

Your Project Calculations



Project Name: MTSOLAR_E56D6KHB95FK

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_E56D6KHB95FK&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=7CdP1zVNYT7H8Ck0baCXAAQCOLFdzddh0VSkgo62vKJRrsBalHytrjs8qllqR

Array Specification

Product:	Beam
Unique ID:	3P-17-8TOP-XD-12-L-5Hx7W-2ECI
Duty Classification:	XD
Module Width:	41.00 in
Module Length:	74.00in
Number of Rows:	5
Number of Columns:	7
Total Number of Modules:	35
Desired Tilt Angle:	30
Front Edge Clearance:	5
Total Array Height at Tilt:	13.59 ft
Total Frame Length:	43.50 ft
Frame Weight:	2659 lbs
Array Dimensions N/S:	17.29 ft
Array Dimensions E/W:	43.75 ft
Rail Length:	207.50 in
Rail Spacing:	3.08 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	9.32 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: 5.50 ft Pile 2: 5.75 ft Pile 3: 5.50 ft
Foundation Volume:	9.926 y ³
Foundation Result:	PASSED
Mount Twist:	0.781119 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	4674 Old Clear Crk Rd, Carson City, NV 89705, USA
Wind Speed:	115 mph
Snow Load:	60 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.026391 ksf



Design Disclaimer

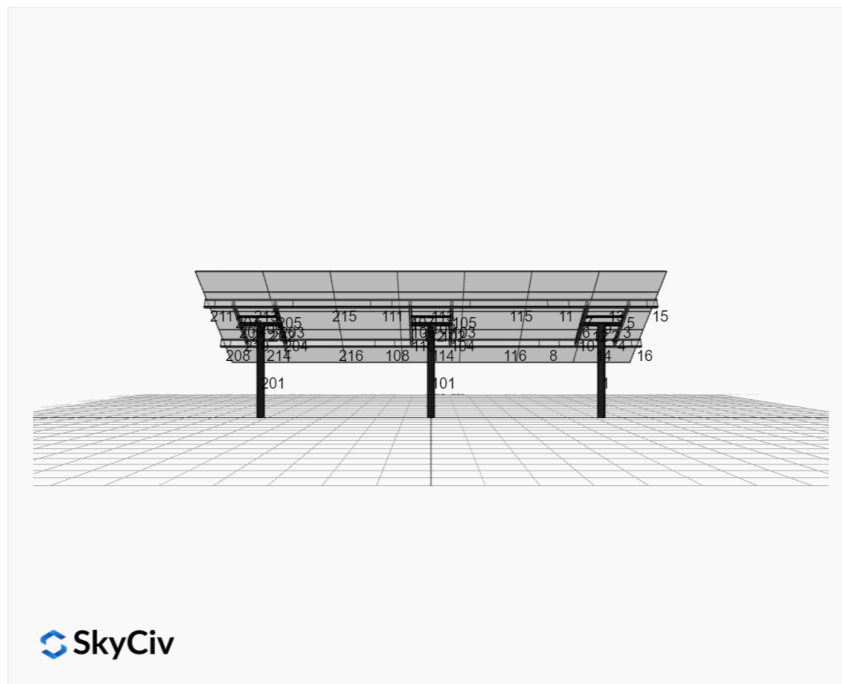
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

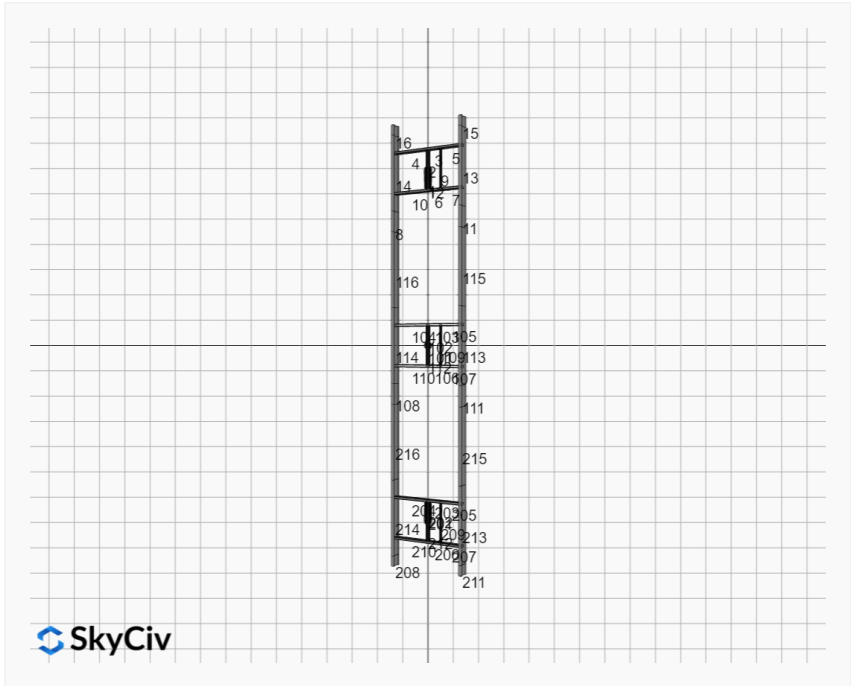
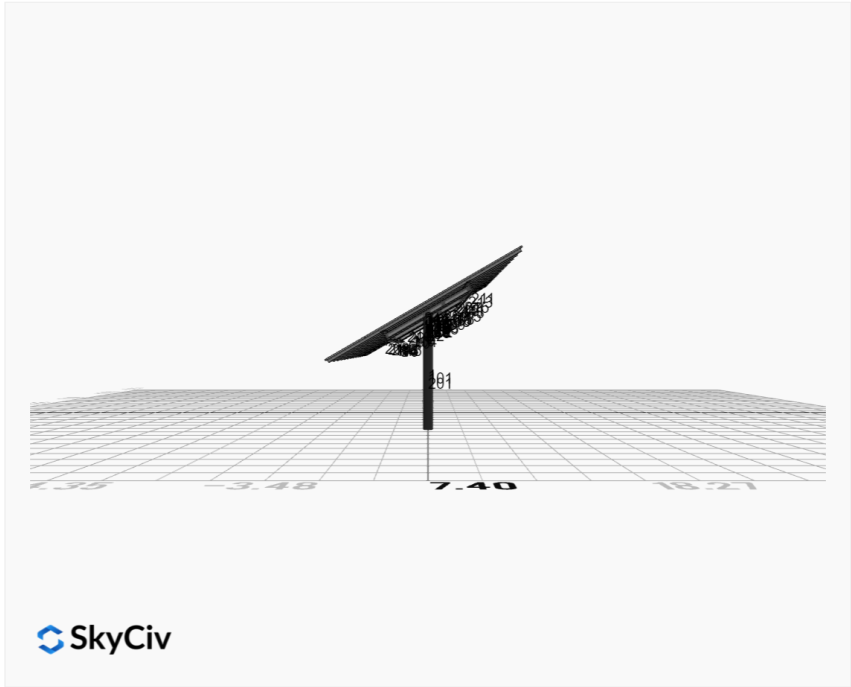
AutoDesigner Input

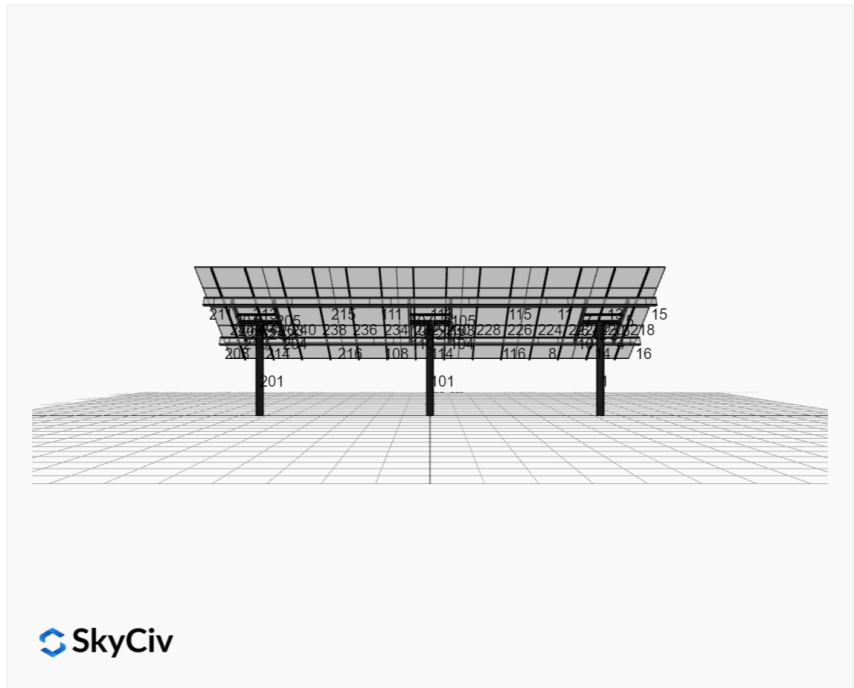
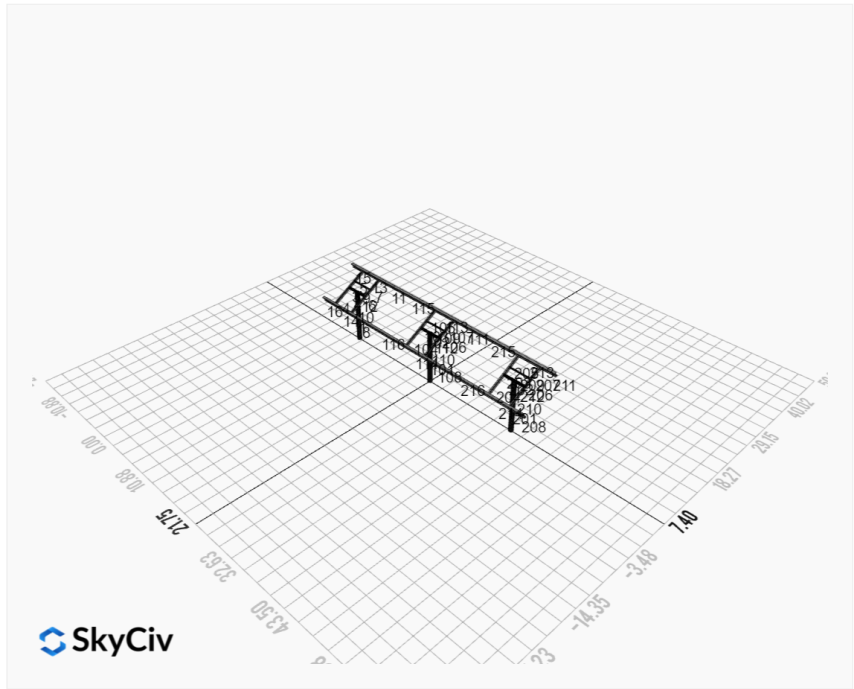
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Design Notes:

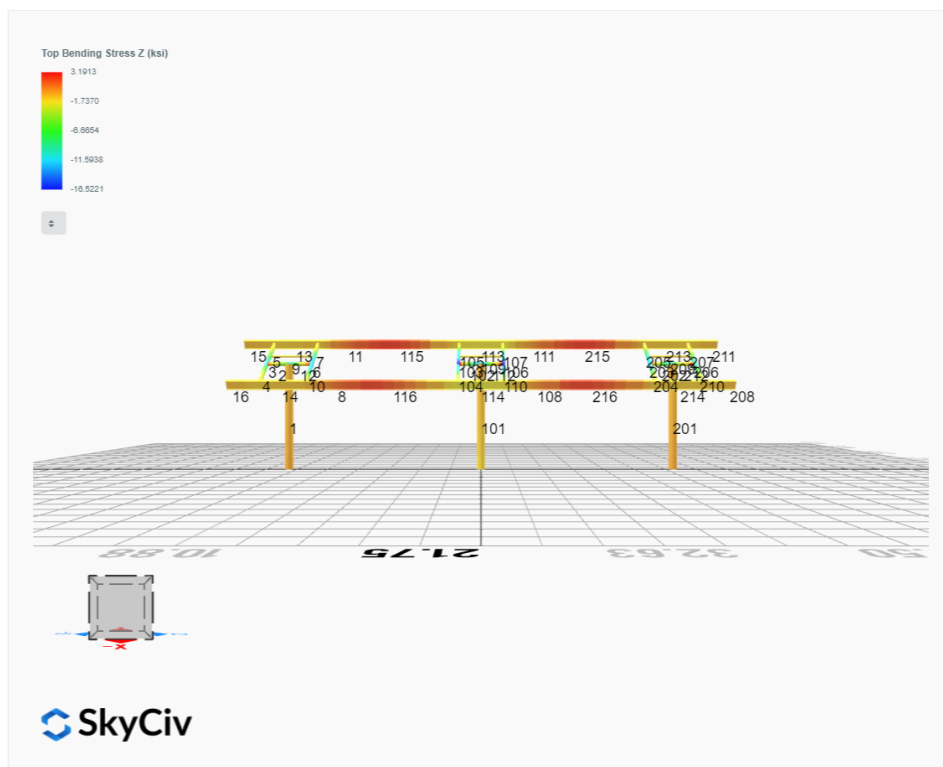
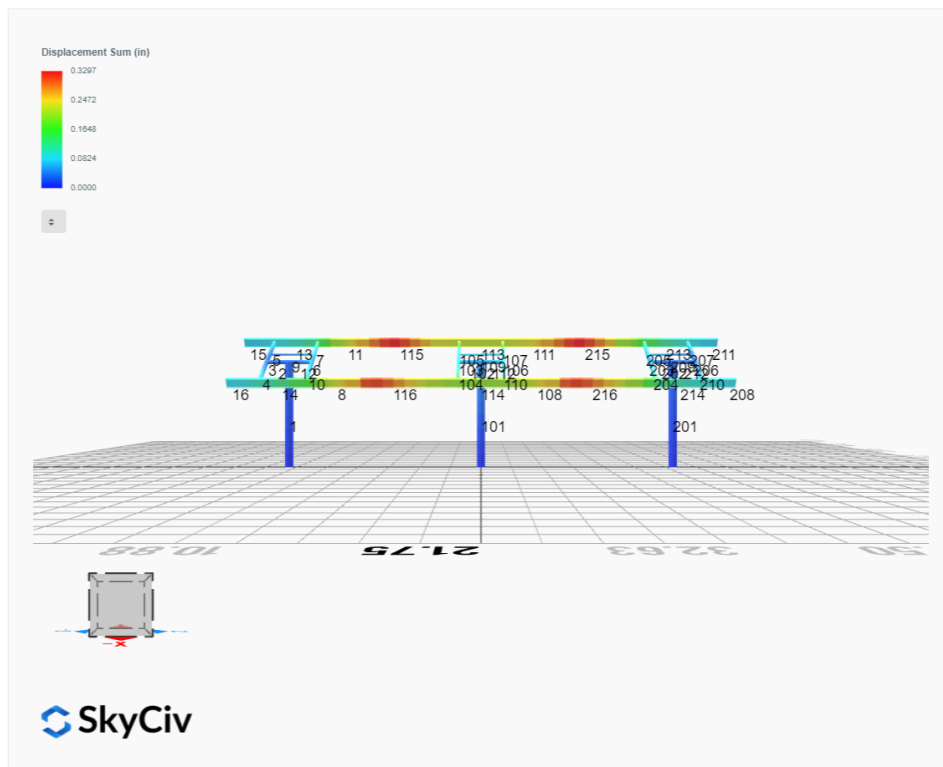
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

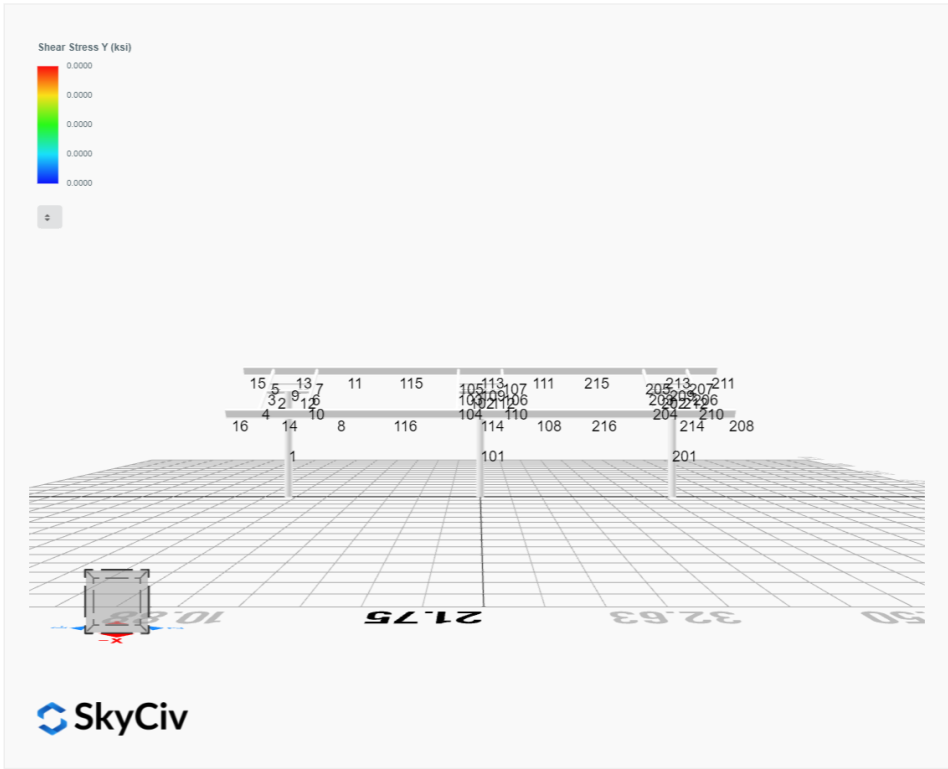
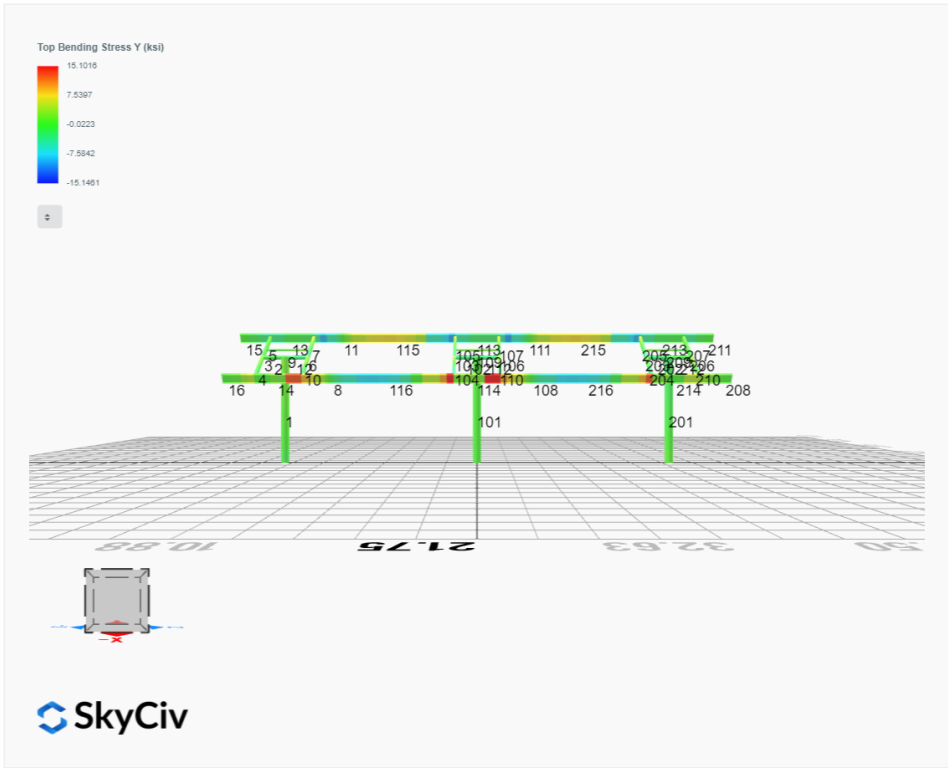


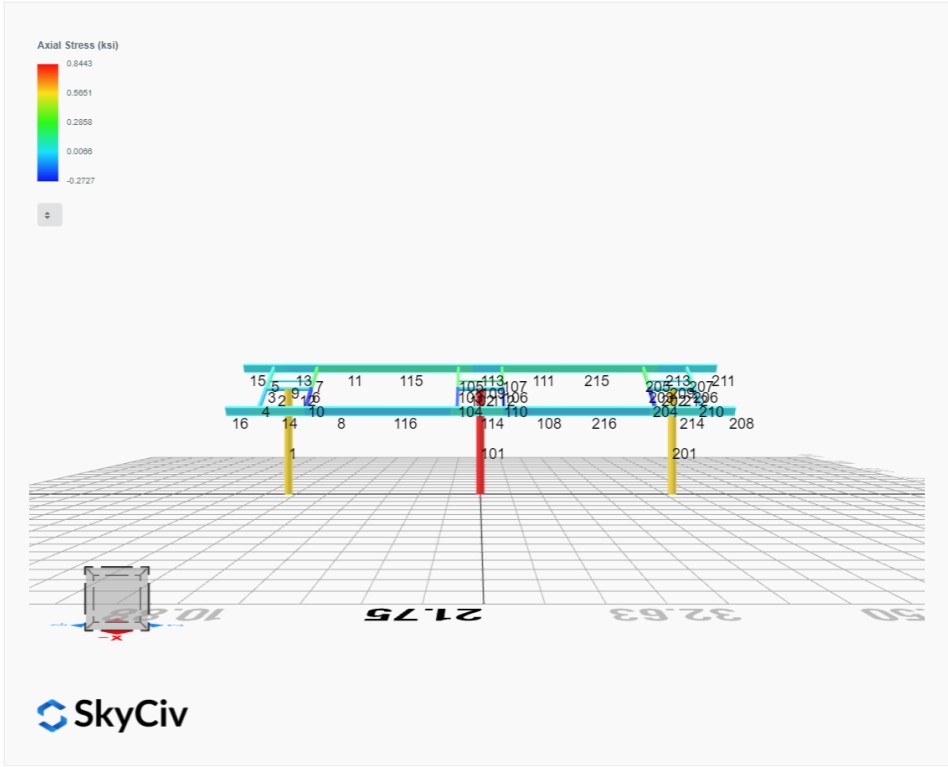




FEM Results (Envelope Worst Case for each member)







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	0.0236	1.8574	0.0687	0.1747	-0.0311	-0.1609
ULS: 2. D + L	0.0236	1.8574	0.0687	0.1747	-0.0311	-0.1609
ULS: 3. D + (S or Lr or R)	0.1142	6.9075	0.3333	0.8490	-0.1523	-0.8660
ULS: 3. D + (S or Lr or R)	0.0236	1.8574	0.0687	0.1747	-0.0311	-0.1609
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0915	5.6450	0.2672	0.6804	-0.1220	-0.6897
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0236	1.8574	0.0687	0.1747	-0.0311	-0.1609
ULS: 5b. D + 0.7E	0.0236	1.8574	0.0687	0.1747	-0.0311	-0.1609
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0915	5.6450	0.2672	0.6804	-0.1220	-0.6897
ULS: 8. 0.6D + 0.7E	0.0142	1.1144	0.0412	0.1048	-0.0186	-0.0965
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6217	4.6492	0.2543	0.6194	-0.4385	15.5718
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6217	4.6492	0.2543	0.6194	-0.4385	15.5718
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4322	-0.5339	-0.0884	-0.2014	0.3141	-13.3907
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2214	-0.1552	-0.0797	-0.1801	0.3019	-17.4134
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1424	7.7388	0.4063	1.0139	-0.4275	11.1098
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1424	7.7388	0.4063	1.0139	-0.4275	11.1098
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1480	3.8515	0.1494	0.3983	0.1369	-10.6121
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9898	4.1356	0.1558	0.4143	0.1277	-13.6291
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2103	3.9513	0.2079	0.5083	-0.3366	11.6387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2103	3.9513	0.2079	0.5083	-0.3366	11.6387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0800	0.0639	-0.0491	-0.1074	0.2278	-10.0833
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9219	0.3480	-0.0426	-0.0914	0.2187	-13.1002
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6311	3.9062	0.2268	0.5495	-0.4260	15.6362
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6311	3.9062	0.2268	0.5495	-0.4260	15.6362
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4227	-1.2769	-0.1158	-0.2713	0.3265	-13.3264
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2119	-0.8981	-0.1072	-0.2500	0.3143	-17.3490

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	12.6379
Shear X	-2.7421
Shear Z	0.6646
Moment X	1.6725
Moment Y (Twist)	0.7809
Moment Z	29.6639

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	7.7388
Shear X	-1.6311
Shear Z	0.4063
Moment X	1.0139
Moment Y (Twist)	0.4385
Moment Z	17.4134

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	-0.0473	2.3905	-0.0000	0.0000	0.0000	0.4100
ULS: 2. D + L	-0.0473	2.3905	-0.0000	0.0000	0.0000	0.4100
ULS: 3. D + (S or Lr or R)	-0.2284	9.4819	0.0000	-0.0000	0.0001	1.9185
ULS: 3. D + (S or Lr or R)	-0.0473	2.3905	-0.0000	0.0000	0.0000	0.4100
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.1831	7.7091	0.0000	-0.0000	0.0000	1.5413
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0473	2.3905	-0.0000	0.0000	0.0000	0.4100
ULS: 5b. D + 0.7E	-0.0473	2.3905	-0.0000	0.0000	0.0000	0.4100

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.1831	7.7091	0.0000	-0.0000	0.0000	1.5413
ULS: 8. 0.6D + 0.7E	-0.0284	1.4343	-0.0000	0.0000	0.0000	0.2460
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2540	6.3286	-0.0000	0.0000	0.0000	20.7516
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.2540	6.3286	-0.0000	0.0000	0.0000	20.7516
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8476	-0.9883	-0.0000	0.0000	0.0000	-16.6611
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4840	-0.3855	-0.0000	0.0000	0.0000	-20.8284
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8382	10.6626	0.0000	-0.0000	0.0000	16.7976
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8382	10.6626	0.0000	-0.0000	0.0000	16.7976
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2381	5.1750	0.0000	-0.0000	0.0000	-11.2619
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9653	5.6271	0.0000	-0.0000	0.0000	-14.3874
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7023	5.3441	-0.0000	0.0000	0.0000	15.6662
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.7023	5.3441	-0.0000	0.0000	0.0000	15.6662
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3739	-0.1436	-0.0000	0.0000	0.0000	-12.3933
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1012	0.3085	-0.0000	0.0000	0.0000	-15.5188
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2351	5.3724	-0.0000	0.0000	0.0000	20.5876
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.2351	5.3724	-0.0000	0.0000	0.0000	20.5876
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8665	-1.9445	-0.0000	0.0000	0.0000	-16.8251
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.5029	-1.3417	-0.0000	0.0000	0.0000	-20.9924

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	17.4919
Shear X	-3.8211
Shear Z	0.0000
Moment X	0.0001
Moment Y (Twist)	0.0003
Moment Z	35.5964

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	10.6626
Shear X	-2.2540
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0001
Moment Z	20.9924

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	0.0236	1.8574	-0.0687	-0.1747	0.0311	-0.1609
ULS: 2. D + L	0.0236	1.8574	-0.0687	-0.1747	0.0311	-0.1609
ULS: 3. D + (S or Lr or R)	0.1142	6.9075	-0.3333	-0.8491	0.1524	-0.8659
ULS: 3. D + (S or Lr or R)	0.0236	1.8574	-0.0687	-0.1747	0.0311	-0.1609
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0915	5.6450	-0.2672	-0.6805	0.1221	-0.6896
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0236	1.8574	-0.0687	-0.1747	0.0311	-0.1609
ULS: 5b. D + 0.7E	0.0236	1.8574	-0.0687	-0.1747	0.0311	-0.1609
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0915	5.6450	-0.2672	-0.6805	0.1221	-0.6896
ULS: 8. 0.6D + 0.7E	0.0142	1.1144	-0.0412	-0.1048	0.0186	-0.0965
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6217	4.6492	-0.2543	-0.6194	0.4385	15.5718
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6217	4.6492	-0.2543	-0.6194	0.4385	15.5718
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4322	-0.5339	0.0884	0.2014	-0.3141	-13.3907
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2214	-0.1552	0.0797	0.1801	-0.3019	-17.4134
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1424	7.7388	-0.4063	-1.0140	0.4276	11.1099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1424	7.7388	-0.4063	-1.0140	0.4276	11.1099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1480	3.8515	-0.1494	-0.3984	-0.1368	-10.6120
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9898	4.1356	-0.1558	-0.4144	-0.1276	-13.6290

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	-1.2103	3.9513	-0.2079	-0.5083	0.3366	11.6387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2103	3.9513	-0.2079	-0.5083	0.3366	11.6387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0800	0.0639	0.0491	0.1074	-0.2278	-10.0833
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9219	0.3480	0.0426	0.0914	-0.2186	-13.1002
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6311	3.9062	-0.2268	-0.5495	0.4260	15.6362
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6311	3.9062	-0.2268	-0.5495	0.4260	15.6362
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4227	-1.2769	0.1158	0.2713	-0.3265	-13.3264
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2119	-0.8981	0.1072	0.2500	-0.3143	-17.3490

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	12.6379
Shear X	-2.7421
Shear Z	-0.6646
Moment X	-1.6726
Moment Y (Twist)	0.7811
Moment Z	29.6645

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	7.7388
Shear X	-1.6311
Shear Z	-0.4063
Moment X	-1.0140
Moment Y (Twist)	0.4385
Moment Z	17.4134

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)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

108	20	1.33	1.33	2.0 5	1.55,1.55,1.55,1.56,1.55,1.55,1.54,1.54,1.44,1.27,1.53,1.53,1.51,1.09,1.55,1.55,1.57,1.49,1.53, 1.53,1.49,1.19,1.53,1.53,1.52,1.25	3 0 0	2 0 0	1
109	3	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
110	17	2.44	2.44	3.7 5	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.16,1.67,1.67,1.67,1.67,1.67, 1.67,1.63,1.71,1.67,1.67,1.66,1.61	3 0 0	2 0 0	1
111	20	1.33	1.33	2.0 5	1.43,1.43,1.43,1.43,1.43,1.43,1.33,1.33,1.10,1.12,1.30,1.30,1.25,1.20,1.38,1.38,1.52,1.76,1.32, 1.32,1.17,1.16,1.30,1.30,1.26,1.21	3 0 0	2 0 0	1
112	6	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
113	20	4.88	4.00	7.5 0	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.96,1.95,1.09,1.09,1.15,2.69,1.07,1.07,1.05,1.04,1.08, 1.08,3.51,3.06,1.09,1.09,1.11,2.26	3 0 0	2 0 0	1
114	20	4.88	4.00	7.5 0	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.22,1.05,1.05,1.06,1.00,1.05,1.05,1.05,1.06,1.05, 1.05,1.06,3.76,1.05,1.05,1.05,1.01	3 0 0	2 0 0	1
115	20	4.84	4.84	7.4 5	1.13,1.13,1.13,1.13,1.13,1.13,1.10,1.10,1.04,1.04,1.09,1.09,1.08,1.06,1.11,1.11,1.15,1.19,1.10, 1.10,1.06,1.05,1.09,1.09,1.08,1.07	3 0 0	2 0 0	1
116	20	4.84	4.84	7.4 5	1.15,1.16,1.15,1.16,1.16,1.15,1.15,1.15,1.14,1.08,1.15,1.15,1.15,1.48,1.16,1.16,1.16,1.14,1.15, 1.15,1.15,1.06,1.15,1.15,1.15,2.39	3 0 0	2 0 0	1
201	9	19.5 8	19.5 8	9.3 2	-	3 0 0	2 0 0	1
202	6	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
203	17	0.92	0.92	1.4 2	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19, 1.19,1.16,1.17,1.18,1.18,1.18,1.18	3 0 0	2 0 0	1
204	17	2.44	2.44	3.7 5	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.63,1.68,1.67,1.67,1.66,2.06,1.67,1.67,1.67,1.67,1.68, 1.68,1.62,1.72,1.67,1.67,1.66,1.55	3 0 0	2 0 0	1
205	17	1.52	1.52	2.3 3	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67, 1.67,1.64,1.66,1.67,1.67,1.66,1.66	3 0 0	2 0 0	1
206	17	0.92	0.92	1.4 2	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.14,1.16,1.18,1.18,1.16,1.17,1.17,1.17,1.18,1.18,1.18, 1.18,1.14,1.16,1.18,1.18,1.17,1.17	3 0 0	2 0 0	1
207	17	1.52	1.52	2.3 3	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67, 1.67,1.63,1.65,1.67,1.67,1.66,1.66	3 0 0	2 0 0	1
208	20	2.10	2.10	1.0 0	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33, 2.33,2.33,2.32,2.33,2.33,2.33,2.32	3 0 0	2 0 0	1
209	3	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
210	17	2.44	2.44	3.7 5	1.70,1.68,1.70,1.67,1.69,1.70,1.67,1.67,1.62,1.68,1.67,1.67,1.65,1.77,1.67,1.67,1.68,1.67,1.68, 1.68,1.61,1.72,1.67,1.67,1.66,1.97	3 0 0	2 0 0	1
211	20	2.10	2.10	1.0 0	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33, 2.33,2.33,2.33,2.33,2.33,2.33,2.33	3 0 0	2 0 0	1
212	6	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
213	20	4.88	4.00	7.5 0	1.45,1.47,1.45,1.48,1.46,1.45,1.44,1.44,1.38,1.57,1.42,1.42,1.42,1.51,1.47,1.47,1.51,1.52,1.43, 1.43,1.40,1.54,1.42,1.42,1.42,1.50	3 0 0	2 0 0	1
214	20	4.88	4.00	7.5 0	1.49,1.53,1.49,1.55,1.52,1.49,1.60,1.60,2.05,1.77,1.61,1.61,1.71,1.91,1.57,1.57,1.51,1.54,1.59, 1.59,1.89,1.93,1.62,1.62,1.68,1.76	3 0 0	2 0 0	1
215	20	4.84	4.84	7.4 5	1.06,1.06,1.06,1.06,1.06,1.06,1.07,1.07,1.12,1.11,1.07,1.07,1.07,1.08,1.06,1.06,1.06,1.06,1.07, 1.07,1.08,1.09,1.07,1.07,1.07,1.08	3 0 0	2 0 0	1
216	20	4.84	4.84	7.4 5	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.05,1.08,1.06,1.06,1.06,1.89,1.06,1.06,1.06,1.06,1.06, 1.06,1.06,1.10,1.06,1.06,1.06,2.03	3 0 0	2 0 0	1

Member Design Capacity

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	236.79	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	42.17	6.46	56.26	44.91
14	159.30	116.35	45.54	6.46	56.26	44.91
15	159.30	137.23	46.90	6.46	56.26	44.91
16	159.30	137.23	46.90	6.46	56.26	44.91
101	377.97	236.79	83.29	83.29	113.39	113.39
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.78	6.46	56.26	44.91
114	159.30	116.35	30.56	6.46	56.26	44.91
115	159.30	104.63	31.97	6.46	56.26	44.91
116	159.30	104.63	32.58	6.46	56.26	44.91
201	377.97	236.79	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	137.23	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	137.23	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	42.17	6.46	56.26	44.91
214	159.30	116.35	45.54	6.46	56.26	44.91
215	159.30	104.63	32.58	6.46	56.26	44.91
216	159.30	104.63	32.27	6.46	56.26	44.91

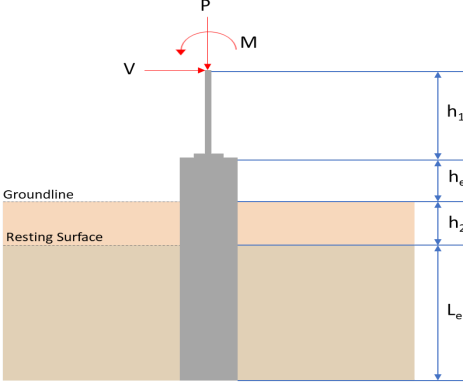
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.053	0.356	0.054	0.024	0.006	0.360	#16	0.400	Not Required	Pass
2	0.001	0.282	0.086	0.067	0.017	0.349	#21	0.054	Not Required	Pass
3	0.007	0.448	0.025	0.044	0.005	0.471	#21	0.046	Not Required	Pass
4	0.005	0.441	0.064	0.044	0.015	0.507	#21	0.082	Not Required	Pass
5	0.006	0.278	0.026	0.044	0.005	0.285	#21	0.076	Not Required	Pass
6	0.011	0.600	0.119	0.061	0.036	0.724	#21	0.046	Not Required	Pass
7	0.011	0.372	0.145	0.059	0.036	0.406	#21	0.076	Not Required	Pass
8	0.003	0.120	0.129	0.031	0.020	0.250	#21	0.102	Not Required	Pass
9	0.001	0.045	0.081	0.002	0.005	0.123	#21	0.206	Not Required	Pass
10	0.012	0.577	0.128	0.058	0.029	0.653	#21	0.082	Not Required	Pass
11	0.006	0.123	0.125	0.033	0.020	0.250	#21	0.102	Not Required	Pass
12	0.001	0.448	0.115	0.097	0.021	0.536	#21	0.054	Not Required	Pass
13	0.008	0.063	0.401	0.047	0.027	0.413	#23	0.306	Not Required	Pass
14	0.004	0.065	0.392	0.045	0.027	0.408	#24	0.204	Not Required	Pass
15	0.000	0.005	0.015	0.008	0.004	0.019	#21	Not Required	Not Required	Pass
16	0.000	0.005	0.015	0.008	0.004	0.019	#21	Not Required	Not Required	Pass
101	0.074	0.427	0.000	0.034	0.000	0.455	#13	0.400	Not Required	Pass
102	0.002	0.515	0.147	0.114	0.025	0.640	#21	0.054	Not Required	Pass
103	0.011	0.727	0.079	0.072	0.018	0.810	#21	0.046	Not Required	Pass
104	0.011	0.743	0.149	0.074	0.033	0.856	#21	0.082	Not Required	Pass
105	0.011	0.451	0.152	0.072	0.039	0.489	#21	0.076	Not Required	Pass
106	0.011	0.727	0.079	0.072	0.018	0.810	#21	0.046	Not Required	Pass
107	0.011	0.451	0.152	0.072	0.039	0.489	#21	0.076	Not Required	Pass
108	0.003	0.085	0.128	0.042	0.021	0.190	#21	0.102	Not Required	Pass
109	0.010	0.056	0.044	0.001	0.000	0.105	#21	0.206	Not Required	Pass
110	0.011	0.743	0.149	0.074	0.033	0.856	#21	0.082	Not Required	Pass
111	0.006	0.102	0.130	0.040	0.021	0.209	#21	0.102	Not Required	Pass
112	0.002	0.515	0.147	0.114	0.025	0.640	#21	0.054	Not Required	Pass
113	0.008	0.134	0.426	0.053	0.028	0.536	#21	0.306	Not Required	Pass
114	0.006	0.176	0.424	0.055	0.028	0.572	#21	0.306	Not Required	Pass
115	0.009	0.197	0.223	0.040	0.021	0.427	#21	0.370	Not Required	Pass
116	0.003	0.183	0.223	0.042	0.021	0.402	#21	0.370	Not Required	Pass
201	0.053	0.356	0.054	0.024	0.006	0.360	#16	0.400	Not Required	Pass
202	0.001	0.448	0.115	0.097	0.021	0.536	#21	0.054	Not Required	Pass
203	0.011	0.600	0.119	0.061	0.036	0.724	#21	0.046	Not Required	Pass
204	0.012	0.577	0.128	0.058	0.029	0.653	#21	0.082	Not Required	Pass
205	0.011	0.372	0.145	0.059	0.036	0.406	#21	0.076	Not Required	Pass
206	0.007	0.448	0.025	0.044	0.005	0.471	#21	0.046	Not Required	Pass
207	0.006	0.278	0.026	0.044	0.005	0.285	#21	0.076	Not Required	Pass
208	0.000	0.005	0.015	0.008	0.004	0.019	#21	Not Required	Not Required	Pass
209	0.001	0.045	0.081	0.002	0.005	0.123	#21	0.206	Not Required	Pass
210	0.005	0.441	0.064	0.044	0.015	0.507	#21	0.082	Not Required	Pass
211	0.000	0.005	0.015	0.008	0.004	0.019	#21	Not Required	Not Required	Pass
212	0.001	0.282	0.086	0.067	0.017	0.349	#21	0.054	Not Required	Pass
213	0.008	0.063	0.401	0.047	0.027	0.413	#23	0.204	Not Required	Pass
214	0.004	0.065	0.392	0.045	0.027	0.408	#24	0.306	Not Required	Pass
215	0.009	0.211	0.223	0.033	0.020	0.438	#21	0.370	Not Required	Pass
216	0.003	0.198	0.223	0.031	0.020	0.421	#21	0.370	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</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.5$ 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>7.739</td> <td>12.638</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.631</td> <td>-2.742</td> </tr> <tr> <td>V_z (kip)</td> <td>0.406</td> <td>0.665</td> </tr> <tr> <td>M_x (kipft)</td> <td>1.014</td> <td>1.673</td> </tr> <tr> <td>M_z (kipft)</td> <td>17.413</td> <td>29.664</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ 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)	7.739	12.638	V_x (kip)	-1.631	-2.742	V_z (kip)	0.406	0.665	M_x (kipft)	1.014	1.673	M_z (kipft)	17.413	29.664	
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																									
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M_x (kipft)	1.014	1.673																										
M_z (kipft)	17.413	29.664																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.631 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25971 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(17.413 \text{ kipft}) + ((-1.631 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7728 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.2019 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.406 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.06465 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.014 \text{ kipft}) + ((0.406 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16146 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.8895 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.2019 \text{ ft}), (2.8895 \text{ ft})]$$

$$L_{e,req} = 5.202 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.202 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.94582$$

Status: **PASS**
Ratio: **0.950**

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

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

$$q = \frac{(7.739 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.48369 \text{ kip/ft}^2$$

$$q = 0.48369 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.48369 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24184$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25971 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7728 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7728 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.7728 \text{ kipft/ft})) + (4 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7838 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (2.7728 \text{ kipft/ft})) + (3 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 [(3 \times (2.7728 \text{ kipft/ft})) + (2 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.21027 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (2.7728 \text{ kipft/ft})) + ((-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.81662 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7838 \text{ ft})}{2}$$

$$p_a = 0.28379 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21027 \text{ kip/ft}^2)}{(0.28379 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.74096$$

p_a - Allowable lateral soil pressure at depth L_e ,

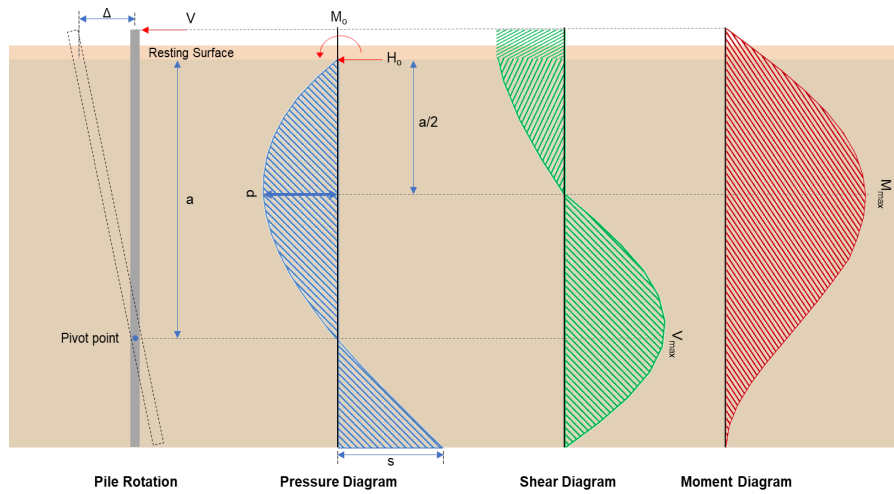
Status: **PASS**
Ratio: **0.740**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.81662 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98984$	<p>Status: PASS Ratio: 0.990</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.06465 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.16146 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.16146 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.06465 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.16146 \text{ kipft/ft})) + (4 \times (0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))}$ $a = 3.9393 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.16146 \text{ kipft/ft})) + (3 \times (0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.16146 \text{ kipft/ft})) + (2 \times (0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$ $p = 0.060824 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.16146 \text{ kipft/ft})) + ((0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$ $s = 0.13458 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.9393 \text{ ft})}{2}$ $p_a = 0.29545 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.060824 \text{ kip/ft}^2)}{(0.29545 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.20587$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.210</p>

$$\text{Ratio} = \frac{(0.13458 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.16313$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.742 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.664 \text{ kipft}) + ((-2.742 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.7236 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.7236 \text{ kipft/ft})}{(-0.43662 \text{ kip/ft})}$$

$$E = 10.818 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7236 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.43662 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.7236 \text{ kipft/ft})) + (4 \times (-0.43662 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7827 \text{ ft}$$

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

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

$$V_{max} = ((-0.43662 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.818 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7827 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.818 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7827 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2317 \text{ kip}$$

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

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

$$M_{max} = ((-0.43662 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(10.818 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7827 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.818 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7827 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (10.818 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7827 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 19.014 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.665 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.10589 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.673 \text{ kipft}) + ((0.665 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.2664 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.2664 \text{ kipft/ft})}{(0.10589 \text{ kip/ft})}$$

$$E = 2.5158 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.2664 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.10589 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.2664 \text{ kipft/ft})) + (4 \times (0.10589 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9385 \text{ ft}$$

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

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

$$V_{max} = ((0.10589 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9385 \text{ ft})}{(5.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9385 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.62543 \text{ kip}$$

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

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

$$M_{max} = ((0.10589 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(2.5158 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9385 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9385 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9385 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 1.5124 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.638 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.176 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.176 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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 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)

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.638 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0047242$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.638 \text{ kip} \rightarrow 12638 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12638 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.17 \text{ kip}$$

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

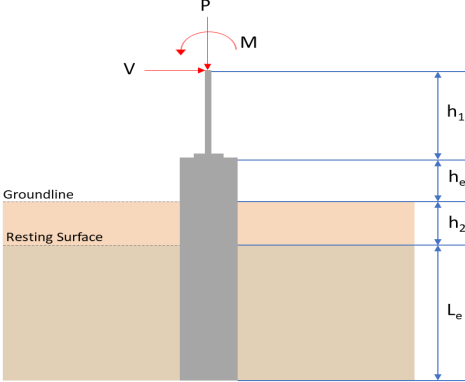
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.17 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.17 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.17 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.19 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.2317 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.2317 \text{ kip})}{(111.19 \text{ kip})}$ $\text{Ratio} = 0.065038$ <p>Considering z-direction:</p> <p>$V_{max} = 0.62543 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.62543 \text{ kip})}{(111.19 \text{ kip})}$ $\text{Ratio} = 0.0056248$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.010</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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 19.014 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.014 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.076176$	<p>Status: PASS Ratio: 0.080</p>
	<p>Considering z-direction: $M_{max} = 1.5124 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.5124 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0060592$	<p>Status: PASS Ratio: 0.010</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 $L = 5.5$ 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>7.739</td> <td>12.638</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.631</td> <td>-2.742</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.406</td> <td>-0.665</td> </tr> <tr> <td>M_x (kipft)</td> <td>-1.014</td> <td>-1.673</td> </tr> <tr> <td>M_z (kipft)</td> <td>17.413</td> <td>29.665</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ 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)	7.739	12.638	V_x (kip)	-1.631	-2.742	V_z (kip)	-0.406	-0.665	M_x (kipft)	-1.014	-1.673	M_z (kipft)	17.413	29.665	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.631 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25971 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(17.413 \text{ kipft}) + ((-1.631 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7728 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.2019 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.406 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.06465 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.014 \text{ kipft}) + ((-0.406 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16146 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.8075 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.2019 \text{ ft}), (1.8075 \text{ ft})]$$

$$L_{e,req} = 5.202 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.202 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.94582$$

Status: **PASS**
Ratio: **0.950**

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

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

$$q = \frac{(7.739 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.48369 \text{ kip/ft}^2$$

$$q = 0.48369 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.48369 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.24184$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25971 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7728 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7728 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.7728 \text{ kipft/ft})) + (4 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7838 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (2.7728 \text{ kipft/ft})) + (3 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 [(3 \times (2.7728 \text{ kipft/ft})) + (2 \times (-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.21027 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (2.7728 \text{ kipft/ft})) + ((-0.25971 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.81662 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7838 \text{ ft})}{2}$$

$$p_a = 0.28379 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21027 \text{ kip/ft}^2)}{(0.28379 \text{ kip/ft}^2)}$$

$$Ratio = 0.74096$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.740**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.81662 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98984$$

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

Considering z-direction:

$H_o = -0.06465 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.16146 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16146 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.06465 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.16146 \text{ kipft/ft})) + (4 \times (-0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9393 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.16146 \text{ kipft/ft})) + (3 \times (-0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.16146 \text{ kipft/ft})) + (2 \times (-0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = -0.019367 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.16146 \text{ kipft/ft})) + ((-0.06465 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = -0.0064747 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9393 \text{ ft})}{2}$$

$$p_a = 0.29545 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.019367 \text{ kip/ft}^2)}{(0.29545 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.065551$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

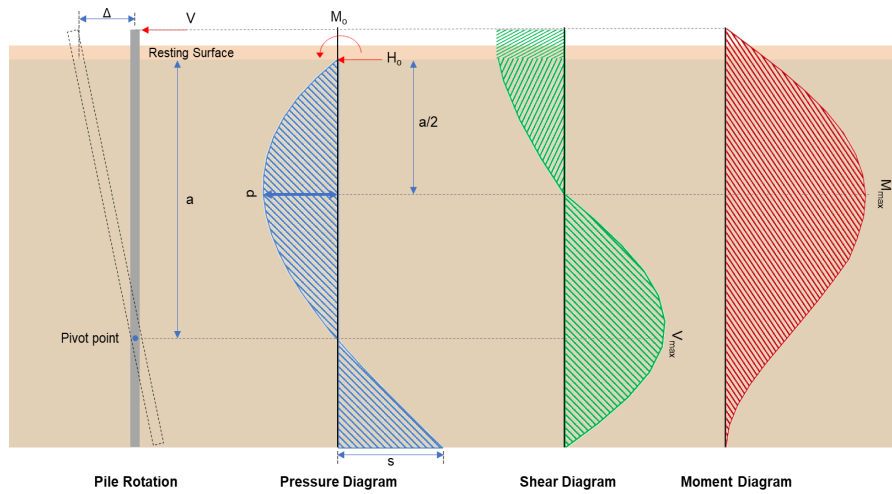
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$\text{Ratio} = \frac{(-0.0064747 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0078481$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.742 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.665 \text{ kipft}) + ((-2.742 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.7237 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.7237 \text{ kipft/ft})}{(-0.43662 \text{ kip/ft})}$$

$$E = 10.819 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7237 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.43662 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.7237 \text{ kipft/ft})) + (4 \times (-0.43662 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7827 \text{ ft}$$

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

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

$$V_{max} = ((-0.43662 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.819 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7827 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.819 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7827 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2319 \text{ kip}$$

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

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

$$M_{max} = ((-0.43662 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(10.819 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7827 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.819 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7827 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (10.819 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7827 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 19.014 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.665 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10589 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.673 \text{ kipft}) + ((-0.665 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.2664 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.2664 \text{ kipft/ft})}{(-0.10589 \text{ kip/ft})}$$

$$E = 2.5158 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.2664 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.10589 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.2664 \text{ kipft/ft})) + (4 \times (-0.10589 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9385 \text{ ft}$$

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

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

$$V_{max} = ((-0.10589 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9385 \text{ ft})}{(5.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9385 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.62543 \text{ kip}$$

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

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

$$M_{max} = ((-0.10589 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(2.5158 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9385 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9385 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (2.5158 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9385 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 1.5124 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.638 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.176 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.176 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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 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)

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.638 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0047242$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.638 \text{ kip} \rightarrow 12638 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12638 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.17 \text{ kip}$$

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

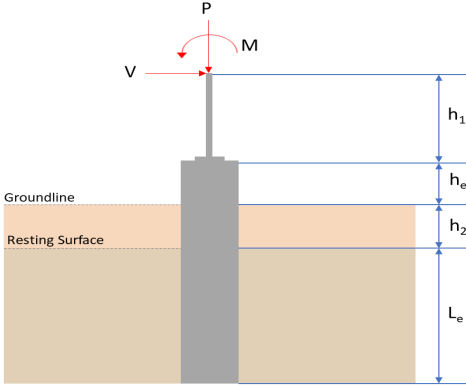
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.17 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.17 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.17 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.19 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.2319 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.2319 \text{ kip})}{(111.19 \text{ kip})}$ $\text{Ratio} = 0.06504$ <p>Considering z-direction:</p> <p>$V_{max} = 0.62543 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.62543 \text{ kip})}{(111.19 \text{ kip})}$ $\text{Ratio} = 0.0056248$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.010</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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 19.014 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.014 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.076178$	<p>Status: PASS Ratio: 0.080</p>
	<p>Considering z-direction: $M_{max} = 1.5124 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.5124 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0060592$	<p>Status: PASS Ratio: 0.010</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 $L = 5.75$ 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 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.663</td> <td>17.492</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.254</td> <td>-3.821</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>20.992</td> <td>35.596</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ 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)	10.663	17.492	V_x (kip)	-2.254	-3.821	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	20.992	35.596	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.254 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.35892 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(20.992 \text{ kipft}) + ((-2.254 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3427 \text{ kipft/ft}$$

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:

$$L_{e,x} = 5.3413 \text{ ft} - \text{Required depth in x-direction,}$$

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.3413 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.341 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.341 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.92887$$

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

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

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

$$q = \frac{(10.663 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.66644 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.66644 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.33322$$

Status: **PASS**
Ratio: **0.330**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.35892$ kip/ft - Lateral force per length of pile,

$M_o = 3.3427$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3427 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.35892 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.3427 \text{ kipft/ft})) + (4 \times (-0.35892 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9731 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3427 \text{ kipft/ft})) + (3 \times (-0.35892 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.3427 \text{ kipft/ft})) + (2 \times (-0.35892 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.19816 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3427 \text{ kipft/ft})) + ((-0.35892 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.8387 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9731 \text{ ft})}{2}$$

$$p_a = 0.29798 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19816 \text{ kip/ft}^2)}{(0.29798 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.66501$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

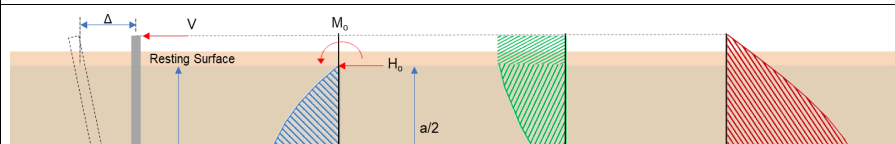
$$\text{Ratio} = \frac{s}{p_s}$$

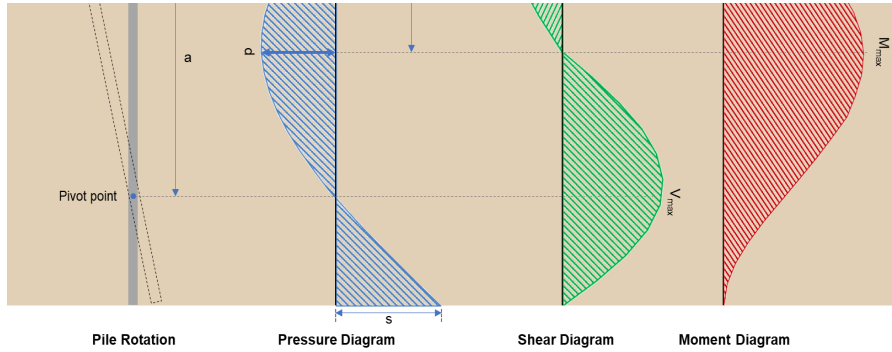
$$\text{Ratio} = \frac{(0.8387 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9724$$

Status: **PASS**
Ratio: **0.670**

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.821 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.60844 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.596 \text{ kipft}) + ((-3.821 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6682 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6682 \text{ kipft/ft})}{(-0.60844 \text{ kip/ft})}$$

$$E = 9.3159 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6682 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.60844 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.6682 \text{ kipft/ft})) + (4 \times (-0.60844 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.973 \text{ ft}$$

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

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

$$V_{max} = ((-0.60844 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.3159 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.973 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.3159 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.973 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.5822 \text{ kip}$$

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

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

$$M_{max} = ((-0.60844 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(9.3159 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.973 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.3159 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.973 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.3159 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.973 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.404 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.492 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.015 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.015 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(17.492 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0065386$$

Status: **PASS**
Ratio: **0.010****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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.492 \text{ kip} \rightarrow 17492 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(17492 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.82 \text{ kip}$$

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

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

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

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

 V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.82 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.82 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.82 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.61 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.5822 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.5822 \text{ kip})}{(111.61 \text{ kip})}$ $\text{Ratio} = 0.076892$	<p>Status: PASS Ratio: 0.080</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: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = 0.85 f'_c S_m$	

$$\phi M_{n,2} = \phi F_y Z_{xk} S_m$$

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

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

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:

$M_{max} = 23.404 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(23.404 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.093767$$

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