})}{(110.95 \text{ kip})}$ $Ratio = 0.077827$ Considering z-direction: $V_{max} = 0.21043 \text{ kip}$ - Maximum shear force in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.21043 \text{ kip})}{(110.95 \text{ kip})}$ $Ratio = 0.0018966$ Status: PASS Ratio: 0.080	
14.5.2.1b	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 23.436 \text{ kipft}$ - Maximum moment in the x-direction, <i>Ratio</i> - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(23.436 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 0.093895$ Status: PASS Ratio: 0.090	
	Considering z-direction: $M_{max} = 0.52141 \text{ kipft}$ - Maximum moment in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$	

$$Ratio = \frac{(0.52141 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.002089$$

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

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 = 48 in - Pile width D = 48 in - Pile depth L = 5.75 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>6.302</td><td>9.849</td></tr><tr><td>Vx (kip)</td><td>-2.509</td><td>-4.188</td></tr><tr><td>Vz (kip)</td><td>-0.166</td><td>-0.258</td></tr><tr><td>Mx (kipft)</td><td>-0.322</td><td>-0.515</td></tr><tr><td>Mz (kipft)</td><td>20.922</td><td>35.081</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.302	9.849	Vx (kip)	-2.509	-4.188	Vz (kip)	-0.166	-0.258	Mx (kipft)	-0.322	-0.515	Mz (kipft)	20.922	35.081	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: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 \text{ } D}$ $H_o = \frac{(-2.509 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.39952 \text{ kip/ft}$	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																										
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	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(20.922 \text{ kipft}) + ((-2.509 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.3315 \text{ kipft/ft}$ 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: $L_{e,x} = 5.212 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.166 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.026433 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.322 \text{ kipft}) + ((-0.166 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.051274 \text{ kipft/ft}$ 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: $L_{e,z} = 1.2761 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.212 \text{ ft}), (1.2761 \text{ ft})]$ $L_{e,req} = 5.212 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.75 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.212 \text{ ft})}{(5.75 \text{ ft})}$ $\text{Ratio} = 0.90643$	Status: PASS Ratio: 0.910
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure	

	$q = \frac{P_v}{A}$ $q = \frac{(6.302 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.39387 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.39387 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.19694$	Status: PASS Ratio: 0.200
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.4375$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.39952 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 3.3315 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (3.3315 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.3315 \text{ kipft/ft})) + (4 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 3.9842 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (3.3315 \text{ kipft/ft})) + (3 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.3315 \text{ kipft/ft})) + (2 \times (-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.17391 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (3.3315 \text{ kipft/ft})) + ((-0.39952 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.79228 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.9842 \text{ ft})}{2}$ $p_a = 0.29882 \text{ kip/ft}^2$ Ratio - Lateral soil capacity	

	$Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.17391 \text{ kip/ft}^2)}{(0.29882 \text{ kip/ft}^2)}$ $Ratio = 0.58201$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{s}{p_s}$ $Ratio = \frac{(0.79228 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $Ratio = 0.91859$	Status: PASS Ratio: 0.580 Status: PASS Ratio: 0.920
	Considering z-direction: $H_o = -0.026433 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.051274 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.051274 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.026433 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.051274 \text{ kipft/ft})) + (4 \times (-0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.1515 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.051274 \text{ kipft/ft})) + (3 \times (-0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.051274 \text{ kipft/ft})) + (2 \times (-0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = -0.009508 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.051274 \text{ kipft/ft})) + ((-0.026433 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = -0.0089726 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.1515 \text{ ft})}{2}$ $p_a = 0.31136 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(-0.009508 \text{ kip/ft}^2)}{(0.31136 \text{ kip/ft}^2)}$	

$$Ratio = -0.030537$$

Status: **PASS**
Ratio: **-0.030**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

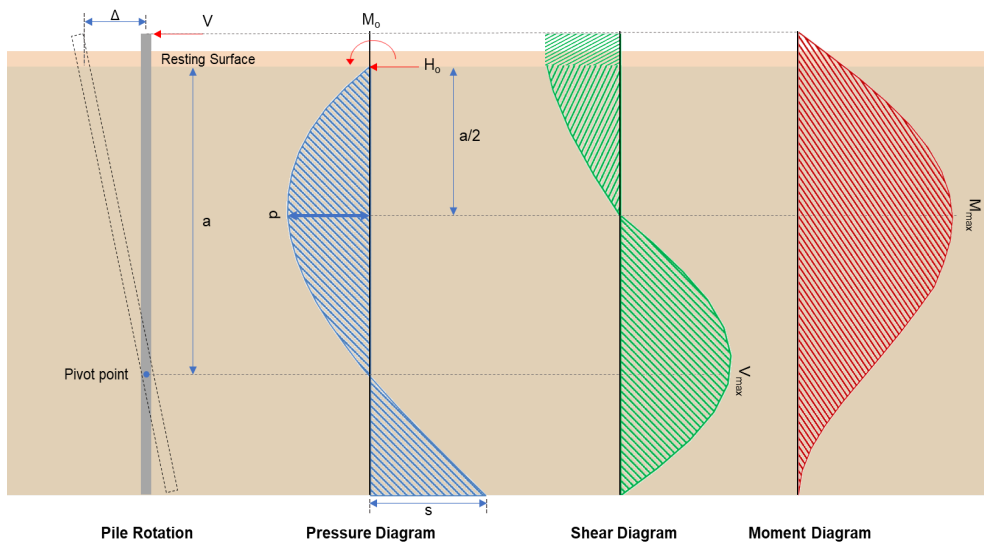
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0089726 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = -0.010403$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.188 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.66688 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.081 \text{ kipft}) + ((-4.188 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.5861 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.5861 \text{ kipft/ft})}{(-0.66688 \text{ kip/ft})}$$

$$E = 8.3766 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.5861 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.66688 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.5861 \text{ kipft/ft})) + (4 \times (-0.66688 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.66688 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})}{(6 \times (5.5861 \text{ kipft/ft})) + (4 \times (-0.66688 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9838 \text{ ft}$$

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

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

$$V_{max} = ((-0.66688 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.3766 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9838 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.3766 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9838 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.6352 \text{ kip}$$

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

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

$$M_{max} = ((-0.66688 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(8.3766 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9838 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.3766 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9838 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.3766 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9838 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.437 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.258 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.041083 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.515 \text{ kipft}) + ((-0.258 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.082006 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.082006 \text{ kipft/ft})}{(-0.041083 \text{ kip/ft})}$$

$$E = 1.9961 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.082006 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.041083 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.082006 \text{ kipft/ft})) + (4 \times (-0.041083 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1484 \text{ ft}$$

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

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

$$V_{max} = ((-0.041083 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.9961 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1484 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (1.9961 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1484 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21105 \text{ kip}$$

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

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

$$M_{max} = ((-0.041083 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(1.9961 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1484 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.9961 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1484 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (1.9961 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1484 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.52289 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(9.849 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.269 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-84.269 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

$$Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

25.2.3	$s_{rebar} = 0.96556$ s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ 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, $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	Status: PASS Ratio: 0.970
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ $Ratio$ - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.849 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0036816$	Status: PASS Ratio: 0.000
22.5.2.2 22.5.1.3 22.5.1.1	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.849 \text{ kip} \rightarrow 9849 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9849 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.8 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.8 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.8 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.8 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.95 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.6352 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

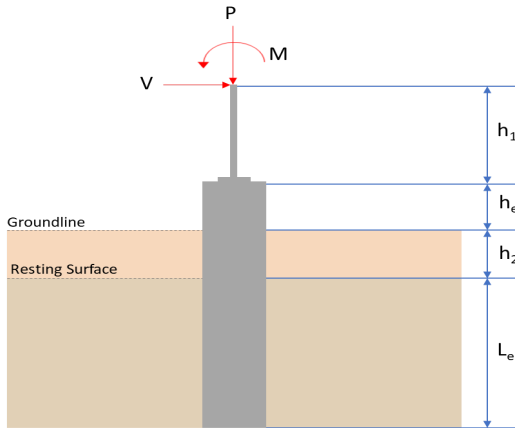
$$Ratio = \frac{V_{max}}{\phi V_n}$$

	$Ratio = \frac{(8.6352 \text{ kip})}{(110.95 \text{ kip})}$ $Ratio = 0.077829$ Considering z-direction: $V_{max} = 0.21105 \text{ kip}$ - Maximum shear force in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.21105 \text{ kip})}{(110.95 \text{ kip})}$ $Ratio = 0.0019022$ Status: PASS Ratio: 0.080	
14.5.2.1b	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 23.437 \text{ kipft}$ - Maximum moment in the x-direction, <i>Ratio</i> - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(23.437 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 0.093897$ Status: PASS Ratio: 0.090	
	Considering z-direction: $M_{max} = 0.52289 \text{ kipft}$ - Maximum moment in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$	

$$Ratio = \frac{(0.52289 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0020949$$

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

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 = 48 in - Pile width D = 48 in - Pile depth L = 6 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>7.239</td><td>11.327</td></tr><tr><td>Vx (kip)</td><td>-2.927</td><td>-4.890</td></tr><tr><td>Vz (kip)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mx (kipft)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mz (kipft)</td><td>24.359</td><td>41.061</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	7.239	11.327	Vx (kip)	-2.927	-4.890	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	24.359	41.061	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
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P (kip)	7.239	11.327																										
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Vz (kip)	0.000	0.000																										
Mx (kipft)	0.000	0.000																										
Mz (kipft)	24.359	41.061																										
	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: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 \text{ } D}$$H_o = \frac{(-2.927 \text{ kip})}{1.57 \times (48 \text{ in})}$$H_o = -0.46608 \text{ kip/ft}$																											

	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(24.359 \text{ kipft}) + ((-2.927 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.8788 \text{ kipft/ft}$ 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: $L_{e,x} = 5.4158 \text{ ft}$ - 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, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.4158 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.416 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.416 \text{ ft})}{(6 \text{ ft})}$ $\text{Ratio} = 0.90267$	Status: PASS Ratio: 0.900
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_v}{A}$ $q = \frac{(7.239 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.45244 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.45244 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.22622$	Status: PASS Ratio: 0.230
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,	

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.46608 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.8788 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8788 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.46608 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.8788 \text{ kipft/ft})) + (4 \times (-0.46608 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1623 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8788 \text{ kipft/ft})) + (3 \times (-0.46608 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.8788 \text{ kipft/ft})) + (2 \times (-0.46608 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.17504 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.8788 \text{ kipft/ft})) + ((-0.46608 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.82686 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1623 \text{ ft})}{2}$$

$$p_a = 0.31217 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17504 \text{ kip/ft}^2)}{(0.31217 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.56072$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

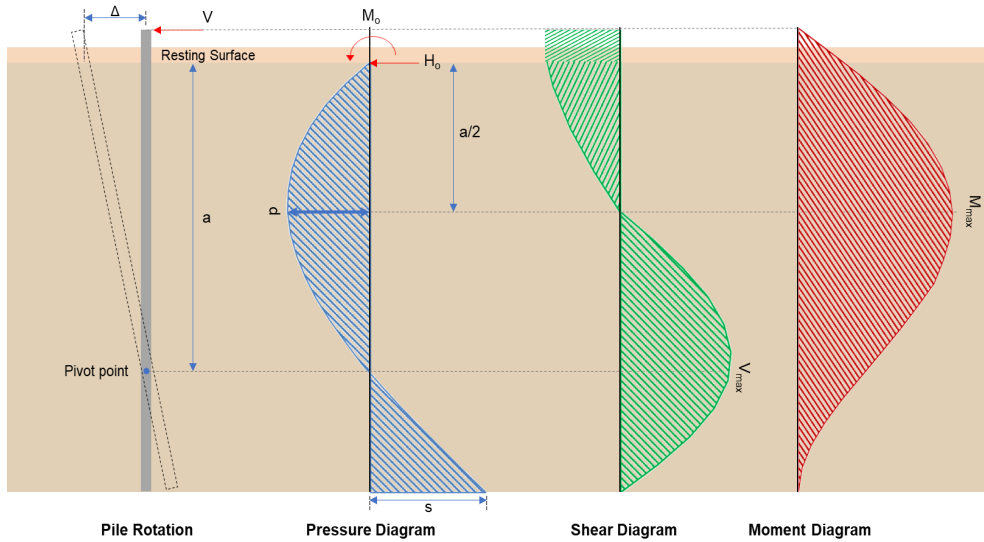
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.82686 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.560**

$$Ratio = 0.91873$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.89 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.77866 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.061 \text{ kipft}) + ((-4.89 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5384 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.5384 \text{ kipft/ft})}{(-0.77866 \text{ kip/ft})}$$

$$E = 8.3969 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5384 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.77866 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.5384 \text{ kipft/ft})) + (4 \times (-0.77866 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1613 \text{ ft}$$

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

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

$$V_{max} = ((-0.77866 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.3969 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1613 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.3969 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1613 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 9.1008 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-0.77866 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(8.3969 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1613 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.3969 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1613 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (8.3969 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1613 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 27.609 \text{ kipft}$	
Table 22.4.2.1 22.4.2.2, 10.6.1.1	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, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\left(\frac{(11.327 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.22 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.22 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$	Status: PASS Ratio: 0.970

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11327 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.08 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.7668 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(9.7668 \text{ kip})}{(111.08 \text{ kip})}$$

$$Ratio = 0.087928$$

Status: **PASS**
Ratio: **0.090**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 27.609 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(27.609 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.11061$$

Status: **PASS**
Ratio: **0.110**