

Your Project Calculations



Project Name: Wachtell 30 CU

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Wachtell%20%2030%20CU&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/10_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=AG5ihW270x6jpmCZ7OT450nmCTUjZKNutD5Bz7WkiWSOySpxiTd5T8mR0dEMHSH3

Array Specification

Product:	Beam
Unique ID:	2P-19.75-8TOP-HD-45-L-5Hx6W-LA1C
Duty Classification:	HD
Module Width:	40.00 in
Module Length:	72.00in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Desired Tilt Angle:	35
Front Edge Clearance:	8
Total Array Height at Tilt:	17.62 ft
Total Frame Length:	34.75 ft
Frame Weight:	1858 lbs
Array Dimensions N/S:	16.88 ft
Array Dimensions E/W:	36.50 ft
Rail Length:	202.50 in
Rail Spacing:	3.04 ft
Rail Check:	PASS (80% utilized)

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	12.84 ft
Number of Poles:	2
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.25 ft Pile 2: 8.25 ft
Foundation Volume:	9.778 y ³
Foundation Result:	PASSED
Mount Twist:	0.436753 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	clay
Site Location:	2733 E Warm Springs Ave, Boise, ID 83712, USA
Wind Speed:	95 mph
Snow Load:	57.4 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.022092 ksf



Design Disclaimer

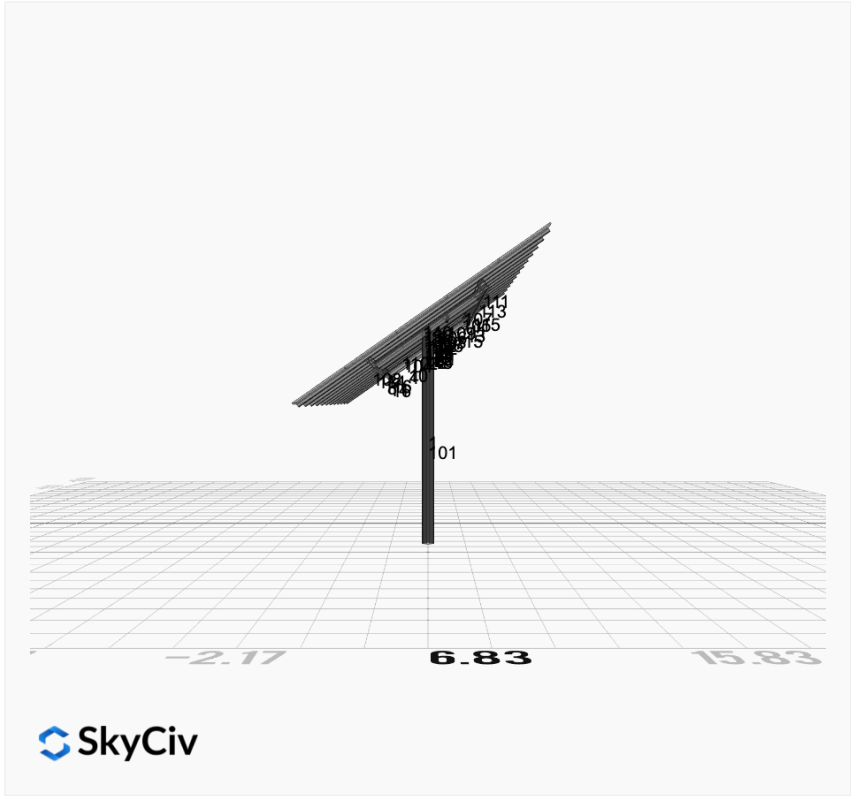
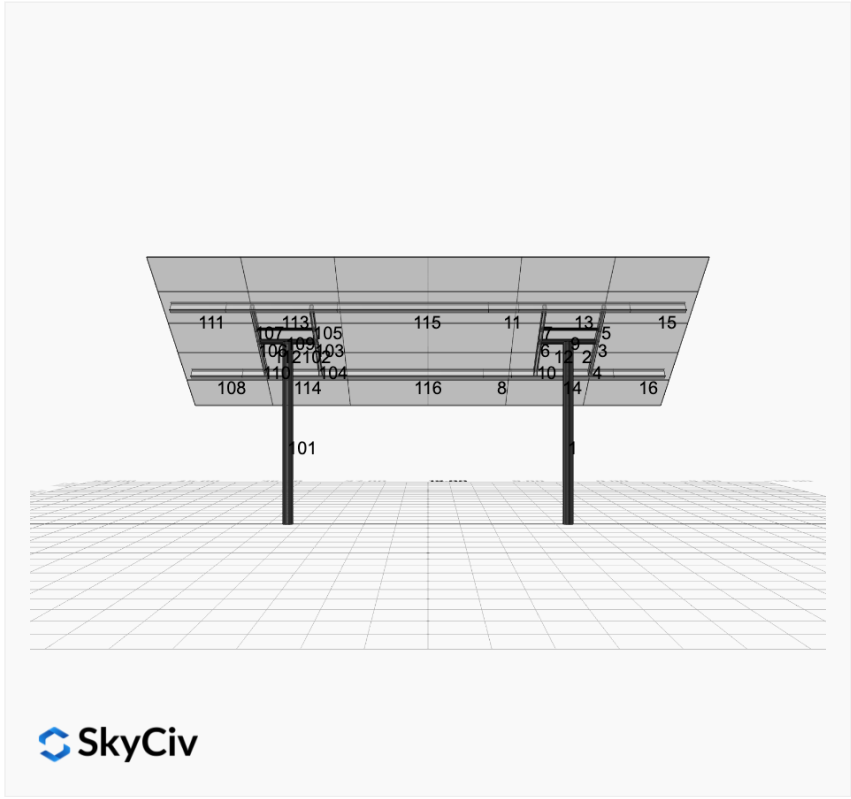
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

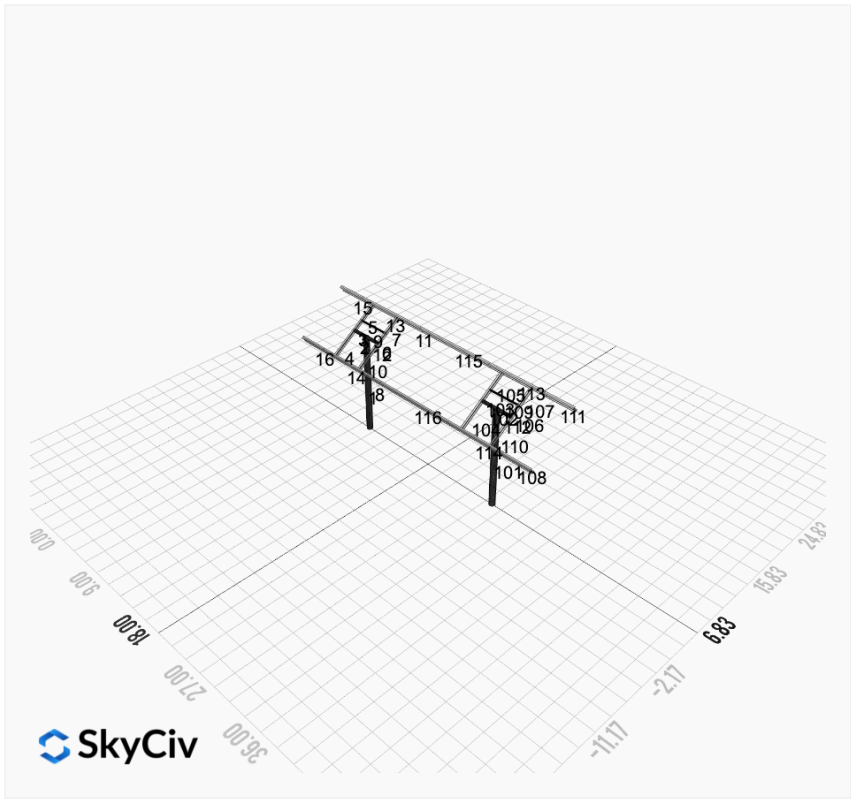
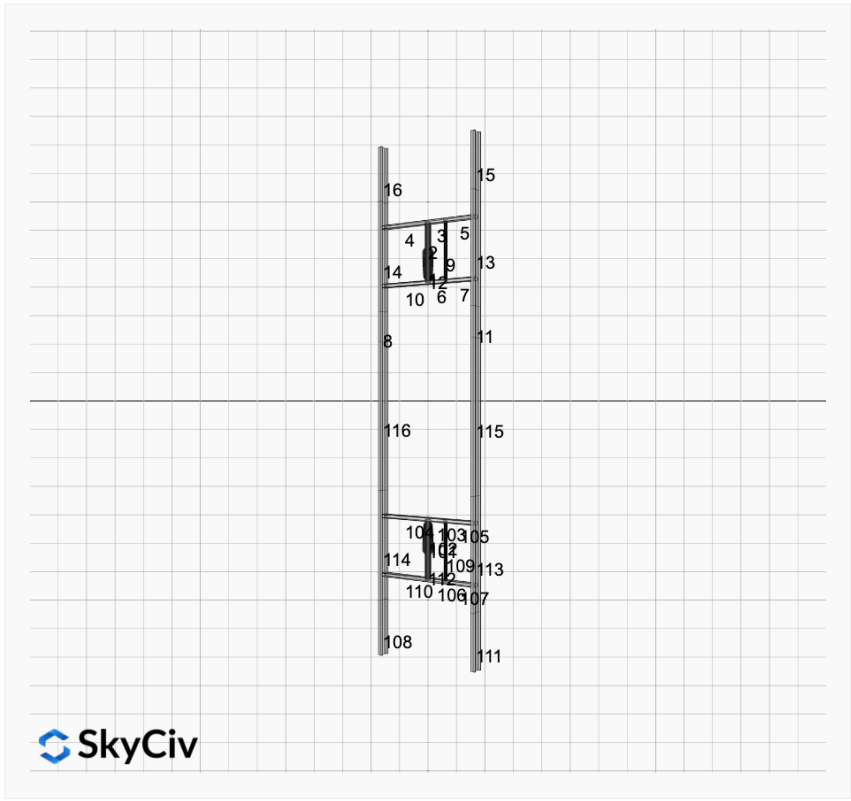
AutoDesigner Input

```
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","project_id":"Wachtell 30 CU","site_address":"2733 E Warm Springs Ave, Boise, ID 83712, USA","module_width":40,"module_length":72,"number_rows":5,"number_columns":6,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":15,"tilt_angle":35,"ground_clearance":8,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"clay","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":true}
```

Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles

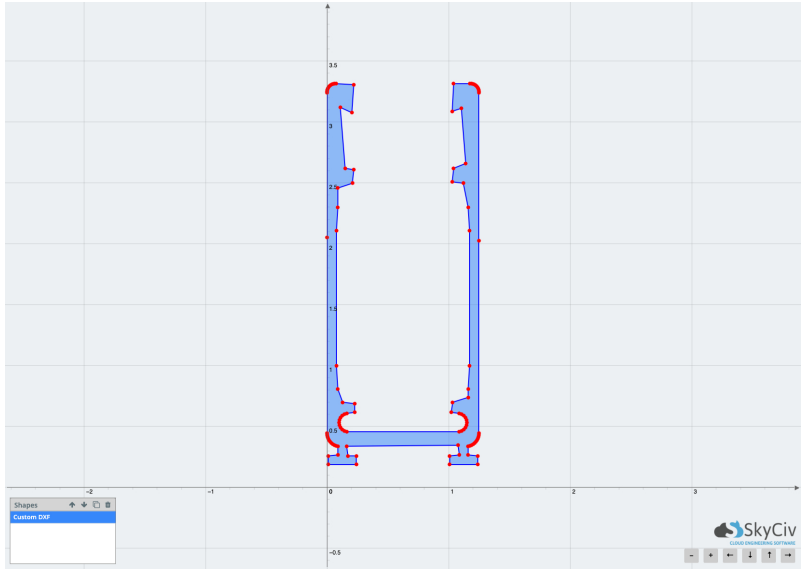






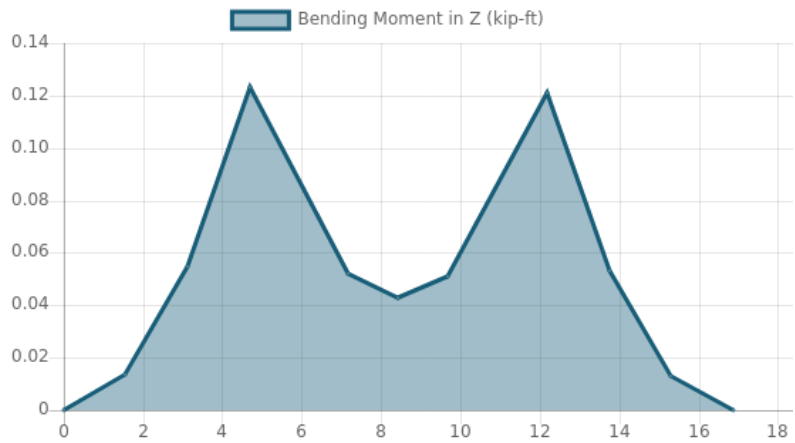
Rail Design Check

Rail Length: 16.875 ft
Additional Restraints Required: None
Tributary Width: 3.041666666666665 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0550 kip/ft
Snow (Y): -0.0385 kip/ft
Wind uplift Case A: 0.0703 kip/ft
Wind uplift Case A: 0.0703 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0950 kip/ft

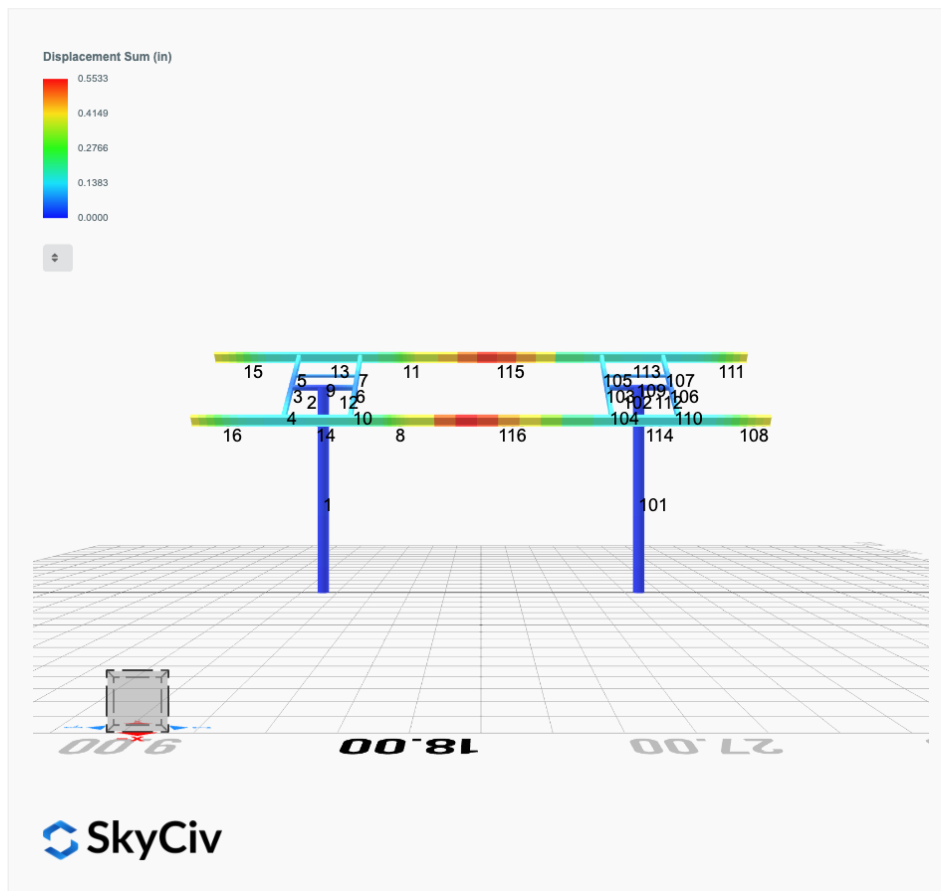


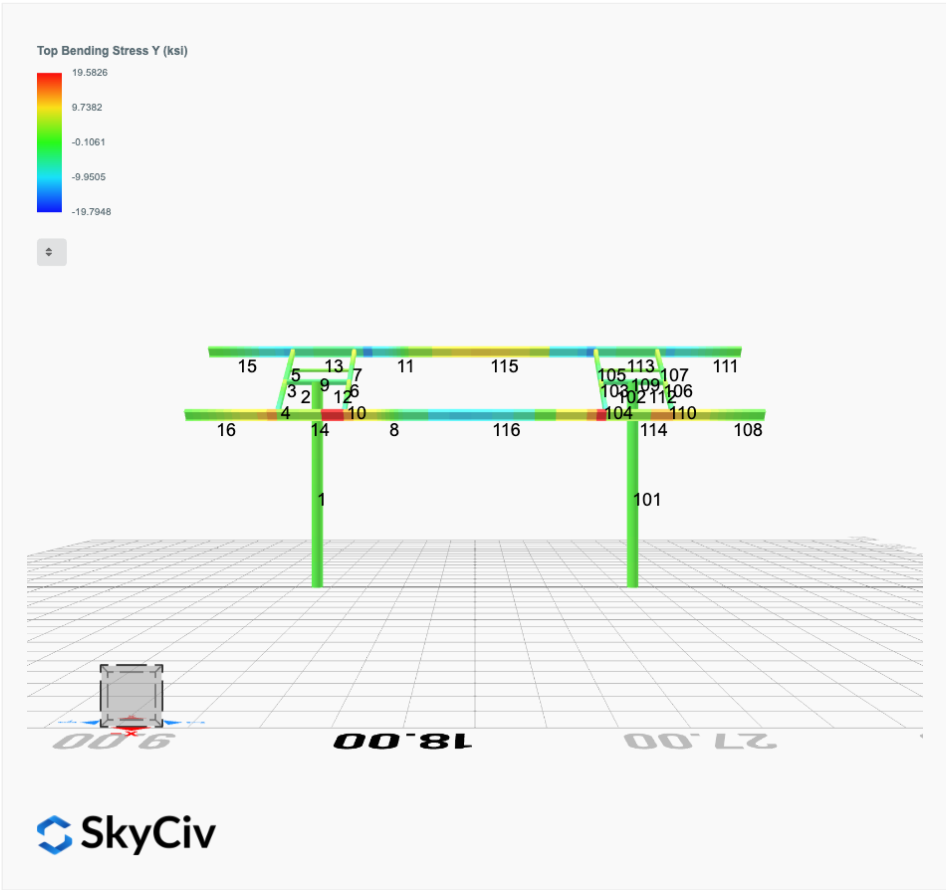
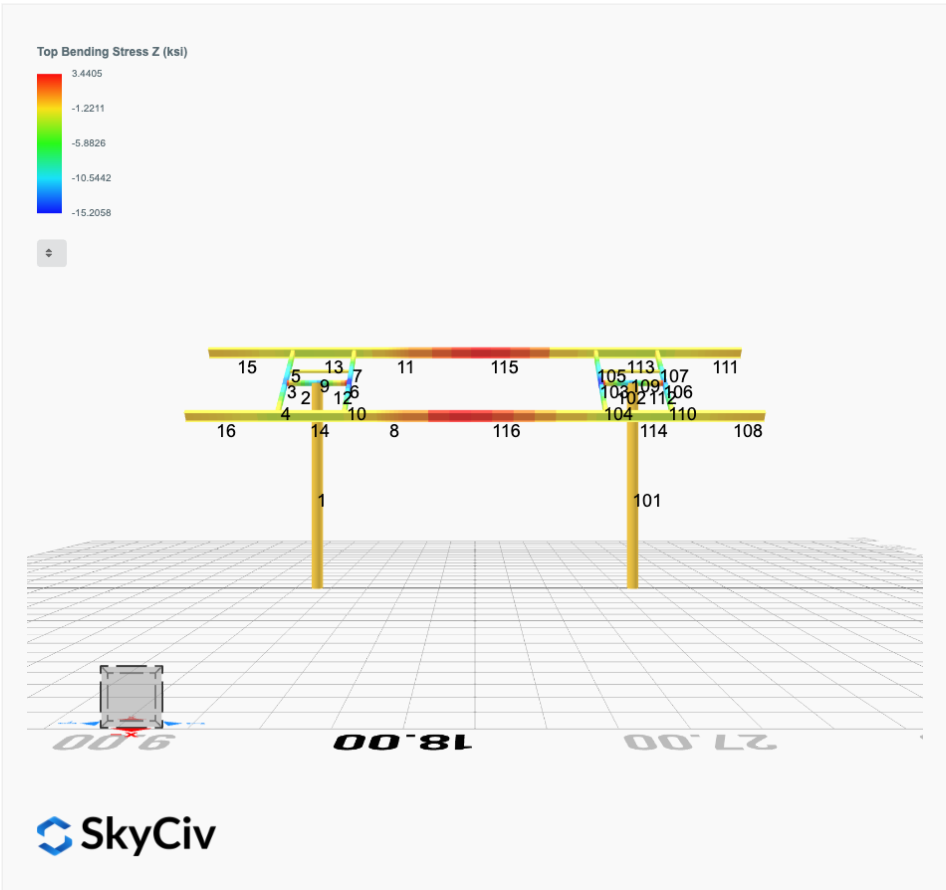
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	27.48667598	0.797	PASS
Material Yield	34.5	27.48667598	0.797	PASS
Material Strength	37	27.48667598	0.743	PASS

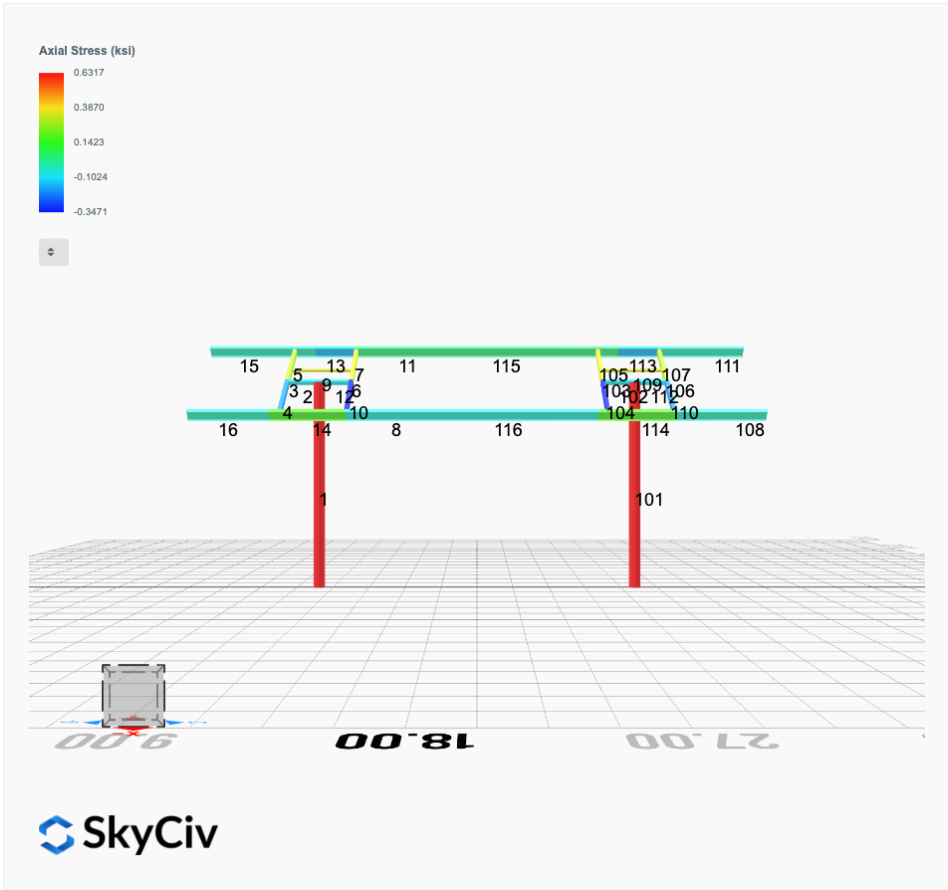
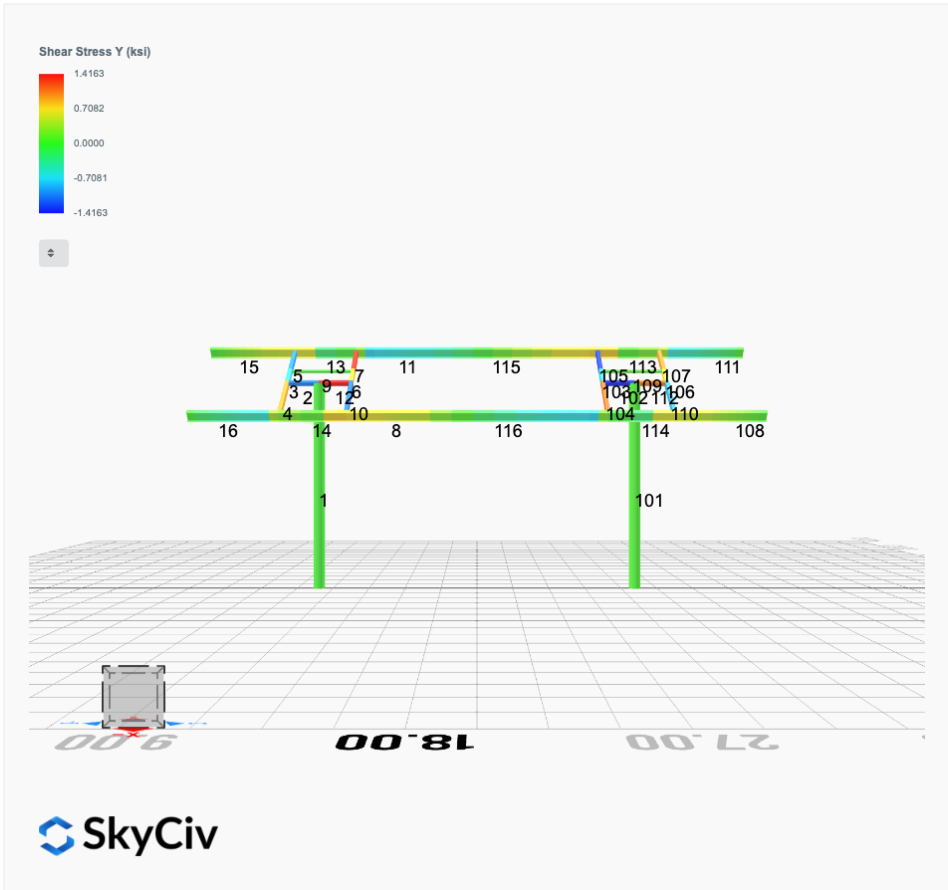
Member 1, ULS: 1. 1.4D



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	0.0000	2.3406	0.0300	0.1199	-0.0150	0.0261
ULS: 2. D + L	0.0000	2.3406	0.0300	0.1199	-0.0150	0.0261
ULS: 3. D + (S or Lr or R)	0.0000	7.6466	0.1212	0.4846	-0.0613	0.0612
ULS: 3. D + (S or Lr or R)	0.0000	2.3406	0.0300	0.1199	-0.0150	0.0261
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.3201	0.0984	0.3934	-0.0497	0.0525
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.3406	0.0300	0.1199	-0.0150	0.0261
ULS: 5b. D + 0.7E	0.0000	2.3406	0.0300	0.1199	-0.0150	0.0261
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.3201	0.0984	0.3934	-0.0497	0.0525
ULS: 8. 0.6D + 0.7E	0.0000	1.4044	0.0180	0.0719	-0.0090	0.0157
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9015	6.4843	0.1115	0.4324	-0.2533	38.5875
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.9015	6.4843	0.1115	0.4324	-0.2533	38.5875
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.4481	-1.1556	-0.0384	-0.1416	0.1859	-30.7347
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.0401	-0.5729	-0.0275	-0.1000	0.1538	-33.7759
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1761	9.4279	0.1594	0.6278	-0.2285	28.9735
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1761	9.4279	0.1594	0.6278	-0.2285	28.9735
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8361	3.6979	0.0471	0.1973	0.1009	-23.0181
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.5301	4.1349	0.0552	0.2285	0.0769	-25.2990
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1761	5.4484	0.0911	0.3543	-0.1937	28.9472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1761	5.4484	0.0911	0.3543	-0.1937	28.9472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8361	-0.2816	-0.0213	-0.0762	0.1357	-23.0445
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.5301	0.1555	-0.0131	-0.0450	0.1116	-25.3254
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9015	5.5481	0.0994	0.3844	-0.2473	38.5771
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.9015	5.5481	0.0994	0.3844	-0.2473	38.5771
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.4481	-2.0919	-0.0504	-0.1895	0.1919	-30.7451
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.0401	-1.5092	-0.0395	-0.1479	0.1598	-33.7864

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.7514
Shear X	-4.8358
Shear Z	0.2500
Moment X	0.9930
Moment Y (Twist)	0.4368
Moment Z	65.8076

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	9.4279
Shear X	-2.9015
Shear Z	0.1594
Moment X	0.6278
Moment Y (Twist)	0.2533
Moment Z	38.5875

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	2.3406	-0.0300	-0.1199	0.0150	0.0261
ULS: 2. D + L	-0.0000	2.3406	-0.0300	-0.1199	0.0150	0.0261
ULS: 3. D + (S or Lr or R)	-0.0000	7.6466	-0.1212	-0.4846	0.0614	0.0613
ULS: 3. D + (S or Lr or R)	-0.0000	2.3406	-0.0300	-0.1199	0.0150	0.0261
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	6.3201	-0.0984	-0.3935	0.0498	0.0525
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.3406	-0.0300	-0.1199	0.0150	0.0261
ULS: 5b. D + 0.7E	-0.0000	2.3406	-0.0300	-0.1199	0.0150	0.0261

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	6.3201	-0.0984	-0.3935	0.0498	0.0525
ULS: 8. 0.6D + 0.7E	-0.0000	1.4044	-0.0180	-0.0719	0.0090	0.0157
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9015	6.4843	-0.1115	-0.4324	0.2533	38.5876
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.9015	6.4843	-0.1115	-0.4324	0.2533	38.5876
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.4481	-1.1556	0.0384	0.1416	-0.1859	-30.7346
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.0401	-0.5729	0.0275	0.1000	-0.1538	-33.7759
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1761	9.4279	-0.1594	-0.6278	0.2285	28.9736
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1761	9.4279	-0.1594	-0.6278	0.2285	28.9736
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8361	3.6979	-0.0471	-0.1974	-0.1008	-23.0180
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.5301	4.1349	-0.0552	-0.2286	-0.0768	-25.2990
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1761	5.4484	-0.0911	-0.3543	0.1937	28.9472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1761	5.4484	-0.0911	-0.3543	0.1937	28.9472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8361	-0.2816	0.0213	0.0762	-0.1357	-23.0445
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.5301	0.1555	0.0131	0.0450	-0.1116	-25.3254
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9015	5.5481	-0.0994	-0.3844	0.2473	38.5771
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.9015	5.5481	-0.0994	-0.3844	0.2473	38.5771
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.4481	-2.0919	0.0504	0.1895	-0.1919	-30.7451
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.0401	-1.5092	0.0395	0.1479	-0.1598	-33.7864

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.7513
Shear X	-4.8358
Shear Z	-0.2500
Moment X	-0.9933
Moment Y (Twist)	0.4368
Moment Z	65.8087

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	9.4279
Shear X	-2.9015
Shear Z	-0.1594
Moment X	-0.6278
Moment Y (Twist)	0.2533
Moment Z	38.5876

Project Details

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

User Name: sales@mtsolar.us
 Project Name: Wachtell 30 CU
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F _y (ksi)	F _u (ksi)
1	29000	50	65

Section Dimensions

ID	Name	d (in)	t _w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
9	8in Pipe Sch 40	8.63	0.32				

ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
16	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

9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D
1	9	26.96	26.96	12.84	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.59,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.71,1.67,1.67,1.66,1.64	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67	300	200	1
8	19	1.33	1.33	2.05	1.66,1.66,1.66,1.66,1.66,1.66,1.64,1.64,1.61,1.87,1.64,1.64,1.63,1.38,1.65,1.65,1.68,1.68,1.64,1.64,1.62,1.96,1.64,1.64,1.63,1.47	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.62,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.72,1.67,1.67,1.66,1.65	300	200	1
11	19	1.33	1.33	2.05	1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.65,1.61,1.66,1.65,1.65,1.64,1.66,1.66,1.66,1.69,1.68,1.65,1.65,1.62,1.66,1.65,1.65,1.64,1.66	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.11,1.11,1.11,1.12,1.11,1.11,1.12,1.12,1.13,1.15,1.12,1.12,1.12,1.14,1.12,1.12,1.11,1.09,1.12,1.12,1.13,1.15,1.12,1.12,1.12,1.14	300	200	1
14	19	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.12,1.12,1.32,1.12,1.12,1.12,1.06,1.11,1.11,1.11,1.12,1.12,1.12,1.12,1.45,1.12,1.12,1.12,1.04	300	200	1
15	19	7.88	7.88	3.75	2.33,2.33	300	200	1
16	19	7.88	7.88	3.75	2.33,2.33	300	200	1
101	9	26.96	26.96	12.84	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
104	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.62,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.72,1.67,1.67,1.66,1.65	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.67	300	200	1
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.67	300	200	1

115	133.20	69.16	17.33	6.12	40.24	43.62
116	133.20	69.16	16.96	6.12	40.24	43.62

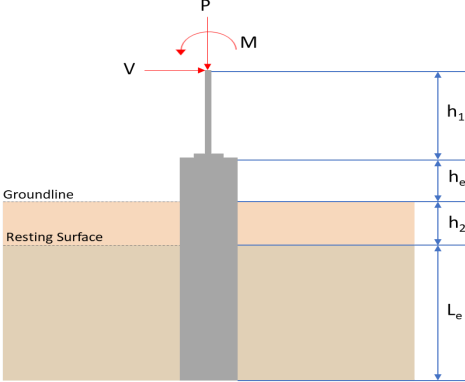
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.095	0.790	0.027	0.043	0.002	0.840	#13	0.551	Not Required	Pass
2	0.004	0.480	0.215	0.110	0.041	0.639	#21	0.035	Not Required	Pass
3	0.012	0.739	0.055	0.073	0.003	0.798	#21	0.045	Not Required	Pass
4	0.011	0.724	0.186	0.073	0.040	0.861	#21	0.080	Not Required	Pass
5	0.011	0.459	0.180	0.073	0.045	0.505	#21	0.074	Not Required	Pass
6	0.015	0.828	0.110	0.083	0.022	0.946	#21	0.045	Not Required	Pass
7	0.015	0.514	0.251	0.082	0.063	0.581	#21	0.074	Not Required	Pass
8	0.002	0.109	0.215	0.056	0.025	0.224	#21	0.095	Not Required	Pass
9	0.017	0.067	0.087	0.002	0.003	0.152	#21	0.204	Not Required	Pass
10	0.016	0.813	0.236	0.081	0.050	0.970	#21	0.080	Not Required	Warn
11	0.003	0.110	0.222	0.057	0.025	0.233	#21	0.095	Not Required	Pass
12	0.003	0.581	0.235	0.130	0.042	0.753	#21	0.035	Not Required	Pass
13	0.008	0.247	0.573	0.073	0.032	0.766	#21	0.286	Not Required	Pass
14	0.009	0.249	0.565	0.072	0.032	0.753	#21	0.190	Not Required	Pass
15	0.000	0.080	0.201	0.035	0.015	0.281	#21	Not Required	Not Required	Pass
16	0.000	0.079	0.201	0.034	0.015	0.280	#21	Not Required	Not Required	Pass
101	0.095	0.790	0.027	0.043	0.002	0.840	#13	0.551	Not Required	Pass
102	0.003	0.581	0.235	0.130	0.042	0.753	#21	0.035	Not Required	Pass
103	0.015	0.828	0.110	0.083	0.022	0.946	#21	0.045	Not Required	Pass
104	0.016	0.813	0.236	0.081	0.050	0.970	#21	0.080	Not Required	Warn
105	0.015	0.514	0.251	0.082	0.063	0.581	#21	0.074	Not Required	Pass
106	0.012	0.739	0.055	0.073	0.003	0.798	#21	0.045	Not Required	Pass
107	0.011	0.459	0.180	0.073	0.045	0.505	#21	0.074	Not Required	Pass
108	0.000	0.079	0.201	0.034	0.015	0.280	#21	Not Required	Not Required	Pass
109	0.017	0.067	0.087	0.002	0.003	0.152	#21	0.204	Not Required	Pass
110	0.011	0.724	0.186	0.073	0.040	0.861	#21	0.080	Not Required	Pass
111	0.000	0.080	0.201	0.035	0.015	0.281	#21	Not Required	Not Required	Pass
112	0.004	0.480	0.215	0.110	0.041	0.639	#21	0.035	Not Required	Pass
113	0.008	0.247	0.573	0.073	0.032	0.766	#21	0.190	Not Required	Pass
114	0.009	0.249	0.565	0.072	0.032	0.754	#21	0.286	Not Required	Pass
115	0.006	0.385	0.319	0.057	0.025	0.706	#21	0.473	Not Required	Pass
116	0.002	0.382	0.319	0.056	0.025	0.702	#21	0.473	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 _n	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 $L = 8.25$ 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="421 1102 1190 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>Clay, sandy clay, silty clay, clayey silt, silt and sandy silt</td> <td>1500.000</td> <td>100.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1460"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.428</td> <td>14.751</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.901</td> <td>-4.836</td> </tr> <tr> <td>V_z (kip)</td> <td>0.159</td> <td>0.250</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.628</td> <td>0.993</td> </tr> <tr> <td>M_z (kipft)</td> <td>38.588</td> <td>65.808</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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000	Load Component	ASD	LRFD	P (kip)	9.428	14.751	V_x (kip)	-2.901	-4.836	V_z (kip)	0.159	0.250	M_x (kipft)	0.628	0.993	M_z (kipft)	38.588	65.808	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000																									
Load Component	ASD	LRFD																										
P (kip)	9.428	14.751																										
V_x (kip)	-2.901	-4.836																										
V_z (kip)	0.159	0.250																										
M_x (kipft)	0.628	0.993																										
M_z (kipft)	38.588	65.808																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.901 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.46194 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(38.588 \text{ kipft}) + ((-2.901 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.1446 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5174 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.159 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.025318 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.628 \text{ kipft}) + ((0.159 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.6192 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5174 \text{ ft}), (2.6192 \text{ ft})]$$

$$L_{e,req} = 7.517 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.517 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91115$$

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

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

$$q = \frac{(9.428 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58925 \text{ kip/ft}^2$$

$$q = 0.00920 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58925 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.39283$$

Status: **PASS**
Ratio: **0.390**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.46194 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.1446 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.1446 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (6.1446 \text{ kipft/ft})) + (4 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.7011 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.1446 \text{ kipft/ft})) + (3 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (6.1446 \text{ kipft/ft})) + (2 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.17612 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.1446 \text{ kipft/ft})) + ((-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.74738 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(5.7011 \text{ ft})}{2}$$

$$p_a = 0.28506 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17612 \text{ kip/ft}^2)}{(0.28506 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61783$$

p_a - Allowable lateral soil pressure at depth L_e ,

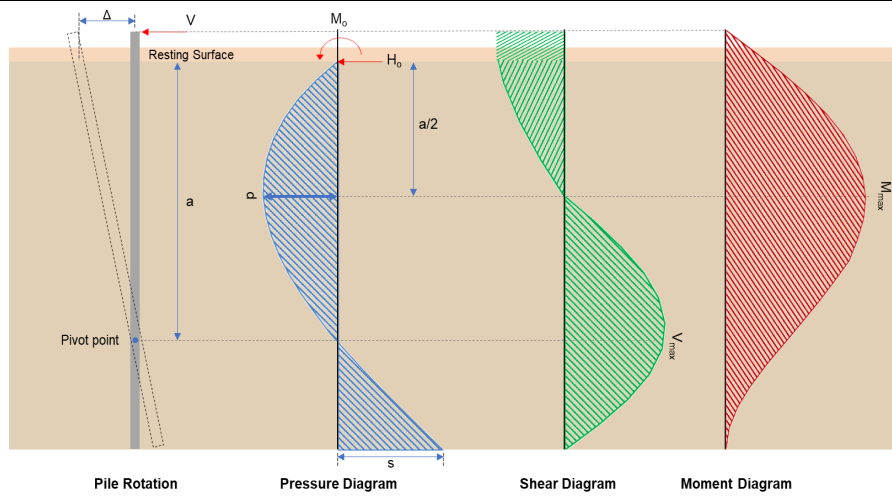
Status: **PASS**
Ratio: **0.620**

	$p_s = R L_e$ $p_s = (100 \text{ psf/ft}) \times (8.25 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74738 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.90592$	<p>Status: PASS Ratio: 0.910</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.025318 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.1 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.1 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.025318 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.1 \text{ kipft/ft})) + (4 \times (0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))}$ $a = 5.9001 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.1 \text{ kipft/ft})) + (3 \times (0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.1 \text{ kipft/ft})) + (2 \times (0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$ $p = 0.016181 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.1 \text{ kipft/ft})) + ((0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$ $s = 0.036044 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (100 \text{ psf/ft}) \times \frac{(5.9001 \text{ ft})}{2}$ $p_a = 0.29501 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.016181 \text{ kip/ft}^2)}{(0.29501 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.05485$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (100 \text{ psf/ft}) \times (8.25 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.050</p>

$$\text{Ratio} = \frac{(0.036044 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.04369$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.836 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.77006 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(65.808 \text{ kipft}) + ((-4.836 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.479 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.479 \text{ kipft/ft})}{(-0.77006 \text{ kip/ft})}$$

$$E = 13.608 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.479 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.77006 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (10.479 \text{ kipft/ft})) + (4 \times (-0.77006 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6979 \text{ ft}$$

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

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

$$V_{max} = ((-0.77006 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6979 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6979 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.022 \text{ kip}$$

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

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

$$M_{max} = ((-0.77006 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(13.608 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6979 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6979 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6979 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 43.158 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.25 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.039809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.993 \text{ kipft}) + ((0.25 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15812 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15812 \text{ kipft/ft})}{(0.039809 \text{ kip/ft})}$$

$$E = 3.972 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15812 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.039809 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.15812 \text{ kipft/ft})) + (4 \times (0.039809 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8992 \text{ ft}$$

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

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

$$V_{max} = ((0.039809 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8992 \text{ ft})}{(8.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8992 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.24181 \text{ kip}$$

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

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

$$M_{max} = ((0.039809 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(3.972 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.8992 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8992 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8992 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.88037 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.751 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.106 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.106 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.751 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.005514$$

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

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.751 \text{ kip} \rightarrow 14751 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14751 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.45 \text{ kip}$$

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

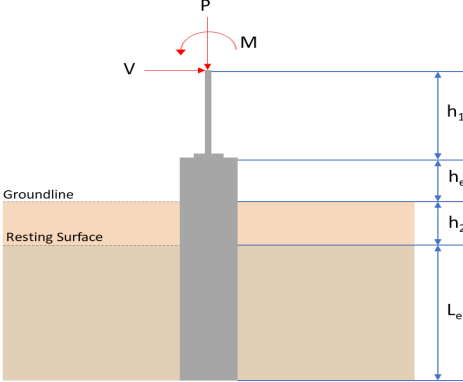
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.45 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.45 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.45 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.37 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.022 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.022 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.09896$ <p>Considering z-direction:</p> <p>$V_{max} = 0.24181 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.24181 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.0021712$	<p>Status: PASS Ratio: 0.100</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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <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: $M_{max} = 43.158 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(43.158 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.17291$	<p>Status: PASS Ratio: 0.170</p>
	<p>Considering z-direction: $M_{max} = 0.88037 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.88037 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0035271$	<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 $L = 8.25$ 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="421 1102 1189 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>Clay, sandy clay, silty clay, clayey silt, silt and sandy silt</td> <td>1500.000</td> <td>100.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.428</td> <td>14.751</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.901</td> <td>-4.836</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.159</td> <td>-0.250</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.628</td> <td>-0.993</td> </tr> <tr> <td>M_z (kipft)</td> <td>38.588</td> <td>65.809</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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000	Load Component	ASD	LRFD	P (kip)	9.428	14.751	V_x (kip)	-2.901	-4.836	V_z (kip)	-0.159	-0.250	M_x (kipft)	-0.628	-0.993	M_z (kipft)	38.588	65.809	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000																									
Load Component	ASD	LRFD																										
P (kip)	9.428	14.751																										
V_x (kip)	-2.901	-4.836																										
V_z (kip)	-0.159	-0.250																										
M_x (kipft)	-0.628	-0.993																										
M_z (kipft)	38.588	65.809																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.901 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.46194 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(38.588 \text{ kipft}) + ((-2.901 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.1446 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5174 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.159 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.025318 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.628 \text{ kipft}) + ((-0.159 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9603 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5174 \text{ ft}), (1.9603 \text{ ft})]$$

$$L_{e,req} = 7.517 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.517 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91115$$

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

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

$$q = \frac{(9.428 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58925 \text{ kip/ft}^2$$

$$q = 0.00920 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58925 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.39283$$

Status: **PASS**
Ratio: **0.390**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.46194 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.1446 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.1446 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (6.1446 \text{ kipft/ft})) + (4 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.7011 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.1446 \text{ kipft/ft})) + (3 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (6.1446 \text{ kipft/ft})) + (2 \times (-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.17612 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.1446 \text{ kipft/ft})) + ((-0.46194 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.74738 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(5.7011 \text{ ft})}{2}$$

$$p_a = 0.28506 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17612 \text{ kip/ft}^2)}{(0.28506 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61783$$

p_a - Allowable lateral soil pressure at depth L_e ,

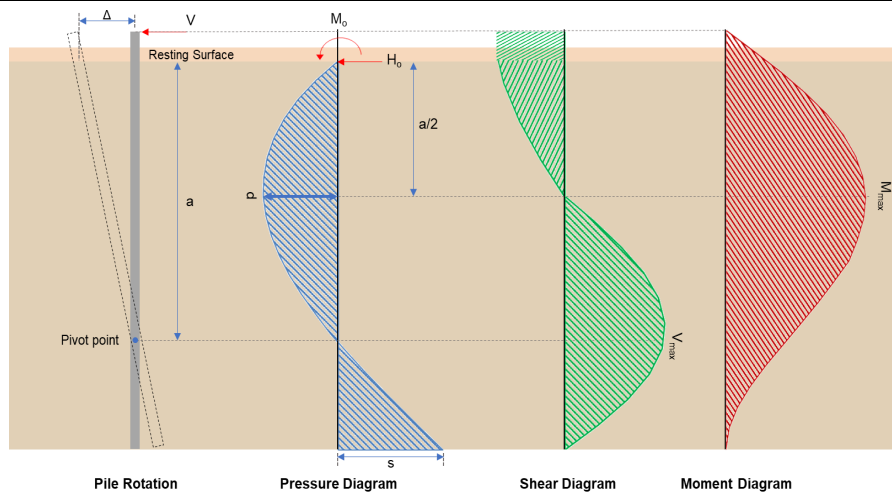
Status: **PASS**
Ratio: **0.620**

	$p_s = R L_e$ $p_s = (100 \text{ psf/ft}) \times (8.25 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74738 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.90592$	<p>Status: PASS Ratio: 0.910</p>
	<p>Considering z-direction:</p> <p>$H_o = -0.025318 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.1 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.1 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.025318 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.1 \text{ kipft/ft})) + (4 \times (-0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))}$ $a = 5.9001 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.1 \text{ kipft/ft})) + (3 \times (-0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.1 \text{ kipft/ft})) + (2 \times (-0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$ $p = -0.0048064 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.1 \text{ kipft/ft})) + ((-0.025318 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$ $s = -0.00078258 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (100 \text{ psf/ft}) \times \frac{(5.9001 \text{ ft})}{2}$ $p_a = 0.29501 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0048064 \text{ kip/ft}^2)}{(0.29501 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.016292$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (100 \text{ psf/ft}) \times (8.25 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: -0.020</p>

$$\text{Ratio} = \frac{(-0.00078258 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.00094858$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.836 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.77006 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(65.809 \text{ kipft}) + ((-4.836 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.479 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.479 \text{ kipft/ft})}{(-0.77006 \text{ kip/ft})}$$

$$E = 13.608 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.479 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.77006 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (10.479 \text{ kipft/ft})) + (4 \times (-0.77006 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6979 \text{ ft}$$

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

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

$$V_{max} = ((-0.77006 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6979 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6979 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.022 \text{ kip}$$

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

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

$$M_{max} = ((-0.77006 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(13.608 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6979 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6979 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.608 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6979 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 43.159 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.25 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.039809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.993 \text{ kipft}) + ((-0.25 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15812 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15812 \text{ kipft/ft})}{(-0.039809 \text{ kip/ft})}$$

$$E = 3.972 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15812 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.039809 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.15812 \text{ kipft/ft})) + (4 \times (-0.039809 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8992 \text{ ft}$$

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

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

$$V_{max} = ((-0.039809 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8992 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8992 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.24181 \text{ kip}$$

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

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

$$M_{max} = ((-0.039809 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(3.972 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.8992 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8992 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.972 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8992 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.88037 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.751 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.106 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.106 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.751 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.005514$$

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

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.751 \text{ kip} \rightarrow 14751 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14751 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.45 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.45 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.45 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.45 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.37 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.022 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.022 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.098961$ <p>Considering z-direction:</p> <p>$V_{max} = 0.24181 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.24181 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.0021712$	<p>Status: PASS Ratio: 0.100</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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <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: $M_{max} = 43.159 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(43.159 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.17291$	<p>Status: PASS Ratio: 0.170</p>
	<p>Considering z-direction: $M_{max} = 0.88037 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.88037 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0035271$	<p>Status: PASS Ratio: 0.000</p>