

Your Project Calculations



Project Name: MulberryElementarySchool-NCGP2023-JB-RevB

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MulberryElementarySchool-NCGP2023-JB-RevB&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=uTb7AA2EDTWBo6braDGX8AmuB7FG48fsJowy8HNYoyesIsSkCUEXPLTP0WwMzYQC

Array Specification

Product:	Beam
Unique ID:	3P-22.5-8TOP-HD-57-L-5Hx9W-K63K
Duty Classification:	HD
Module Width:	40.91 in
Module Length:	82.52in
Number of Rows:	5
Number of Columns:	9
Total Number of Modules:	45
Desired Tilt Angle:	21
Front Edge Clearance:	10
Total Array Height at Tilt:	16.15 ft
Total Frame Length:	62.00 ft
Frame Weight:	3007 lbs
Array Dimensions N/S:	17.25 ft
Array Dimensions E/W:	62.64 ft
Rail Length:	207.05 in
Rail Spacing:	3.44 ft
Rail Check:	

Support Specifications

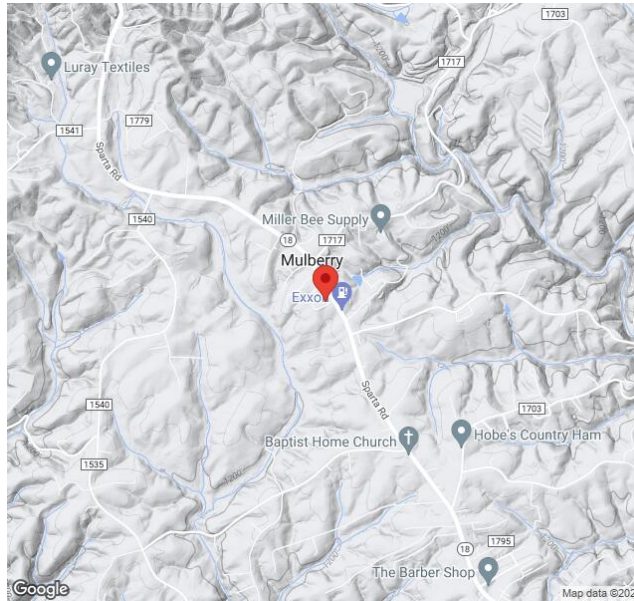
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	13.09 ft
Number of Poles:	3
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.50 ft Pile 2: 6.75 ft Pile 3: 6.50 ft
Foundation Volume:	11.704 y ³
Foundation Result:	PASSED
Mount Twist:	0.315099 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	190 Mulberry School Rd, North Wilkesboro, NC 28659, USA
Wind Speed:	115 mph
Snow Load:	20 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.010776 ksf



Design Disclaimer

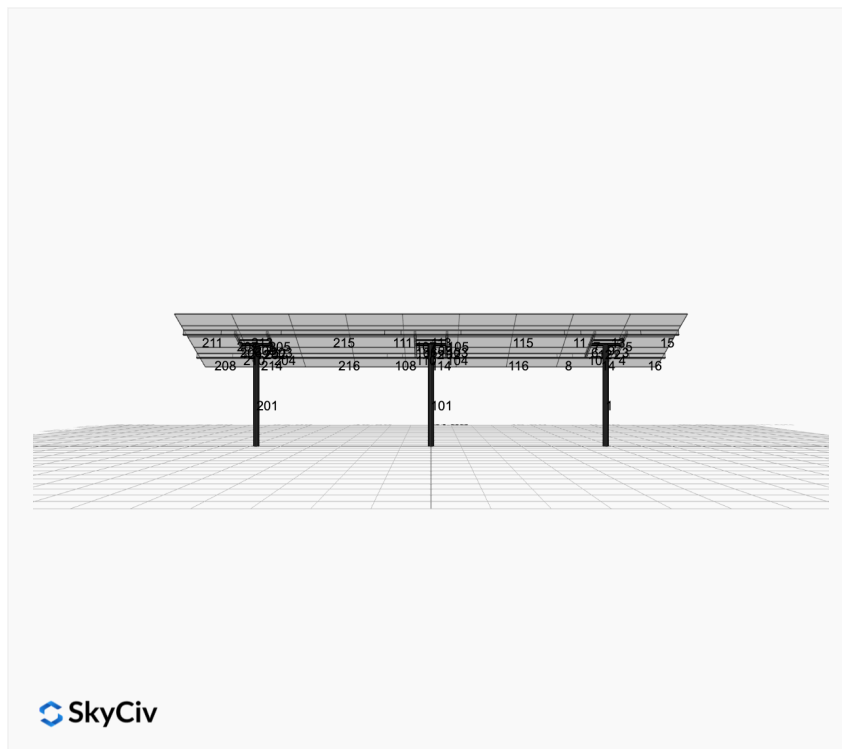
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

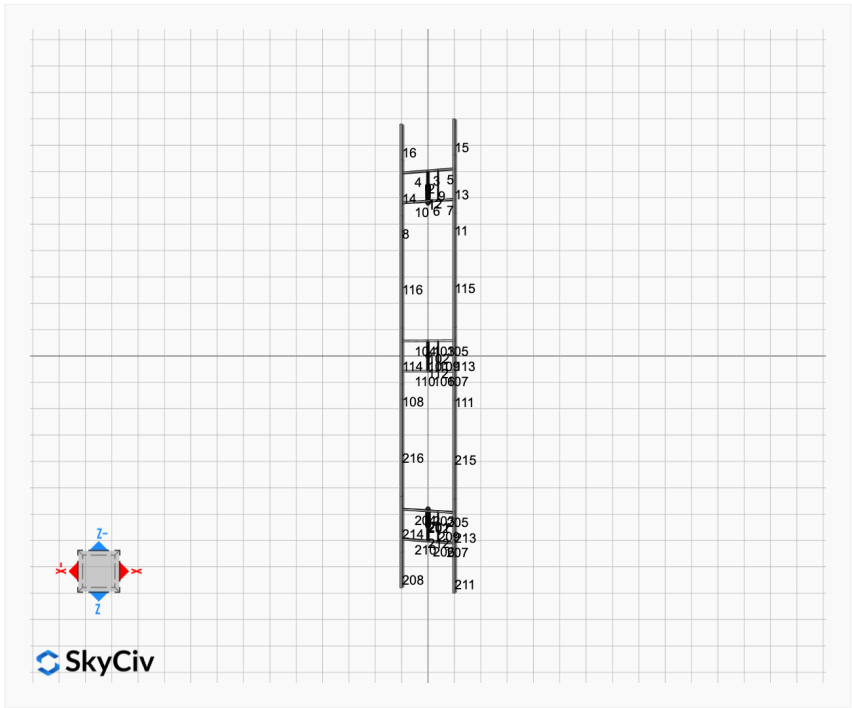
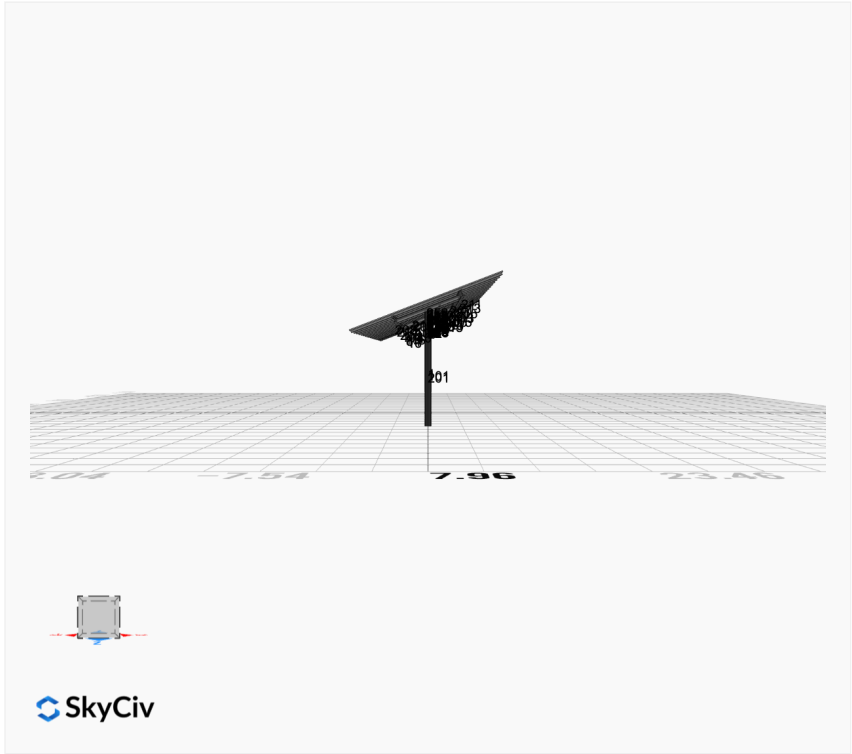
AutoDesigner Input

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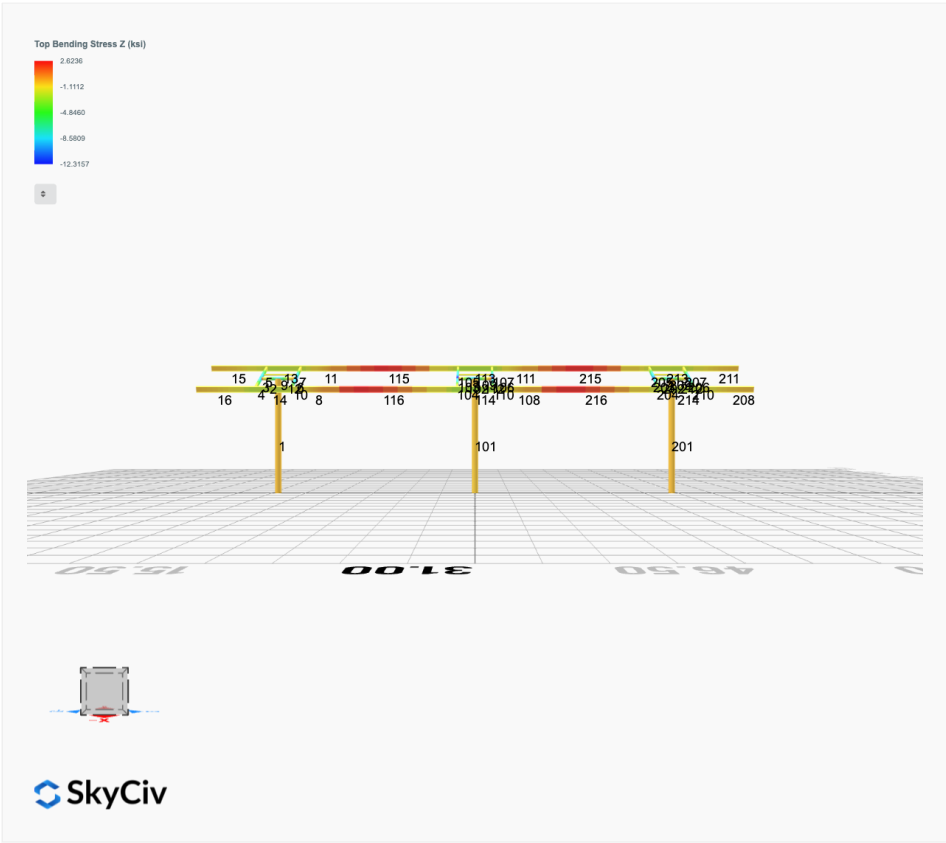
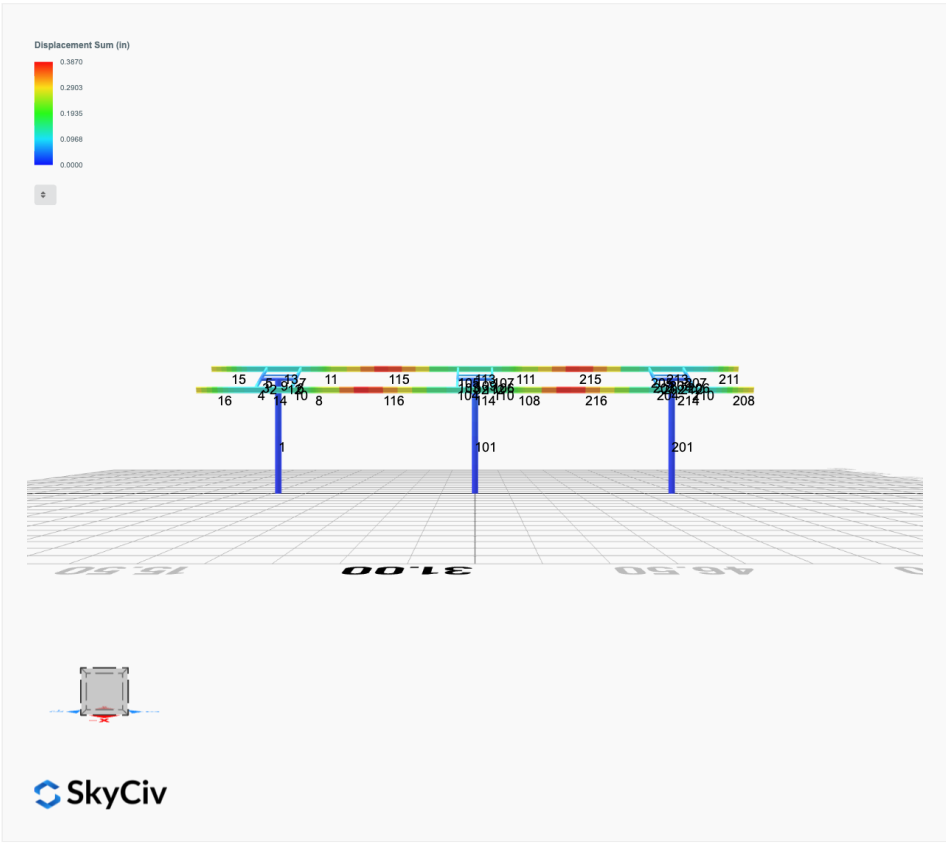
Design Notes:

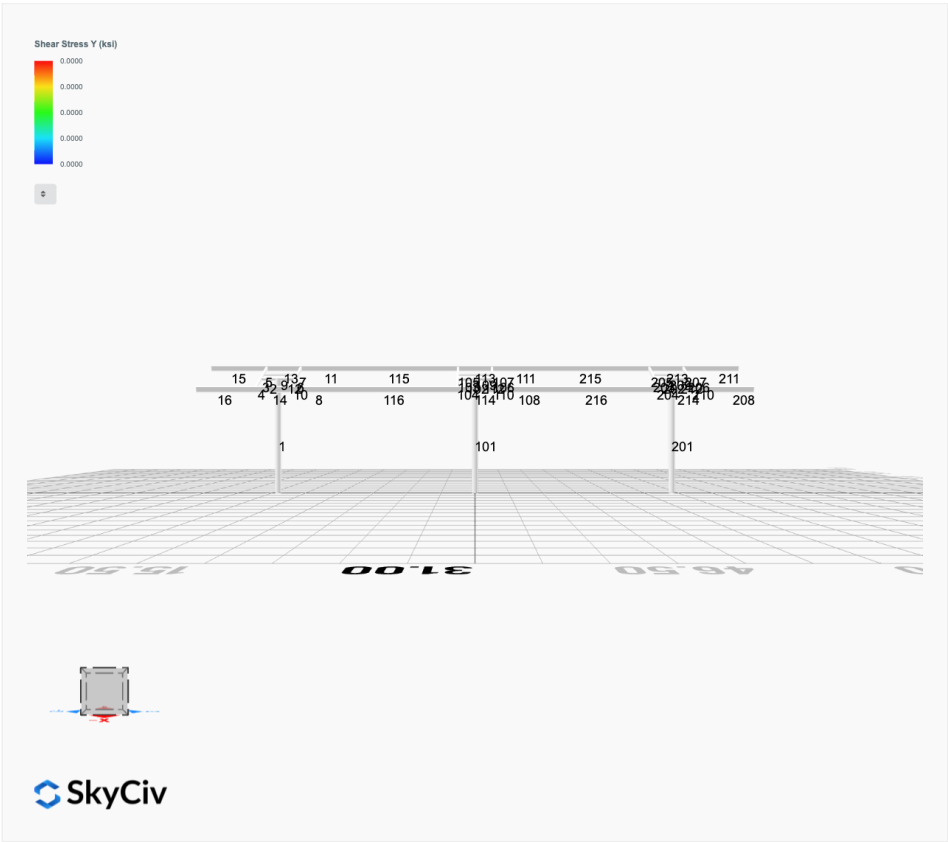
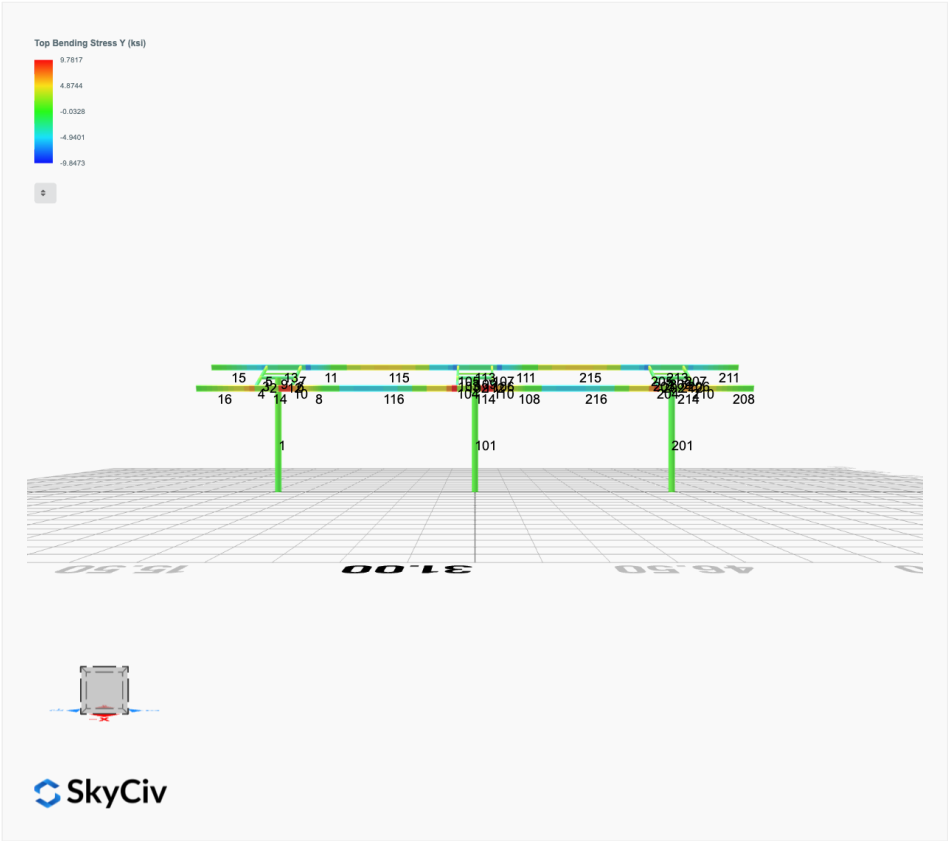
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

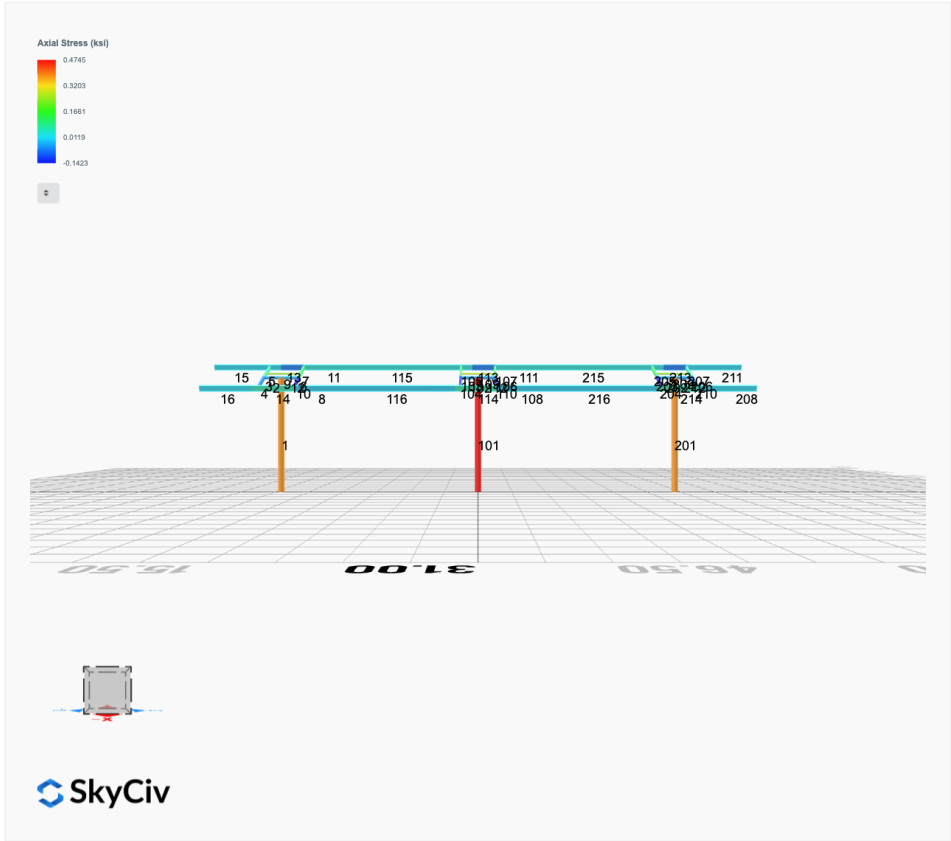




FEM Results (Envelope Worst Case for each member)







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	0.0073	2.5103	0.0342	0.1382	-0.0113	-0.0583
ULS: 2. D + L	0.0073	2.5103	0.0342	0.1382	-0.0113	-0.0583
ULS: 3. D + (S or Lr or R)	0.0202	5.8987	0.0947	0.3830	-0.0317	-0.2028
ULS: 3. D + (S or Lr or R)	0.0073	2.5103	0.0342	0.1382	-0.0113	-0.0583
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0170	5.0516	0.0796	0.3218	-0.0266	-0.1667
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0073	2.5103	0.0342	0.1382	-0.0113	-0.0583
ULS: 5b. D + 0.7E	0.0073	2.5103	0.0342	0.1382	-0.0113	-0.0583
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0170	5.0516	0.0796	0.3218	-0.0266	-0.1667
ULS: 8. 0.6D + 0.7E	0.0044	1.5062	0.0205	0.0829	-0.0068	-0.0350
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6687	6.8336	0.1309	0.5187	-0.1839	23.3879
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6687	6.8336	0.1309	0.5187	-0.1839	23.3879
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4514	-1.2214	-0.0453	-0.1733	0.1293	-17.7893
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2772	-0.7471	-0.0478	-0.1828	0.1401	-26.7762
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2400	8.2941	0.1521	0.6072	-0.1560	17.4180
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2400	8.2941	0.1521	0.6072	-0.1560	17.4180
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1000	2.2528	0.0200	0.0882	0.0789	-13.4650
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9694	2.6086	0.0181	0.0811	0.0870	-20.2051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2497	5.7528	0.1067	0.4236	-0.1408	17.5264
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2497	5.7528	0.1067	0.4236	-0.1408	17.5264
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0904	-0.2885	-0.0254	-0.0954	0.0942	-13.3566
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9598	0.0673	-0.0273	-0.1025	0.1022	-20.0967
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6716	5.8295	0.1172	0.4634	-0.1794	23.4112
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6716	5.8295	0.1172	0.4634	-0.1794	23.4112
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4485	-2.2255	-0.0590	-0.2285	0.1339	-17.7660
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2743	-1.7512	-0.0615	-0.2380	0.1446	-26.7529

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	12.0373
Shear X	-2.7933
Shear Z	0.2344
Moment X	0.9317
Moment Y (Twist)	0.3148
Moment Z	45.6947

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	8.2941
Shear X	-1.6716
Shear Z	0.1521
Moment X	0.6072
Moment Y (Twist)	0.1839
Moment Z	26.7762

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	-0.0146	2.8493	0.0000	0.0000	0.0000	0.2044
ULS: 2. D + L	-0.0146	2.8493	0.0000	0.0000	0.0000	0.2044
ULS: 3. D + (S or Lr or R)	-0.0403	6.8351	0.0000	-0.0000	0.0000	0.5283
ULS: 3. D + (S or Lr or R)	-0.0146	2.8493	0.0000	0.0000	0.0000	0.2044
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0339	5.8387	0.0000	-0.0000	0.0000	0.4473
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0146	2.8493	0.0000	0.0000	0.0000	0.2044
ULS: 5b. D + 0.7E	-0.0146	2.8493	0.0000	0.0000	0.0000	0.2044

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0339	5.8387	0.0000	-0.0000	0.0000	0.4473
ULS: 8. 0.6D + 0.7E	-0.0088	1.7096	0.0000	0.0000	0.0000	0.1226
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9263	7.9151	0.0000	-0.0000	0.0000	26.7502
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.9263	7.9151	0.0000	-0.0000	0.0000	26.7502
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6445	-1.5333	0.0000	-0.0000	0.0000	-19.8624
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4011	-0.9404	0.0000	0.0000	0.0000	-29.4785
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4677	9.6380	0.0000	-0.0000	0.0000	20.3567
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4677	9.6380	0.0000	-0.0000	0.0000	20.3567
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2104	2.5517	0.0000	-0.0000	0.0000	-14.6028
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0278	2.9964	0.0000	-0.0000	0.0000	-21.8149
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4484	6.6486	0.0000	-0.0000	0.0000	20.1138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4484	6.6486	0.0000	-0.0000	0.0000	20.1138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2297	-0.4376	0.0000	-0.0000	0.0000	-14.8457
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0471	0.0071	0.0000	0.0000	0.0000	-22.0578
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9205	6.7753	0.0000	-0.0000	0.0000	26.6685
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.9205	6.7753	0.0000	-0.0000	0.0000	26.6685
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6503	-2.6730	0.0000	-0.0000	0.0000	-19.9441
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4069	-2.0801	0.0000	0.0000	0.0000	-29.5603

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	14.0163
Shear X	-3.2134
Shear Z	0.0000
Moment X	0.0002
Moment Y (Twist)	0.0003
Moment Z	50.2765

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	9.6380
Shear X	-1.9263
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	29.5603

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	0.0073	2.5103	-0.0342	-0.1382	0.0113	-0.0583
ULS: 2. D + L	0.0073	2.5103	-0.0342	-0.1382	0.0113	-0.0583
ULS: 3. D + (S or Lr or R)	0.0202	5.8987	-0.0947	-0.3830	0.0317	-0.2028
ULS: 3. D + (S or Lr or R)	0.0073	2.5103	-0.0342	-0.1382	0.0113	-0.0583
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0169	5.0516	-0.0796	-0.3218	0.0266	-0.1666
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0073	2.5103	-0.0342	-0.1382	0.0113	-0.0583
ULS: 5b. D + 0.7E	0.0073	2.5103	-0.0342	-0.1382	0.0113	-0.0583
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0169	5.0516	-0.0796	-0.3218	0.0266	-0.1666
ULS: 8. 0.6D + 0.7E	0.0044	1.5062	-0.0205	-0.0829	0.0068	-0.0350
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6687	6.8336	-0.1309	-0.5187	0.1840	23.3880
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6687	6.8336	-0.1309	-0.5187	0.1840	23.3880
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4514	-1.2214	0.0453	0.1732	-0.1293	-17.7893
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2772	-0.7471	0.0478	0.1828	-0.1401	-26.7762
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2400	8.2941	-0.1521	-0.6072	0.1561	17.4180
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2400	8.2941	-0.1521	-0.6072	0.1561	17.4180
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1000	2.2528	-0.0200	-0.0883	-0.0788	-13.4649
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9694	2.6085	-0.0181	-0.0811	-0.0869	-20.2051

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	-1.2497	5.7528	-0.1067	-0.4236	0.1408	17.5264
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2497	5.7528	-0.1067	-0.4236	0.1408	17.5264
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0904	-0.2885	0.0254	0.0954	-0.0942	-13.3565
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9597	0.0673	0.0273	0.1025	-0.1022	-20.0967
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6716	5.8295	-0.1172	-0.4634	0.1794	23.4113
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6716	5.8295	-0.1172	-0.4634	0.1794	23.4113
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4485	-2.2255	0.0590	0.2285	-0.1338	-17.7660
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2743	-1.7512	0.0615	0.2380	-0.1446	-26.7529

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	12.0373
Shear X	-2.7933
Shear Z	-0.2344
Moment X	-0.9317
Moment Y (Twist)	0.3151
Moment Z	45.6953

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	8.2941
Shear X	-1.6716
Shear Z	-0.1521
Moment X	-0.6072
Moment Y (Twist)	0.1840
Moment Z	26.7762

Project Details

Design Code: AISC 360-16 LRFD
Provision: LRFD
Country: United States

User Name: sales@mtsolar.us
Project Name: MulberryElementarySchool-NCGP2023-JB-RevB
Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions								
ID	Name	d (in)	t_w (in)					
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	HSS5x3x16	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_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (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	92.39	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	L S T	L S C	L D
1	9	27.49	27.49	13.09	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
4	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.70	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.16,1.18,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1
8	19	1.33	1.33	2.05	2.10,2.10,2.10,2.10,2.10,2.10,2.10,2.10,2.36,2.10,2.10,2.10,2.22,2.10,2.10,2.10,2.11,2.10,2.10,2.10,2.25,2.10,2.10,2.10,1.72	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.72	300	200	1
11	19	1.33	1.33	2.05	2.11,2.11,2.11,2.11,2.11,2.11,2.12,2.12,2.14,2.20,2.12,2.12,2.13,2.15,2.11,2.11,2.10,1.87,2.12,2.12,2.15,2.26,2.12,2.12,2.12,2.14	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.06,1.09,1.08,1.08,1.07,1.09,1.08,1.08,1.09,1.07,1.08,1.08,1.06,1.09,1.07,1.07,1.07,1.09	300	200	1
14	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.19,1.08,1.08,1.08,1.72,1.08,1.08,1.08,1.09,1.08,1.08,1.14,1.08,1.08,1.08,2.96	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	27.49	27.49	13.09	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
104	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.72	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1
106	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1

						0	0	
108	19	1.33	1.33	2.05	2.26,2.26,2.26,2.26,2.26,2.26,2.26,2.26,2.27,2.12,2.26,2.26,2.27,1.66,2.26,2.26,2.26,2.29,2.26,2.26,2.27,2.14,2.26,2.26,2.27,1.26	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.68,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.72	300	200	1
111	19	1.33	1.33	2.05	2.28,2.28,2.28,2.28,2.28,2.28,2.36,2.36,2.10,2.09,2.33,2.33,2.14,2.11,2.32,2.32,2.23,1.67,2.35,2.35,2.07,2.09,2.30,2.30,2.15,2.12	300	200	1
112	5	1.30	1.30	2.00	-	300	200	1
113	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.05,1.03,1.03,1.03,1.03,1.03,1.02,1.02,1.03,1.03,1.06,1.03,1.03,1.03,1.03	300	200	1
114	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.02,1.02,1.03,1.19,1.02,1.02,1.03,2.96,1.03,1.03,1.02,1.03,1.02,1.03,1.13,1.02,1.02,1.03,1.04	300	200	1
115	19	8.42	8.42	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.16,1.16,1.14,1.13,1.16,1.16,1.15,1.14,1.16,1.16,1.19,1.26,1.16,1.16,1.13,1.13,1.16,1.16,1.15,1.14	300	200	1
116	19	8.42	8.42	12.95	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.18,1.18,1.18,1.13,1.18,1.18,1.18,1.17,1.18,1.18,1.15,1.18,1.18,1.18,1.11	300	200	1
201	9	27.49	27.49	13.09	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.16,1.18,1.18,1.18,1.18,1.18	300	200	1
204	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.72	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1
206	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	300	200	1
208	19	9.97	9.97	4.75	2.33,2.33	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.70	300	200	1
211	19	9.97	9.97	4.75	2.33,2.33	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.06,1.09,1.08,1.08,1.07,1.09,1.08,1.08,1.09,1.07,1.08,1.08,1.06,1.09,1.07,1.07,1.07,1.09	300	200	1
214	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.19,1.08,1.08,1.08,1.72,1.08,1.08,1.08,1.09,1.08,1.08,1.14,1.08,1.08,1.08,2.96	300	200	1
215	19	8.42	8.42	12.95	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.15,1.16,1.14,1.14,1.14,1.15,1.14,1.14,1.14,1.15,1.14,1.14,1.15,1.16,1.14,1.14,1.14,1.15	300	200	1
216	19	8.42	8.42	12.95	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.16,1.14,1.14,1.14,1.19,1.14,1.14,1.14,1.14,1.14,1.14,1.15,1.14,1.14,1.14,1.27	300	200	1

Member Design Capacity

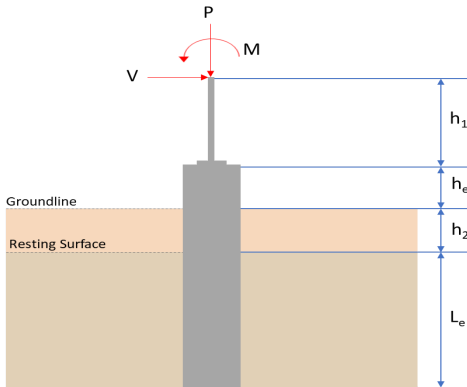
Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	150.31	83.29	83.29	113.39	113.39
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	126.01	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	126.01	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.29	6.12	40.24	43.62
14	133.20	104.94	24.75	6.12	40.24	43.62
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62
101	377.97	150.31	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	126.01	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	126.01	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	104.94	23.37	6.12	40.24	43.62
114	133.20	104.94	23.37	6.12	40.24	43.62
115	133.20	46.28	12.24	6.12	40.24	43.62
116	133.20	46.28	12.02	6.12	40.24	43.62
201	377.97	150.31	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	104.94	24.29	6.12	40.24	43.62
214	133.20	104.94	24.75	6.12	40.24	43.62
215	133.20	46.28	12.35	6.12	40.24	43.62
216	133.20	46.28	12.35	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	0.080	0.549	0.026	0.025	0.002	0.553	#16	0.561	Not Required	Pass
2	0.003	0.412	0.125	0.090	0.023	0.537	#13	0.035	Not Required	Pass
3	0.006	0.660	0.030	0.066	0.005	0.668	#13	0.045	Not Required	Pass
4	0.005	0.622	0.110	0.063	0.024	0.689	#21	0.080	Not Required	Pass
5	0.005	0.409	0.106	0.066	0.027	0.420	#13	0.074	Not Required	Pass
6	0.007	0.761	0.049	0.077	0.006	0.792	#13	0.045	Not Required	Pass
7	0.007	0.472	0.147	0.076	0.037	0.493	#13	0.074	Not Required	Pass
8	0.001	0.068	0.155	0.051	0.012	0.184	#24	0.095	Not Required	Pass
9	0.013	0.086	0.052	0.002	0.002	0.135	#21	0.204	Not Required	Pass
10	0.007	0.705	0.138	0.071	0.029	0.765	#21	0.080	Not Required	Pass
11	0.002	0.067	0.158	0.055	0.012	0.190	#21	0.095	Not Required	Pass
12	0.003	0.508	0.140	0.105	0.025	0.647	#13	0.053	Not Required	Pass
13	0.005	0.282	0.330	0.068	0.015	0.541	#21	0.286	Not Required	Pass
14	0.005	0.266	0.326	0.063	0.015	0.518	#21	0.190	Not Required	Pass
15	0.000	0.103	0.131	0.036	0.008	0.230	#21	Not Required	Not Required	Pass
16	0.000	0.098	0.131	0.034	0.008	0.227	#21	Not Required	Not Required	Pass
101	0.093	0.604	0.000	0.028	0.000	0.605	#16	0.561	Not Required	Pass
102	0.003	0.536	0.154	0.114	0.027	0.692	#13	0.035	Not Required	Pass
103	0.007	0.822	0.036	0.082	0.002	0.841	#13	0.045	Not Required	Pass
104	0.007	0.787	0.142	0.079	0.029	0.862	#21	0.080	Not Required	Pass
105	0.007	0.510	0.148	0.082	0.038	0.533	#13	0.074	Not Required	Pass
106	0.007	0.822	0.036	0.082	0.002	0.841	#13	0.045	Not Required	Pass
107	0.007	0.510	0.148	0.082	0.038	0.533	#13	0.074	Not Required	Pass
108	0.001	0.081	0.160	0.055	0.012	0.239	#21	0.095	Not Required	Pass
109	0.015	0.082	0.039	0.001	0.000	0.125	#21	0.204	Not Required	Pass
110	0.007	0.787	0.142	0.079	0.029	0.862	#21	0.080	Not Required	Pass
111	0.002	0.070	0.163	0.057	0.012	0.234	#21	0.095	Not Required	Pass
112	0.003	0.536	0.154	0.114	0.027	0.692	#13	0.035	Not Required	Pass
113	0.005	0.304	0.335	0.070	0.015	0.616	#21	0.286	Not Required	Pass
114	0.006	0.321	0.333	0.068	0.015	0.623	#21	0.286	Not Required	Pass
115	0.005	0.511	0.175	0.057	0.012	0.666	#21	0.601	Not Required	Pass
116	0.003	0.469	0.177	0.055	0.012	0.636	#21	0.601	Not Required	Pass
201	0.080	0.549	0.026	0.025	0.002	0.553	#16	0.561	Not Required	Pass
202	0.003	0.508	0.140	0.105	0.025	0.648	#13	0.053	Not Required	Pass
203	0.007	0.761	0.049	0.077	0.006	0.792	#13	0.045	Not Required	Pass
204	0.007	0.705	0.138	0.071	0.029	0.765	#21	0.080	Not Required	Pass
205	0.007	0.472	0.147	0.076	0.037	0.493	#13	0.074	Not Required	Pass
206	0.006	0.660	0.030	0.066	0.005	0.668	#13	0.045	Not Required	Pass
207	0.005	0.409	0.106	0.066	0.027	0.420	#13	0.074	Not Required	Pass
208	0.000	0.098	0.131	0.034	0.008	0.227	#21	Not Required	Not Required	Pass
209	0.013	0.086	0.052	0.002	0.002	0.135	#21	0.204	Not Required	Pass
210	0.005	0.622	0.110	0.063	0.024	0.689	#21	0.080	Not Required	Pass
211	0.000	0.103	0.131	0.036	0.008	0.230	#21	Not Required	Not Required	Pass
212	0.003	0.412	0.125	0.090	0.023	0.537	#13	0.035	Not Required	Pass
213	0.005	0.282	0.330	0.068	0.015	0.540	#21	0.190	Not Required	Pass
214	0.005	0.266	0.326	0.063	0.015	0.518	#21	0.286	Not Required	Pass
215	0.005	0.521	0.176	0.055	0.012	0.668	#21	0.601	Not Required	Pass
216	0.003	0.480	0.175	0.051	0.012	0.644	#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 = 6.5 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>8.294</td><td>12.037</td></tr><tr><td>Vx (kip)</td><td>-1.672</td><td>-2.793</td></tr><tr><td>Vz (kip)</td><td>0.152</td><td>0.234</td></tr><tr><td>Mx (kipft)</td><td>0.607</td><td>0.932</td></tr><tr><td>Mz (kipft)</td><td>26.776</td><td>45.695</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)	8.294	12.037	Vx (kip)	-1.672	-2.793	Vz (kip)	0.152	0.234	Mx (kipft)	0.607	0.932	Mz (kipft)	26.776	45.695	
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	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, $H_o = \frac{V_x}{1.57\text{ }D}$ $H_o = \frac{(-1.672\text{ kip})}{1.57 \times (48\text{ in})}$ $H_o = -0.26624\text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x\text{ }H)}{1.57\text{ }D}$																											

	$M_o = \frac{(26.776 \text{ kipft}) + ((-1.672 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 4.2637 \text{ kipft/ft}$ 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: $L_{e,x} = 6.2283 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.152 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.024204 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.607 \text{ kipft}) + ((0.152 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.096656 \text{ kipft/ft}$ 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: $L_{e,z} = 2.2211 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.2283 \text{ ft}), (2.2211 \text{ ft})]$ $L_{e,req} = 6.228 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.5 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.228 \text{ ft})}{(6.5 \text{ ft})}$ $\text{Ratio} = 0.95815$	Status: PASS Ratio: 0.960
	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 $q = \frac{P_o}{A}$ $q = \frac{(8.294 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.518375 \text{ kip/ft}^2$	

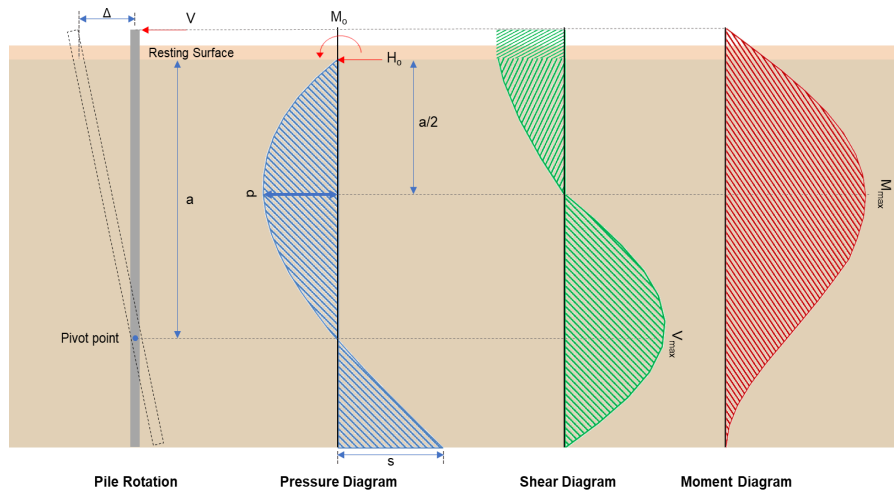
	$q = 0.01031 \text{ kip/ft}$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.51837 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.25919$	Status: PASS Ratio: 0.260
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.625$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.26624 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 4.2637 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (4.2637 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.2637 \text{ kipft/ft})) + (4 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))}$ $a = 4.4487 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (4.2637 \text{ kipft/ft})) + (3 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^3 [(3 \times (4.2637 \text{ kipft/ft})) + (2 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$ $p = 0.26776 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (4.2637 \text{ kipft/ft})) + ((-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$ $s = 0.96523 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4487 \text{ ft})}{2}$ $p_a = 0.33365 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.26776 \text{ kip/ft}^2)}{(0.33365 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.80252$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.800

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.96523 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98998$	Status: PASS Ratio: 0.990
	Considering z-direction: $H_o = 0.024204 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.096656 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.096656 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.024204 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.096656 \text{ kipft/ft})) + (4 \times (0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))}$ $a = 4.6152 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.096656 \text{ kipft/ft})) + (3 \times (0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^3 \times [(3 \times (0.096656 \text{ kipft/ft})) + (2 \times (0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$ $p = 0.021644 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.096656 \text{ kipft/ft})) + ((0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$ $s = 0.049795 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.6152 \text{ ft})}{2}$ $p_a = 0.34614 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.021644 \text{ kip/ft}^2)}{(0.34614 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.062529$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.060

$$Ratio = \frac{(0.049795 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.051071$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.793 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44475 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(45.695 \text{ kipft}) + ((-2.793 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.2763 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.2763 \text{ kipft/ft})}{(-0.44475 \text{ kip/ft})}$$

$$E = 16.361 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2763 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.44475 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.2763 \text{ kipft/ft})) + (4 \times (-0.44475 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4468 \text{ ft}$$

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

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

$$V_{max} = ((-0.44475 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4468 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4468 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.1013 \text{ kip}$$

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

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

$$M_{max} = ((-0.44475 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(16.361 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4468 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4468 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4468 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.525 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.234 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.037261 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.932 \text{ kipft}) + ((0.234 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14841 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14841 \text{ kipft/ft})}{(0.037261 \text{ kip/ft})}$$

$$E = 3.9829 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14841 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.037261 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.14841 \text{ kipft/ft})) + (4 \times (0.037261 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6156 \text{ ft}$$

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

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

$$V_{max} = ((0.037261 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6156 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6156 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.26061 \text{ kip}$$

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

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

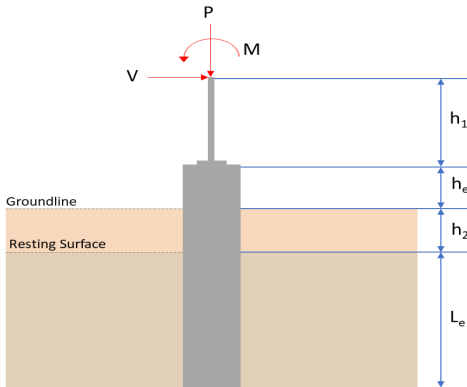
$$M_{max} = ((0.037261 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(3.9829 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6156 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6156 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6156 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.76033 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(12.037 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.196 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.196 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(12.037 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0044995$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.037 \text{ kip} \rightarrow 12037 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12037 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 120.09 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (120.09 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 120.09 \text{ kip}$	

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, $A_v = \frac{\pi d_{tes}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ 22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b) $V_{s,b} = \frac{2 A_v f_{ysk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.09 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.14 \text{ kip}$ Considering x-direction: $V_{max} = 9.1013 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(9.1013 \text{ kip})}{(111.14 \text{ kip})}$ $Ratio = 0.081891$ Considering z-direction: $V_{max} = 0.26061 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.26061 \text{ kip})}{(111.14 \text{ kip})}$ $Ratio = 0.0023449$	Status: PASS Ratio: 0.080 Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

14.5.2.1b	$\lambda = 1$ - 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, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 28.525 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(28.525 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.11428$	Status: PASS Ratio: 0.110
	Considering z-direction: $M_{max} = 0.76033 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.76033 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0030462$	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 = 6.5 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>8.294</td><td>12.037</td></tr><tr><td>Vx (kip)</td><td>-1.672</td><td>-2.793</td></tr><tr><td>Vz (kip)</td><td>-0.152</td><td>-0.234</td></tr><tr><td>Mx (kipft)</td><td>-0.607</td><td>-0.932</td></tr><tr><td>Mz (kipft)</td><td>26.776</td><td>45.695</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)	8.294	12.037	Vx (kip)	-1.672	-2.793	Vz (kip)	-0.152	-0.234	Mx (kipft)	-0.607	-0.932	Mz (kipft)	26.776	45.695	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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Mx (kipft)	-0.607	-0.932																										
Mz (kipft)	26.776	45.695																										
	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, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.672 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26624 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(26.776 \text{ kipft}) + ((-1.672 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 4.2637 \text{ kipft/ft}$ 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: $L_{e,x} = 6.2283 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.152 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.024204 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.607 \text{ kipft}) + ((-0.152 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.096656 \text{ kipft/ft}$ 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: $L_{e,z} = 1.7341 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.2283 \text{ ft}), (1.7341 \text{ ft})]$ $L_{e,req} = 6.228 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.5 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.228 \text{ ft})}{(6.5 \text{ ft})}$ $\text{Ratio} = 0.95815$	Status: PASS Ratio: 0.960
	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 $q = \frac{P_o}{A}$ $q = \frac{(8.294 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.518375 \text{ kip/ft}^2$	

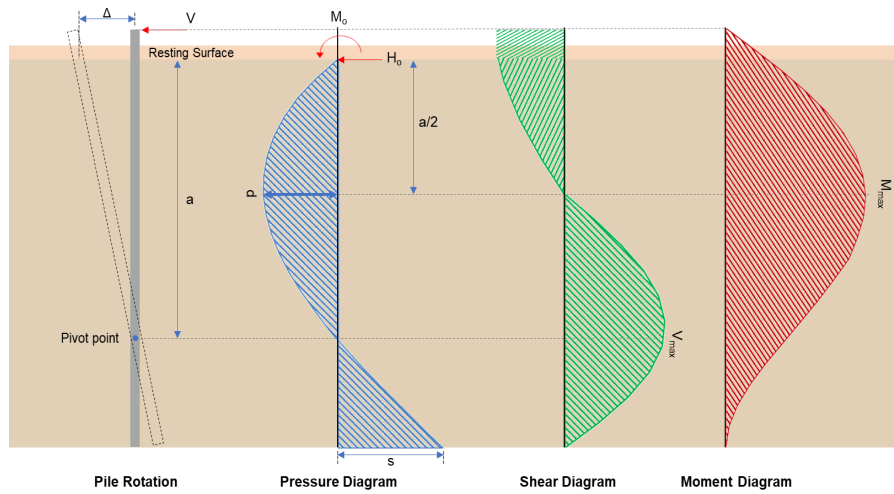
	$q = 0.01031 \text{ kip/ft}$	
	Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.51837 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.25919$	Status: PASS Ratio: 0.260
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.625$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.26624 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 4.2637 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (4.2637 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.2637 \text{ kipft/ft})) + (4 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))}$ $a = 4.4487 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (4.2637 \text{ kipft/ft})) + (3 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^3 [(3 \times (4.2637 \text{ kipft/ft})) + (2 \times (-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$ $p = 0.26776 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (4.2637 \text{ kipft/ft})) + ((-0.26624 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$ $s = 0.96523 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4487 \text{ ft})}{2}$ $p_a = 0.33365 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.26776 \text{ kip/ft}^2)}{(0.33365 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.80252$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.800

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.96523 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98998$	Status: PASS Ratio: 0.990
	Considering z-direction: $H_o = -0.024204 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.096656 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.096656 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.024204 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.096656 \text{ kipft/ft})) + (4 \times (-0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))}$ $a = 4.6152 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.096656 \text{ kipft/ft})) + (3 \times (-0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^3 \times [(3 \times (0.096656 \text{ kipft/ft})) + (2 \times (-0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$ $p = -0.0052393 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.096656 \text{ kipft/ft})) + ((-0.024204 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$ $s = 0.0051106 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.6152 \text{ ft})}{2}$ $p_a = 0.34614 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0052393 \text{ kip/ft}^2)}{(0.34614 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.015136$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.020

$$Ratio = \frac{(0.0051106 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.0052417$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.793 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44475 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(45.695 \text{ kipft}) + ((-2.793 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.2763 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.2763 \text{ kipft/ft})}{(-0.44475 \text{ kip/ft})}$$

$$E = 16.361 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2763 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.44475 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.2763 \text{ kipft/ft})) + (4 \times (-0.44475 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4468 \text{ ft}$$

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

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

$$V_{max} = ((-0.44475 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4468 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4468 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.1013 \text{ kip}$$

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

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

$$M_{max} = ((-0.44475 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(16.361 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4468 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4468 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (16.361 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4468 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.525 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.234 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.037261 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.932 \text{ kipft}) + ((-0.234 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14841 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14841 \text{ kipft/ft})}{(-0.037261 \text{ kip/ft})}$$

$$E = 3.9829 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14841 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.037261 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.14841 \text{ kipft/ft})) + (4 \times (-0.037261 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6156 \text{ ft}$$

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

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

$$V_{max} = ((-0.037261 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6156 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6156 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.26061 \text{ kip}$$

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

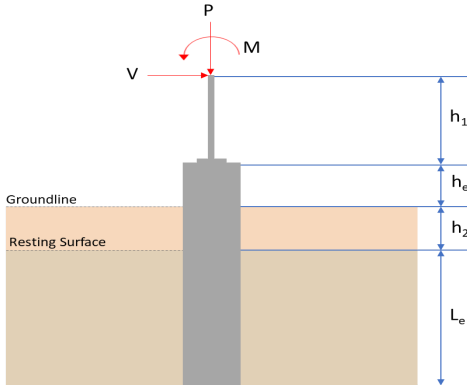
$$M_{max} = (H_o b L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$$

$$M_{max} = ((-0.037261 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(3.9829 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6156 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6156 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.9829 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6156 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.76033 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(12.037 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.196 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.196 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(12.037 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0044995$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.037 \text{ kip} \rightarrow 12037 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12037 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 120.09 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (120.09 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 120.09 \text{ kip}$	

14.5.2.1b	$\lambda = 1$ - 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, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 28.525 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(28.525 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.11428$	Status: PASS Ratio: 0.110
	Considering z-direction: $M_{max} = 0.76033 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.76033 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0030462$	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 = 6.75 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><thead><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr></thead><tbody><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></tbody></table> Tabulation of Loads <table><thead><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr></thead><tbody><tr><td>P (kip)</td><td>9.638</td><td>14.016</td></tr><tr><td>Vx (kip)</td><td>-1.926</td><td>-3.213</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>29.560</td><td>50.276</td></tr></tbody></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)	9.638	14.016	Vx (kip)	-1.926	-3.213	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	29.560	50.276	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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Mx (kipft)	0.000	0.000																										
Mz (kipft)	29.560	50.276																										
	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, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.926 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.30669 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(29.56 \text{ kipft}) + ((-1.926 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 4.707 \text{ kipft/ft}$ 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: $L_{e,x} = 6.3762 \text{ ft}$ - Required depth in x-direction, Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(6.3762 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.376 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.75 \text{ ft}$ <i>Ratio</i> - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(6.376 \text{ ft})}{(6.75 \text{ ft})}$ $Ratio = 0.94459$	Status: PASS Ratio: 0.940
	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 $q = \frac{P_v}{A}$ $q = \frac{(9.638 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.60237 \text{ kip/ft}^2$ Check bearing capacity ratio: <i>Ratio</i> - Capacity $Ratio = \frac{q}{q_o}$ $Ratio = \frac{(0.60237 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.30119$	Status: PASS Ratio: 0.300
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.30669$ kip/ft - Lateral force per length of pile,

$M_o = 4.707$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.707 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.30669 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.707 \text{ kipft/ft})) + (4 \times (-0.30669 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6275 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.707 \text{ kipft/ft})) + (3 \times (-0.30669 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^3 \times [(3 \times (4.707 \text{ kipft/ft})) + (2 \times (-0.30669 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.26257 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.707 \text{ kipft/ft})) + ((-0.30669 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.96709 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6275 \text{ ft})}{2}$$

$$p_a = 0.34706 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26257 \text{ kip/ft}^2)}{(0.34706 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75654$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

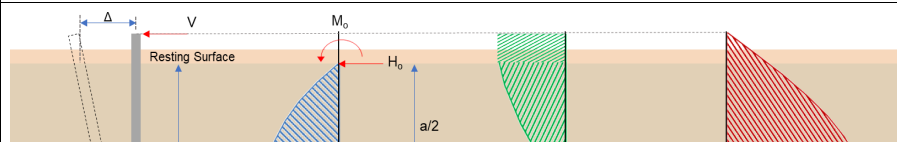
$$\text{Ratio} = \frac{s}{p_s}$$

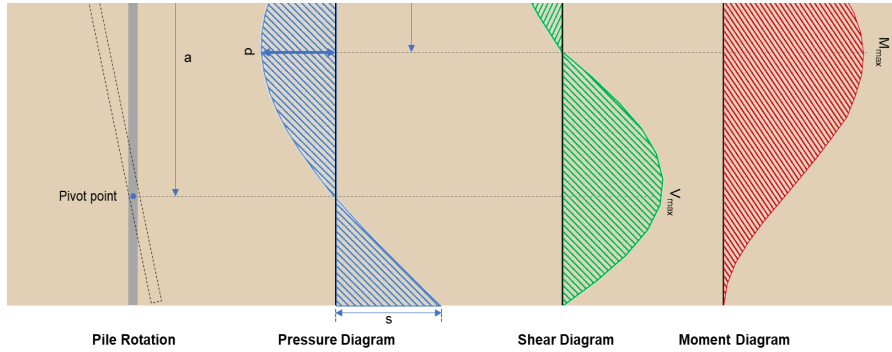
$$\text{Ratio} = \frac{(0.96709 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95515$$

Status: **PASS**
Ratio: **0.760**

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.213 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.51162 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(50.276 \text{ kipft}) + ((-3.213 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.0057 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.0057 \text{ kipft/ft})}{(-0.51162 \text{ kip/ft})}$$

$$E = 15.648 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.0057 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.51162 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.0057 \text{ kipft/ft})) + (4 \times (-0.51162 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6256 \text{ ft}$$

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

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

$$V_{max} = ((-0.51162 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.648 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6256 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.648 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6256 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.7482 \text{ kip}$$

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

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

$$M_{max} = ((-0.51162 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(15.648 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6256 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.648 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6256 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.648 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6256 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 31.641 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(14.016 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.13 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.13 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(14.016 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0052393$	Status: PASS Ratio: 0.010
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.016 \text{ kip} \rightarrow 14016 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14016 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 120.35 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (120.35 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 120.35 \text{ kip}$	

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 $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.35 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.31 \text{ kip}$ Considering x-direction: $V_{max} = 9.7482 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(9.7482 \text{ kip})}{(111.31 \text{ kip})}$ $Ratio = 0.087576$	Status: PASS Ratio: 0.090
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: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$	

	$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ 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: $M_{max} = 31.641 \text{ kipft}$ - Maximum moment in the x-direction, <i>Ratio</i> - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(31.641 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.12677$	
		Status: PASS Ratio: 0.130