

Your Project Calculations



Project Name: Corwin-RevB

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Corwin-RevB&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=NmX6jQlzsBs4slPGSsawCiuRibjRyUIXyBemCrX4qZ0V2f8EftLa8wA2F5GUpt

Array Specification

Product:	Beam
Unique ID:	3P-17-6TOP-HD-24-L-4Hx8W-411D
Duty Classification:	HD
Module Width:	41.00 in
Module Length:	70.00in
Number of Rows:	4
Number of Columns:	8
Total Number of Modules:	32
Desired Tilt Angle:	46
Front Edge Clearance:	6
Total Array Height at Tilt:	15.89 ft
Total Frame Length:	45.50 ft
Frame Weight:	2494 lbs
Array Dimensions N/S:	13.83 ft
Array Dimensions E/W:	47.33 ft
Rail Length:	166.00 in
Rail Spacing:	2.96 ft
Rail Check:	PASS (46% utilized)

Support Specifications

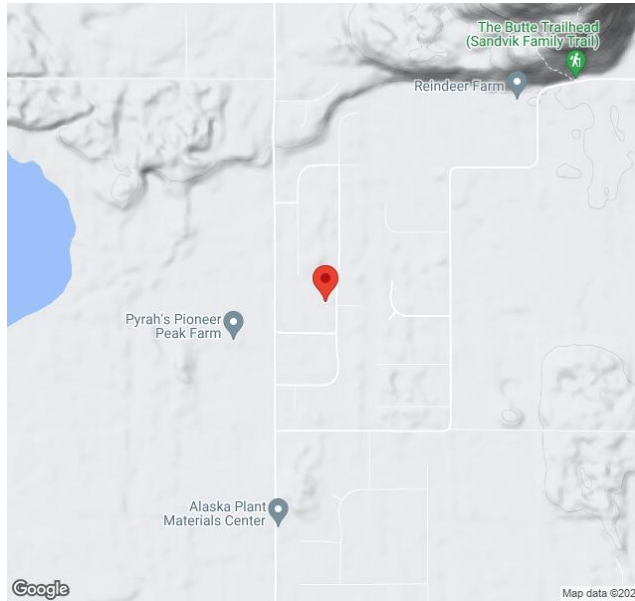
Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	10.98 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.50 ft Pile 2: 6.50 ft Pile 3: 6.50 ft
Foundation Volume:	11.556 y ³
Foundation Result:	PASSED
Mount Twist:	0.846864 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	4360 Trellis Ave, Palmer, AK 99645, USA
Wind Speed:	116 mph
Snow Load:	100 psf
Design Uplift Pressure:	0.026449 ksf
Design Downforce Pressure:	-0.026449 ksf
Design Snow Pressure:	0.026391 ksf



Design Disclaimer

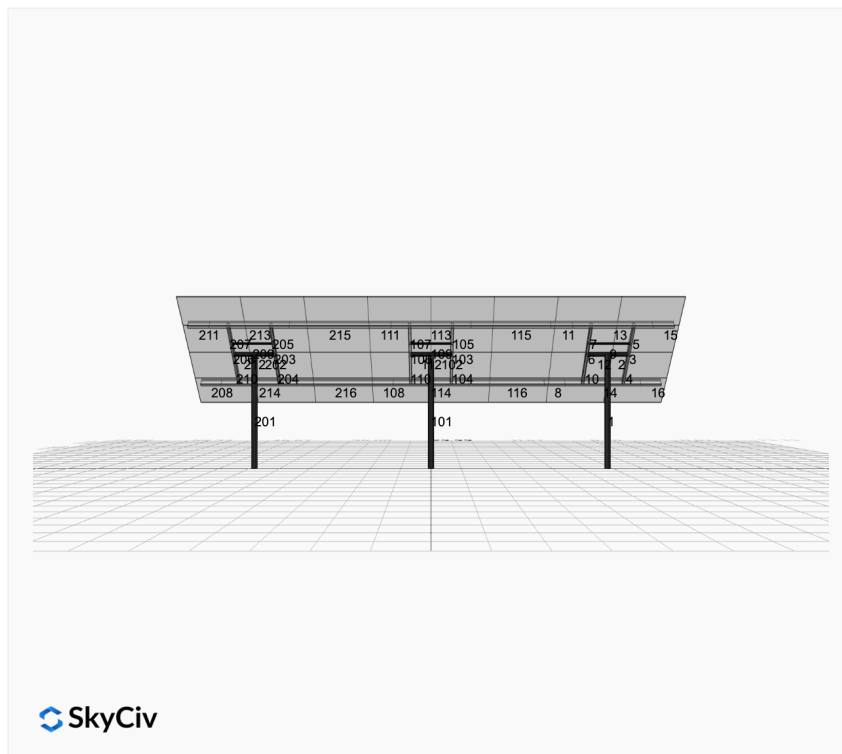
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

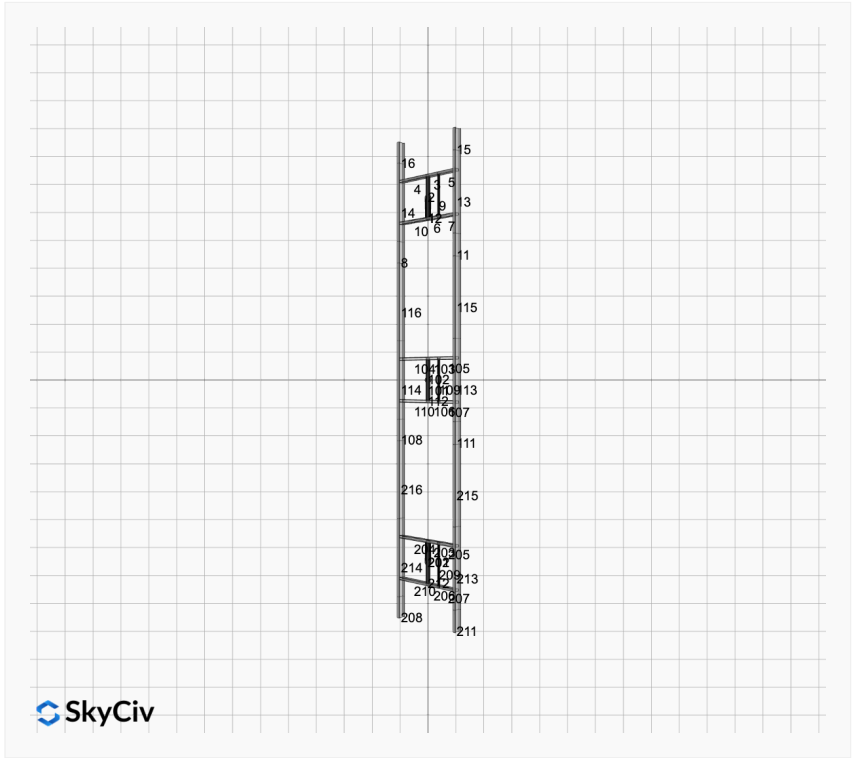
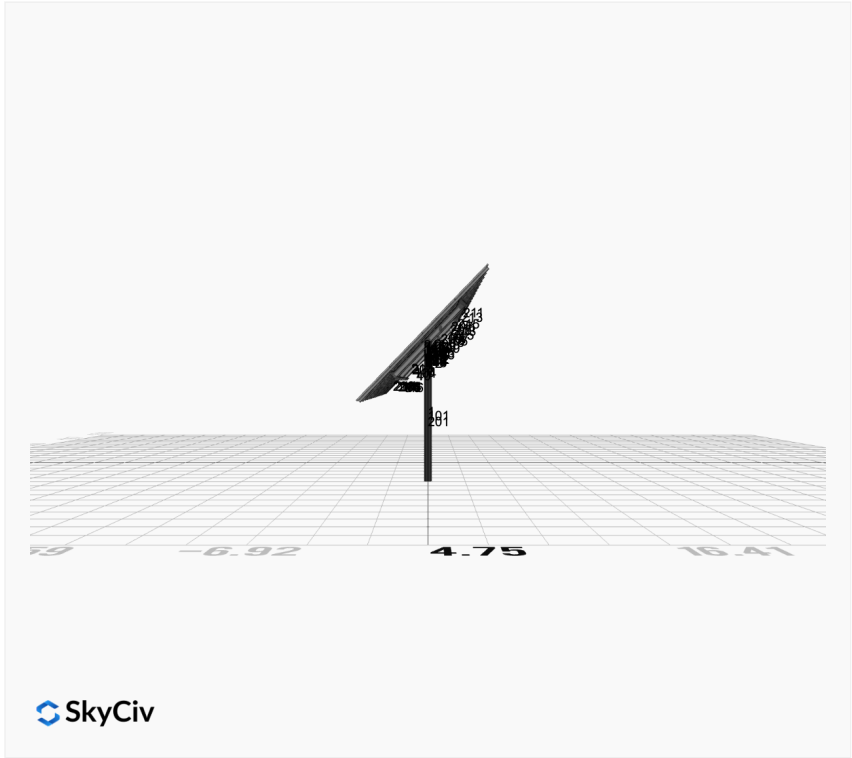
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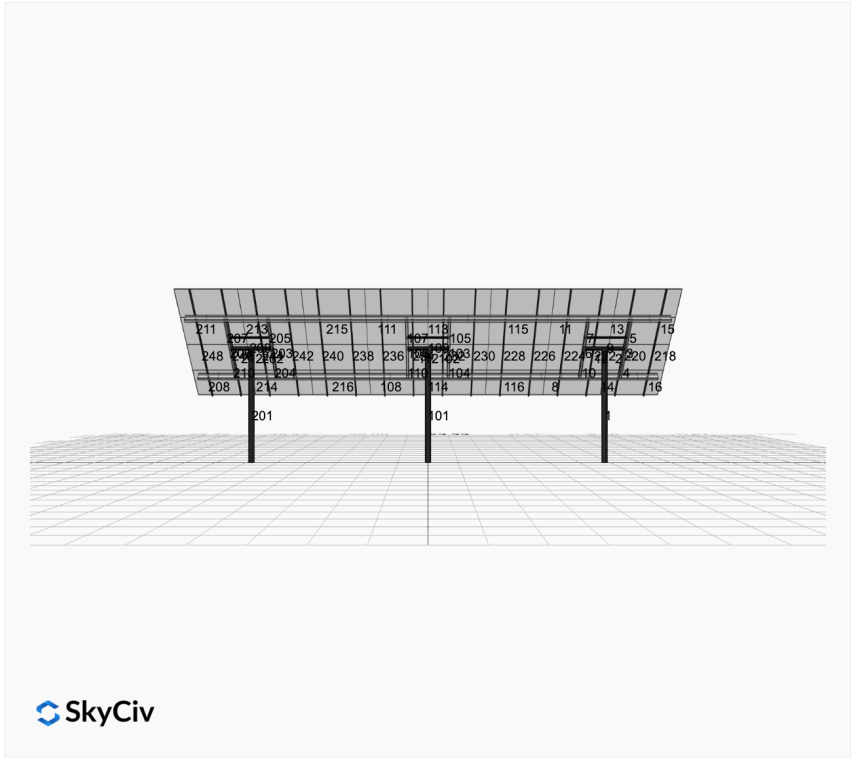
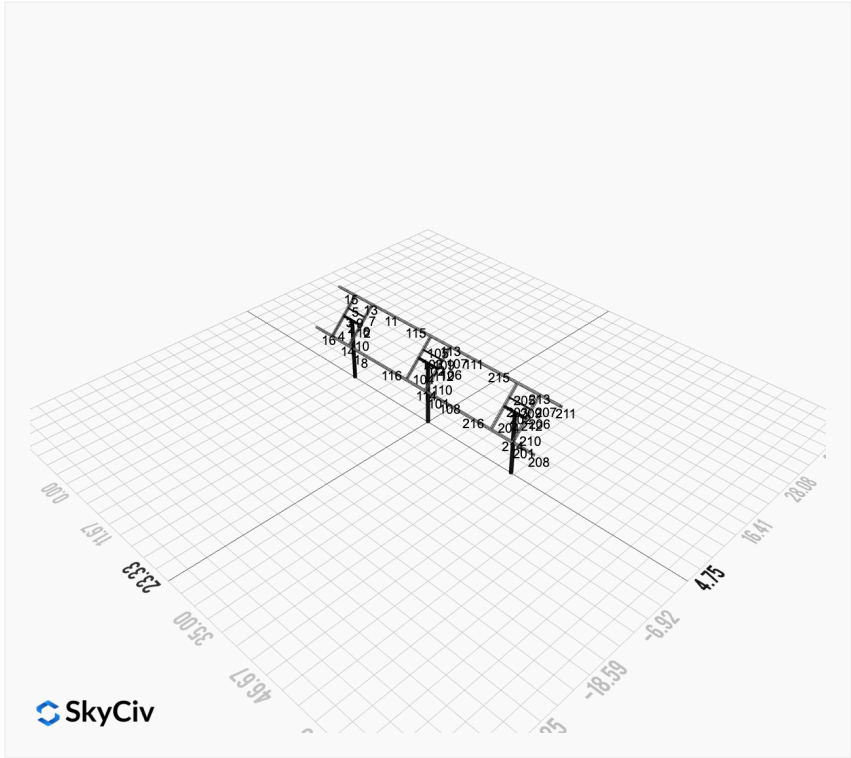
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Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent







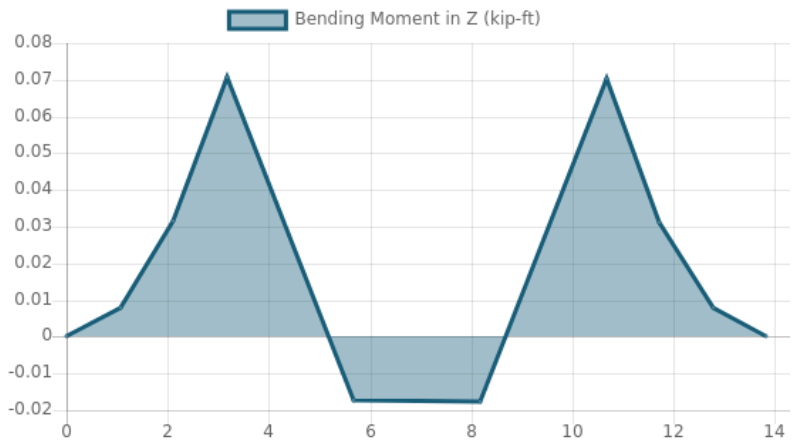
Rail Design Check

Rail Length: 13.833333333333334 ft
Additional Restraints Required: None
Tributary Width: 2.9583333333333335 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0542 kip/ft
Snow (Y): -0.0562 kip/ft
Wind uplift Case A: 0.0782 kip/ft
Wind downforce Case A: 0.0782 kip/ft
Dead (Panel load) (X): 0.0097 kip/ft
Dead (Panel load) (Y): -0.0101 kip/ft

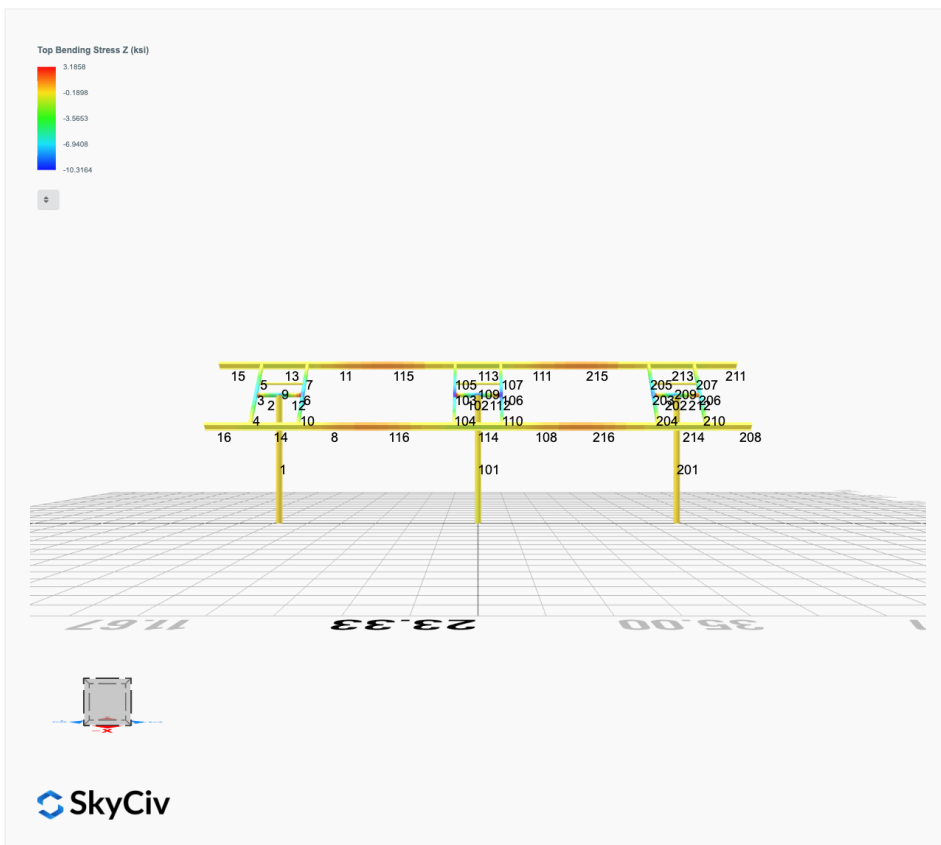
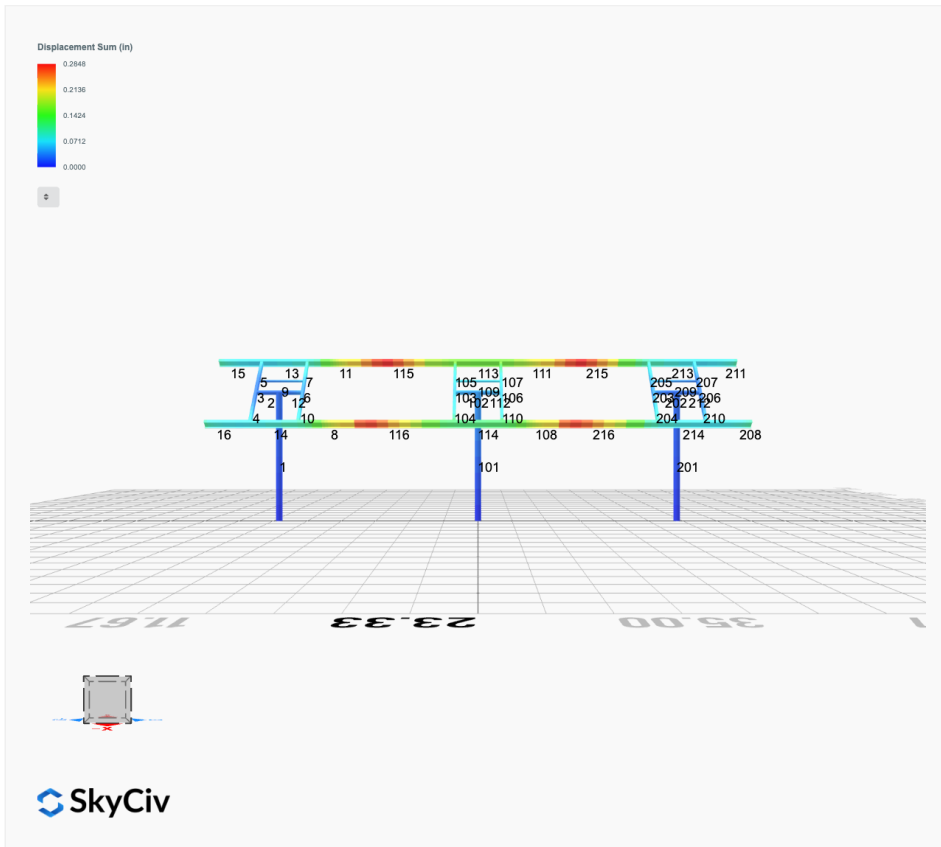


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	15.69946441	0.455	PASS
Material Yield	34.5	15.69946441	0.455	PASS
Material Strength	37	15.69946441	0.424	PASS

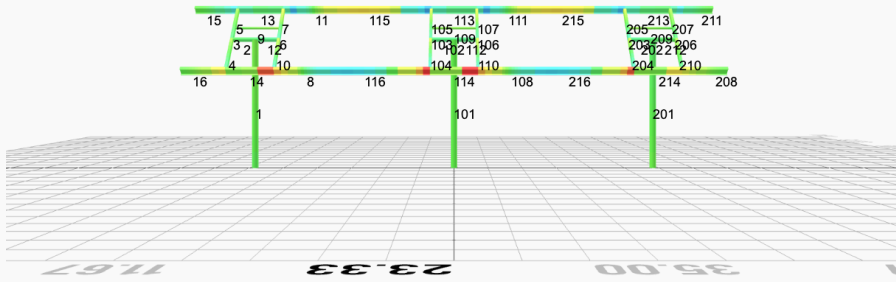
Member 1, ULS: 1. 1.4D



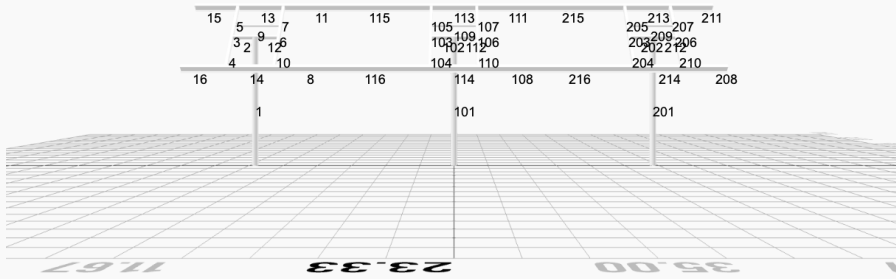
FEM Results (Envelope Worst Case for each member)

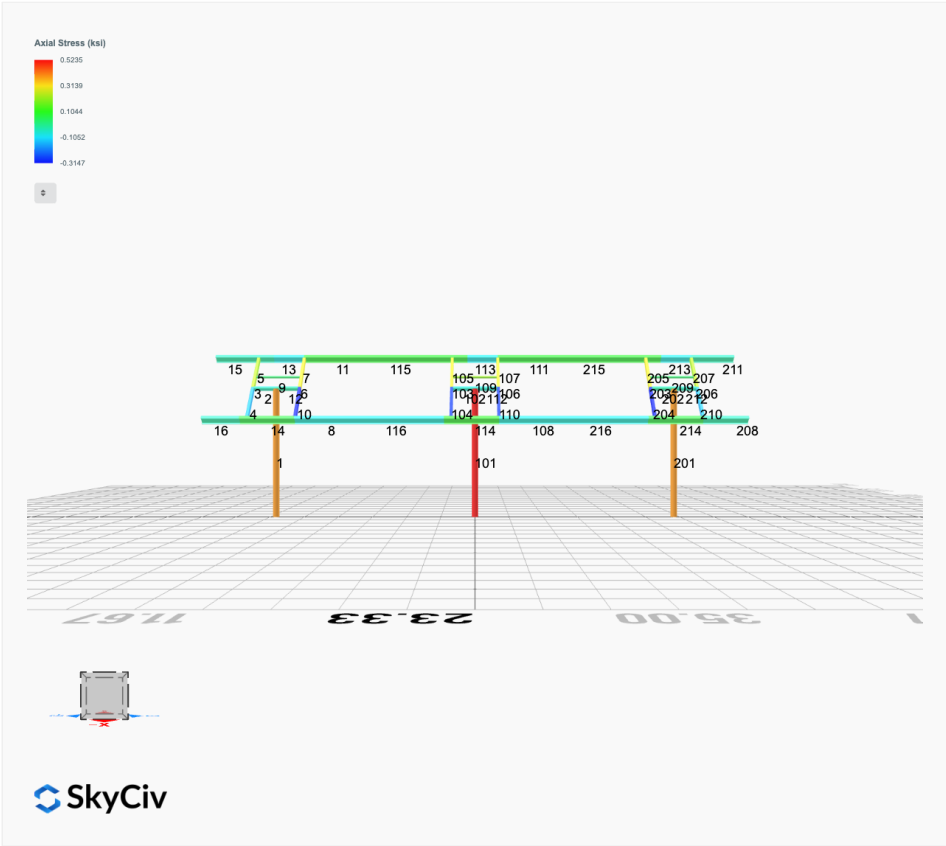


Top Bending Stress Y (ksi)



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	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 2. D + L	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 3. D + (S or Lr or R)	0.0415	5.3375	0.1424	0.4898	-0.1325	-0.3371
ULS: 3. D + (S or Lr or R)	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0338	4.4451	0.1160	0.3988	-0.1078	-0.2717
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 5b. D + 0.7E	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0338	4.4451	0.1160	0.3988	-0.1078	-0.2717
ULS: 8. 0.6D + 0.7E	0.0064	1.0608	0.0219	0.0754	-0.0203	-0.0453
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3598	4.0034	0.1695	0.5637	-0.4822	26.6918
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3755	-0.4644	-0.0925	-0.2986	0.4020	-25.7959
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7440	6.1217	0.2156	0.7273	-0.4442	19.8037
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0338	4.4451	0.1160	0.3988	-0.1078	-0.2717
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8074	2.7709	0.0192	0.0806	0.2189	-19.5620
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0338	4.4451	0.1160	0.3988	-0.1078	-0.2717
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7672	3.4446	0.1362	0.4542	-0.3701	20.0000
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7843	0.0937	-0.0602	-0.1925	0.2930	-19.3658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0107	1.7680	0.0366	0.1256	-0.0338	-0.0755
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3641	3.2962	0.1548	0.5135	-0.4687	26.7220
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0064	1.0608	0.0219	0.0754	-0.0203	-0.0453
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3712	-1.1716	-0.1071	-0.3488	0.4155	-25.7657
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0064	1.0608	0.0219	0.0754	-0.0203	-0.0453

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	9.6983
Shear X	-3.9743
Shear Z	0.3272
Moment X	1.1126
Moment Y (Twist)	0.8470
Moment Z	45.4141

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	6.1217
Shear X	-2.3755
Shear Z	0.2156
Moment X	0.7273
Moment Y (Twist)	0.4822
Moment Z	26.7220

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	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 2. D + L	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 3. D + (S or Lr or R)	-0.0831	6.4568	0.0000	-0.0000	0.0000	0.8006
ULS: 3. D + (S or Lr or R)	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0677	5.3568	0.0000	-0.0000	0.0000	0.6544
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 5b. D + 0.7E	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0677	5.3568	0.0000	-0.0000	0.0000	0.6544
ULS: 8. 0.6D + 0.7E	-0.0129	1.2341	-0.0000	0.0000	0.0000	0.1293
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7550	4.8040	-0.0000	0.0000	0.0000	30.3859
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7235	-0.6966	-0.0000	0.0000	0.0000	-28.7897
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1178	7.4172	0.0000	-0.0000	0.0000	23.2821
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0677	5.3568	0.0000	-0.0000	0.0000	0.6544
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9911	3.2917	0.0000	-0.0000	0.0000	-21.0996
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0677	5.3568	0.0000	-0.0000	0.0000	0.6544
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0716	4.1172	-0.0000	0.0000	0.0000	22.8433
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0373	-0.0083	-0.0000	0.0000	0.0000	-21.5384
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0215	2.0568	-0.0000	0.0000	0.0000	0.2156
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7464	3.9813	-0.0000	0.0000	0.0000	30.2997
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0129	1.2341	-0.0000	0.0000	0.0000	0.1293
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7321	-1.5193	-0.0000	0.0000	0.0000	-28.8759
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0129	1.2341	-0.0000	0.0000	0.0000	0.1293

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	11.7923
Shear X	-4.6016
Shear Z	0.0000
Moment X	0.0001
Moment Y (Twist)	0.0002
Moment Z	51.9707

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	7.4172
Shear X	-2.7550
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	30.3859

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	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 2. D + L	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 3. D + (S or Lr or R)	0.0415	5.3375	-0.1424	-0.4898	0.1326	-0.3371
ULS: 3. D + (S or Lr or R)	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0338	4.4451	-0.1160	-0.3988	0.1079	-0.2717
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 5b. D + 0.7E	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0338	4.4451	-0.1160	-0.3988	0.1079	-0.2717
ULS: 8. 0.6D + 0.7E	0.0064	1.0608	-0.0219	-0.0754	0.0203	-0.0453
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3598	4.0034	-0.1695	-0.5637	0.4822	26.6918
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3755	-0.4644	0.0925	0.2986	-0.4020	-25.7959
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7441	6.1217	-0.2156	-0.7274	0.4442	19.8037
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0338	4.4451	-0.1160	-0.3988	0.1079	-0.2717
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8074	2.7709	-0.0192	-0.0806	-0.2189	-19.5620
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0338	4.4451	-0.1160	-0.3988	0.1079	-0.2717

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	-1.7672	3.4446	-0.1362	-0.4542	0.3701	20.0000
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7843	0.0937	0.0602	0.1925	-0.2930	-19.3658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0107	1.7680	-0.0366	-0.1256	0.0338	-0.0755
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3641	3.2962	-0.1548	-0.5135	0.4687	26.7220
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0064	1.0608	-0.0219	-0.0754	0.0203	-0.0453
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3712	-1.1716	0.1071	0.3488	-0.4155	-25.7657
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0064	1.0608	-0.0219	-0.0754	0.0203	-0.0453

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	9.6983
Shear X	-3.9743
Shear Z	-0.3272
Moment X	-1.1128
Moment Y (Twist)	0.8469
Moment Z	45.4153

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	6.1217
Shear X	-2.3755
Shear Z	-0.2156
Moment X	-0.7274
Moment Y (Twist)	0.4822
Moment Z	26.7220

Project Details

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



Design Input Information

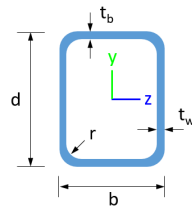
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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				
8	6in Pipe Sch 80	6.63	0.43				



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_{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
8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60

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	378.22	119.58	62.23	62.23	113.47	113.47
2	198.33	194.54	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	27.50	6.12	40.24	43.62
14	133.20	104.94	27.73	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62
101	378.22	119.58	62.23	62.23	113.47	113.47
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	23.83	6.12	40.24	43.62
114	133.20	104.94	24.06	6.12	40.24	43.62
115	133.20	93.89	24.18	6.12	40.24	43.62
116	133.20	93.89	25.33	6.12	40.24	43.62
201	378.22	119.58	62.23	62.23	113.47	113.47
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	102.39	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	102.39	32.87	6.12	40.24	43.62
212	198.33	194.54	21.95	21.95	59.50	59.50
213	133.20	104.94	27.27	6.12	40.24	43.62
214	133.20	104.94	27.50	6.12	40.24	43.62
215	133.20	93.89	24.87	6.12	40.24	43.62
216	133.20	93.89	24.87	6.12	40.24	43.62

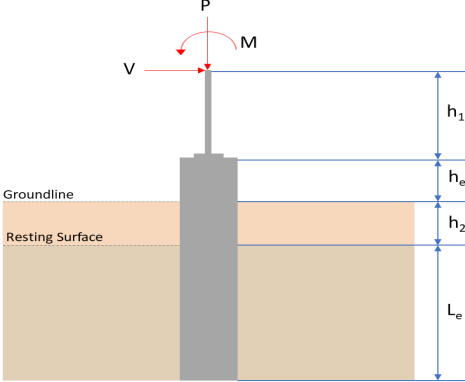
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.081	0.730	0.040	0.035	0.003	0.779	#13	0.630	Not Required	Pass
2	0.001	0.269	0.156	0.066	0.031	0.375	#21	0.081	Not Required	Pass
3	0.010	0.420	0.055	0.041	0.013	0.480	#21	0.045	Not Required	Pass
4	0.009	0.414	0.093	0.041	0.023	0.506	#21	0.080	Not Required	Pass
5	0.009	0.261	0.077	0.042	0.018	0.274	#21	0.074	Not Required	Pass
6	0.014	0.531	0.123	0.054	0.035	0.654	#21	0.045	Not Required	Pass
7	0.014	0.330	0.174	0.053	0.042	0.364	#21	0.074	Not Required	Pass
8	0.003	0.093	0.135	0.030	0.020	0.226	#21	0.095	Not Required	Pass
9	0.003	0.039	0.070	0.003	0.003	0.092	#13	0.204	Not Required	Pass
10	0.014	0.501	0.169	0.050	0.038	0.608	#21	0.080	Not Required	Pass
11	0.004	0.094	0.132	0.033	0.020	0.226	#21	0.095	Not Required	Pass
12	0.001	0.381	0.195	0.090	0.039	0.518	#13	0.053	Not Required	Pass
13	0.005	0.092	0.413	0.045	0.028	0.445	#21	0.286	Not Required	Pass
14	0.005	0.087	0.409	0.042	0.028	0.428	#23	0.190	Not Required	Pass
15	0.000	0.017	0.060	0.014	0.008	0.077	#21	Not Required	Not Required	Pass
16	0.000	0.017	0.060	0.014	0.008	0.077	#21	Not Required	Not Required	Pass
101	0.099	0.835	0.000	0.041	0.000	0.874	#13	0.630	Not Required	Pass
102	0.002	0.397	0.209	0.096	0.039	0.548	#21	0.053	Not Required	Pass
103	0.014	0.565	0.099	0.056	0.023	0.669	#21	0.045	Not Required	Pass
104	0.014	0.581	0.179	0.058	0.039	0.719	#21	0.080	Not Required	Pass
105	0.014	0.350	0.183	0.056	0.045	0.396	#21	0.074	Not Required	Pass
106	0.014	0.565	0.099	0.056	0.023	0.669	#21	0.045	Not Required	Pass
107	0.014	0.350	0.183	0.056	0.045	0.396	#21	0.074	Not Required	Pass
108	0.003	0.079	0.130	0.035	0.021	0.185	#21	0.095	Not Required	Pass
109	0.009	0.032	0.047	0.001	0.000	0.080	#21	0.204	Not Required	Pass
110	0.014	0.581	0.179	0.058	0.039	0.719	#21	0.080	Not Required	Pass
111	0.004	0.102	0.132	0.032	0.021	0.202	#21	0.095	Not Required	Pass
112	0.002	0.397	0.209	0.096	0.039	0.548	#21	0.053	Not Required	Pass
113	0.005	0.092	0.436	0.044	0.028	0.509	#21	0.286	Not Required	Pass
114	0.007	0.132	0.434	0.046	0.028	0.544	#21	0.286	Not Required	Pass
115	0.006	0.165	0.232	0.032	0.021	0.393	#21	0.346	Not Required	Pass
116	0.003	0.146	0.231	0.035	0.021	0.374	#21	0.346	Not Required	Pass
201	0.081	0.730	0.040	0.035	0.003	0.779	#13	0.630	Not Required	Pass
202	0.001	0.381	0.195	0.090	0.039	0.518	#13	0.053	Not Required	Pass
203	0.014	0.531	0.123	0.054	0.035	0.654	#21	0.045	Not Required	Pass
204	0.014	0.501	0.169	0.050	0.038	0.608	#21	0.080	Not Required	Pass
205	0.014	0.330	0.174	0.053	0.042	0.364	#21	0.074	Not Required	Pass
206	0.010	0.420	0.055	0.041	0.013	0.480	#21	0.045	Not Required	Pass
207	0.009	0.261	0.077	0.042	0.018	0.274	#21	0.074	Not Required	Pass
208	0.000	0.017	0.060	0.014	0.008	0.077	#21	Not Required	Not Required	Pass
209	0.003	0.039	0.070	0.003	0.003	0.092	#13	0.204	Not Required	Pass
210	0.009	0.414	0.093	0.042	0.023	0.506	#21	0.080	Not Required	Pass
211	0.000	0.017	0.060	0.014	0.008	0.077	#21	Not Required	Not Required	Pass
212	0.001	0.269	0.156	0.066	0.031	0.375	#21	0.081	Not Required	Pass
213	0.005	0.092	0.413	0.045	0.028	0.445	#21	0.190	Not Required	Pass
214	0.005	0.088	0.409	0.042	0.028	0.428	#23	0.286	Not Required	Pass
215	0.006	0.168	0.232	0.033	0.020	0.393	#21	0.346	Not Required	Pass
216	0.003	0.153	0.232	0.030	0.020	0.382	#21	0.346	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																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (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> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.122</td> <td>9.698</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.376</td> <td>-3.974</td> </tr> <tr> <td>V_z (kip)</td> <td>0.216</td> <td>0.327</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.727</td> <td>1.113</td> </tr> <tr> <td>M_z (kipft)</td> <td>26.722</td> <td>45.414</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.122	9.698	V_x (kip)	-2.376	-3.974	V_z (kip)	0.216	0.327	M_x (kipft)	0.727	1.113	M_z (kipft)	26.722	45.414	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.376 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.37834 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(26.722 \text{ kipft}) + ((-2.376 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.2551 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9086 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.216 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.034395 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.727 \text{ kipft}) + ((0.216 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11576 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.4253 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.9086 \text{ ft}), (2.4253 \text{ ft})]$$

$$L_{e,req} = 5.909 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.909 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.90908$$

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

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

$$q = \frac{(6.122 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.382625 \text{ kip/ft}^2$$

$$q = 0.00200 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.38263 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.19131$$

Status: **PASS**
Ratio: **0.190**

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:

$H_o = -0.37834 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.2551 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.2551 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.2551 \text{ kipft/ft})) + (4 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.484 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (4.2551 \text{ kipft/ft})) + (3 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 [(3 \times (4.2551 \text{ kipft/ft})) + (2 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.21035 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (4.2551 \text{ kipft/ft})) + ((-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.85931 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.484 \text{ ft})}{2}$$

$$p_a = 0.3363 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21035 \text{ kip/ft}^2)}{(0.3363 \text{ kip/ft}^2)}$$

$$Ratio = 0.62548$$

p_a - Allowable lateral soil pressure at depth L_e ,

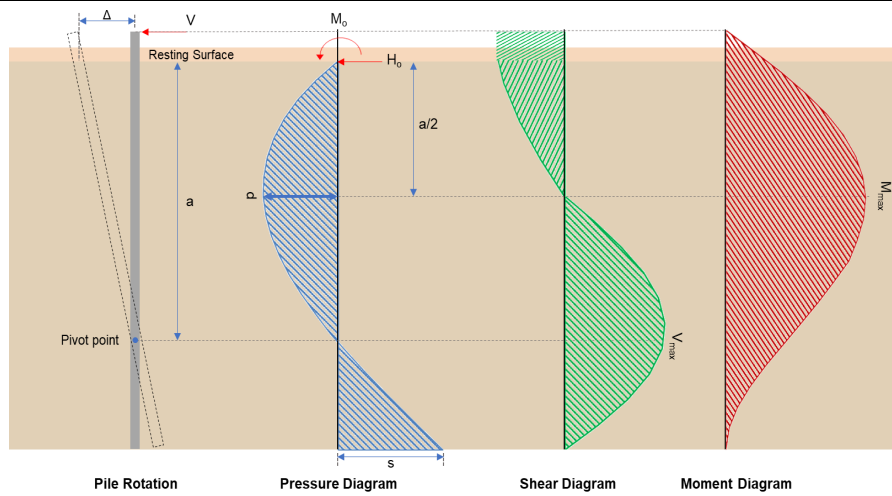
Status: **PASS**
Ratio: **0.630**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.85931 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.88134$	Status: PASS Ratio: 0.880
	<p>Considering z-direction:</p> <p>$H_o = 0.034395 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11576 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11576 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.034395 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.11576 \text{ kipft/ft})) + (4 \times (0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))}$ $a = 4.6382 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.11576 \text{ kipft/ft})) + (3 \times (0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.11576 \text{ kipft/ft})) + (2 \times (0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$ $p = 0.028722 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.11576 \text{ kipft/ft})) + ((0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$ $s = 0.064629 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.6382 \text{ ft})}{2}$ $p_a = 0.34787 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.028722 \text{ kip/ft}^2)}{(0.34787 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.082568$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.080

$$Ratio = \frac{(0.064629 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.066286$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.974 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.6328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(45.414 \text{ kipft}) + ((-3.974 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.2315 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.2315 \text{ kipft/ft})}{(-0.6328 \text{ kip/ft})}$$

$$E = 11.428 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2315 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.6328 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.2315 \text{ kipft/ft})) + (4 \times (-0.6328 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4823 \text{ ft}$$

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

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

$$V_{max} = ((-0.6328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4823 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4823 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.5442 \text{ kip}$$

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

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

$$M_{max} = ((-0.6328 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(11.428 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4823 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4823 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4823 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 29.525 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.327 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.05207 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.113 \text{ kipft}) + ((0.327 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.17723 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.17723 \text{ kipft/ft})}{(0.05207 \text{ kip/ft})}$$

$$E = 3.4037 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17723 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.05207 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.17723 \text{ kipft/ft})) + (4 \times (0.05207 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6367 \text{ ft}$$

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

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

$$V_{max} = ((0.05207 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6367 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6367 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33166 \text{ kip}$$

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

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

$$M_{max} = ((0.05207 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(3.4037 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6367 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6367 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6367 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.95708 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - 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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(0.608 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.94 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.94 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

Status: **PASS**
Ratio: **0.970**

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 No. 10: 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)**

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(9.698 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0030464$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - 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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 9.698 \text{ kip} \rightarrow 9698 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(9698 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.09 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

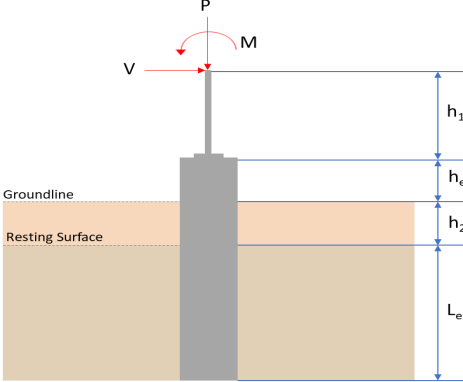
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.09 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.09 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.09 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.29 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.5442 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.5442 \text{ kip})}{(118.29 \text{ kip})}$ $\text{Ratio} = 0.080687$ <p>Considering z-direction:</p> <p>$V_{max} = 0.33166 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.33166 \text{ kip})}{(118.29 \text{ kip})}$ $\text{Ratio} = 0.0028039$	<p>Status: PASS Ratio: 0.080</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 273.423 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2545.9 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42 \text{kipft}), (2545.9 \text{kipft})]$ $\phi M_n = 273.42 \text{kipft}$ <p>Considering x-direction: $M_{max} = 29.525 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(29.525 \text{kipft})}{(273.42 \text{kipft})}$ $\text{Ratio} = 0.10798$	<p>Status: PASS Ratio: 0.110</p>
	<p>Considering z-direction: $M_{max} = 0.95708 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.95708 \text{kipft})}{(273.42 \text{kipft})}$ $\text{Ratio} = 0.0035003$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (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> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.417</td> <td>11.792</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.755</td> <td>-4.602</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>30.386</td> <td>51.971</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	7.417	11.792	V_x (kip)	-2.755	-4.602	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	30.386	51.971	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_x (kipft)	0.000	0.000																										
M_z (kipft)	30.386	51.971																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.755 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.43869 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(30.386 \text{ kipft}) + ((-2.755 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 4.8385 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 6.0967 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.0967 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.097 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.5 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.097 \text{ ft})}{(6.5 \text{ ft})}$ $\text{Ratio} = 0.938$	<p>Status: PASS Ratio: 0.940</p>
	<p>End-bearing Capacity (ASD) A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_u}{A}$ $q = \frac{(7.417 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.46356 \text{ kip/ft}^2$ <p>Check bearing capacity ratio: <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.46356 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.23178$	<p>Status: PASS Ratio: 0.230</p>
<p>Czerniak</p>	<p>Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.43869$ kip/ft - Lateral force per length of pile,

$M_o = 4.8385$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.8385 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.43869 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.8385 \text{ kipft/ft})) + (4 \times (-0.43869 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4861 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.8385 \text{ kipft/ft})) + (3 \times (-0.43869 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (4.8385 \text{ kipft/ft})) + (2 \times (-0.43869 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.23493 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.8385 \text{ kipft/ft})) + ((-0.43869 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.96931 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4861 \text{ ft})}{2}$$

$$p_a = 0.33646 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23493 \text{ kip/ft}^2)}{(0.33646 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.69826$$

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

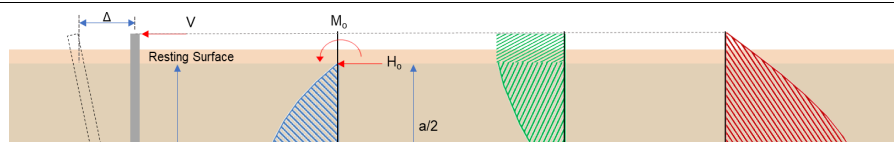
$$\text{Ratio} = \frac{s}{p_s}$$

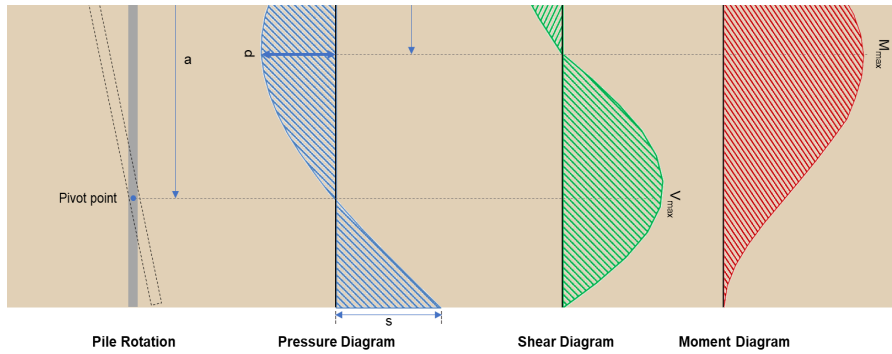
$$\text{Ratio} = \frac{(0.96931 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99416$$

Status: **PASS**
Ratio: **0.700**

Status: **PASS**
Ratio: **0.990**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.602 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.7328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(51.971 \text{ kipft}) + ((-4.602 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.2756 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.2756 \text{ kipft/ft})}{(-0.7328 \text{ kip/ft})}$$

$$E = 11.293 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.2756 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.7328 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (8.2756 \text{ kipft/ft})) + (4 \times (-0.7328 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4835 \text{ ft}$$

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

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

$$V_{max} = ((-0.7328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.293 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4835 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.293 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4835 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.945 \text{ kip}$$

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

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

$$M_{max} = ((-0.7328 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(11.293 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4835 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.293 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4835 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.293 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4835 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.841 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - 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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.792 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.87 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.87 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

Status: **PASS**
Ratio: **0.970**

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 No. 10: 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)**

Axial Compression Strength (ACI 318-19, LRFD)22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.792 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0037042$$

Status: **PASS**
Ratio: **0.000****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$ - Effective width,22.5.2.2 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

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 11.792 \text{ kip} \rightarrow 11792 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(11792 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.37 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

 V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.37 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.37 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.37 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.47 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 10.945 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.945 \text{ kip})}{(118.47 \text{ kip})}$ $\text{Ratio} = 0.092386$	<p>Status: PASS Ratio: 0.090</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 273.423 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$$\phi M_{n,z} = \phi S_x F_y$$

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

$$\phi M_{n,z} = 2545.9 \text{ kipft}$$

Therefore,
 ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$

$$\phi M_n = 273.42 \text{ kipft}$$

Considering x-direction:

$M_{max} = 33.841 \text{ kipft}$ - Maximum moment in the x-direction,

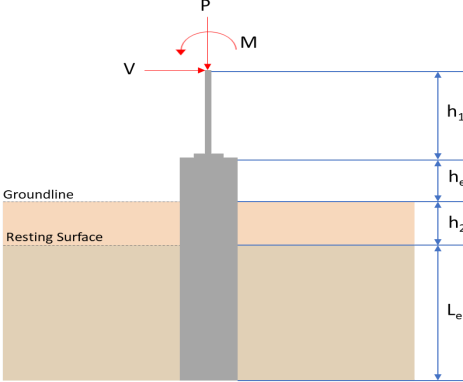
Ratio - Capacity

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

$$\text{Ratio} = \frac{(33.841 \text{ kipft})}{(273.42 \text{ kipft})}$$

$$\text{Ratio} = 0.12377$$

Status: **PASS**
Ratio: **0.120**

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (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> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.122</td> <td>9.698</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.376</td> <td>-3.974</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.216</td> <td>-0.327</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.727</td> <td>-1.113</td> </tr> <tr> <td>M_z (kipft)</td> <td>26.722</td> <td>45.415</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.122	9.698	V_x (kip)	-2.376	-3.974	V_z (kip)	-0.216	-0.327	M_x (kipft)	-0.727	-1.113	M_z (kipft)	26.722	45.415	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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$$M_o = \frac{(26.722 \text{ kipft}) + ((-2.376 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.2551 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9086 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.216 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.034395 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.727 \text{ kipft}) + ((-0.216 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11576 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7755 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.9086 \text{ ft}), (1.7755 \text{ ft})]$$

$$L_{e,req} = 5.909 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.909 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.90908$$

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

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

$$q = \frac{(6.122 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.38263 \text{ kip/ft}^2$$

$$q = 0.00200 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.38263 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19131$$

Status: **PASS**
Ratio: **0.190**

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:

$H_o = -0.37834 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.2551 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.2551 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.2551 \text{ kipft/ft})) + (4 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.484 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.2551 \text{ kipft/ft})) + (3 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (4.2551 \text{ kipft/ft})) + (2 \times (-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.21035 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.2551 \text{ kipft/ft})) + ((-0.37834 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.85931 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.484 \text{ ft})}{2}$$

$$p_a = 0.3363 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21035 \text{ kip/ft}^2)}{(0.3363 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.62548$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.630**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.85931 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88134$$

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = -0.034395 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.11576 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11576 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.034395 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.11576 \text{ kipft/ft})) + (4 \times (-0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6382 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.11576 \text{ kipft/ft})) + (3 \times (-0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.11576 \text{ kipft/ft})) + (2 \times (-0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = -0.0076659 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.11576 \text{ kipft/ft})) + ((-0.034395 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.0011307 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6382 \text{ ft})}{2}$$

$$p_a = 0.34787 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0076659 \text{ kip/ft}^2)}{(0.34787 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.022037$$

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

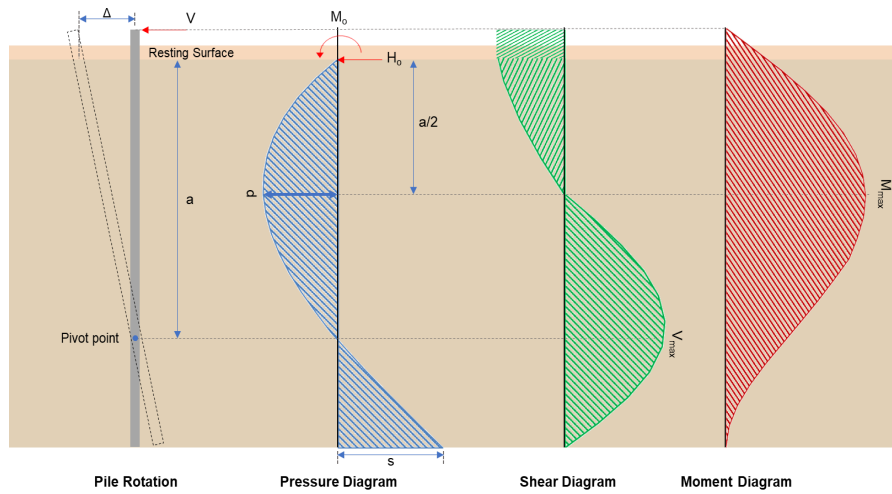
$$\text{Ratio} = \frac{s}{p_s}$$

Status: **PASS**
Ratio: **-0.020**

$$\text{Ratio} = \frac{(0.0011307 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0011597$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.974 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.6328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(45.415 \text{ kipft}) + ((-3.974 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.2317 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.2317 \text{ kipft/ft})}{(-0.6328 \text{ kip/ft})}$$

$$E = 11.428 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2317 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.6328 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.2317 \text{ kipft/ft})) + (4 \times (-0.6328 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4823 \text{ ft}$$

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

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

$$V_{max} = ((-0.6328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4823 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4823 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.5444 \text{ kip}$$

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

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

$$M_{max} = ((-0.6328 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(11.428 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4823 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4823 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (11.428 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4823 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 29.525 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.327 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.05207 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.113 \text{ kipft}) + ((-0.327 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.17723 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.17723 \text{ kipft/ft})}{(-0.05207 \text{ kip/ft})}$$

$$E = 3.4037 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17723 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.05207 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.17723 \text{ kipft/ft})) + (4 \times (-0.05207 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6367 \text{ ft}$$

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

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

$$V_{max} = ((-0.05207 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6367 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6367 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33166 \text{ kip}$$

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

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

$$M_{max} = ((-0.05207 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(3.4037 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6367 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6367 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.4037 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6367 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.95708 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - 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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(0.608 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.94 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.94 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

Status: **PASS**
Ratio: **0.970**

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 No. 10: 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)**

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(9.698 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0030464$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - 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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 9.698 \text{ kip} \rightarrow 9698 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(9698 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.09 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.09 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.09 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.09 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.29 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.5444 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.5444 \text{ kip})}{(118.29 \text{ kip})}$ $\text{Ratio} = 0.080688$ <p>Considering z-direction:</p> <p>$V_{max} = 0.33166 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.33166 \text{ kip})}{(118.29 \text{ kip})}$ $\text{Ratio} = 0.0028039$	<p>Status: PASS Ratio: 0.080</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 29.525\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(29.525\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.10798$	<p>Status: PASS Ratio: 0.110</p>
	<p>Considering z-direction: $M_{max} = 0.95708\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.95708\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0035003$	<p>Status: PASS Ratio: 0.000</p>