

Your Project Calculations



Project Name: WilsonResidence CU C

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=WilsonResidence%20CU%20C&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=58a5UEXt7JAPwqHCBthyOIN2KcTNG0jZZt0xLNpOay5JSLTuHxSpCtdk6y7YNI

Array Specification

Product:	Beam
Unique ID:	3P-17-6TOP-SD-12-L-5Hx6W-A6JH
Duty Classification:	SD
Module Width:	44.63 in
Module Length:	89.25in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Desired Tilt Angle:	46
Front Edge Clearance:	3
Total Array Height at Tilt:	16.45 ft
Total Frame Length:	43.50 ft
Frame Weight:	2076 lbs
Array Dimensions N/S:	18.80 ft
Array Dimensions E/W:	45.13 ft
Rail Length:	225.63 in
Rail Spacing:	3.72 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	9.76 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.75 ft Pile 3: 6.25 ft
Foundation Volume:	11.407 y ³
Foundation Result:	PASSED
Mount Twist:	1.721053 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	18680 Gunsmoke Rd, Yucca, AZ 86438, USA
Wind Speed:	115 mph
Snow Load:	10 psf
Design Uplift Pressure:	0.022931 ksf
Design Downforce Pressure:	-0.022931 ksf
Design Snow Pressure:	0.002639 ksf



Design Disclaimer

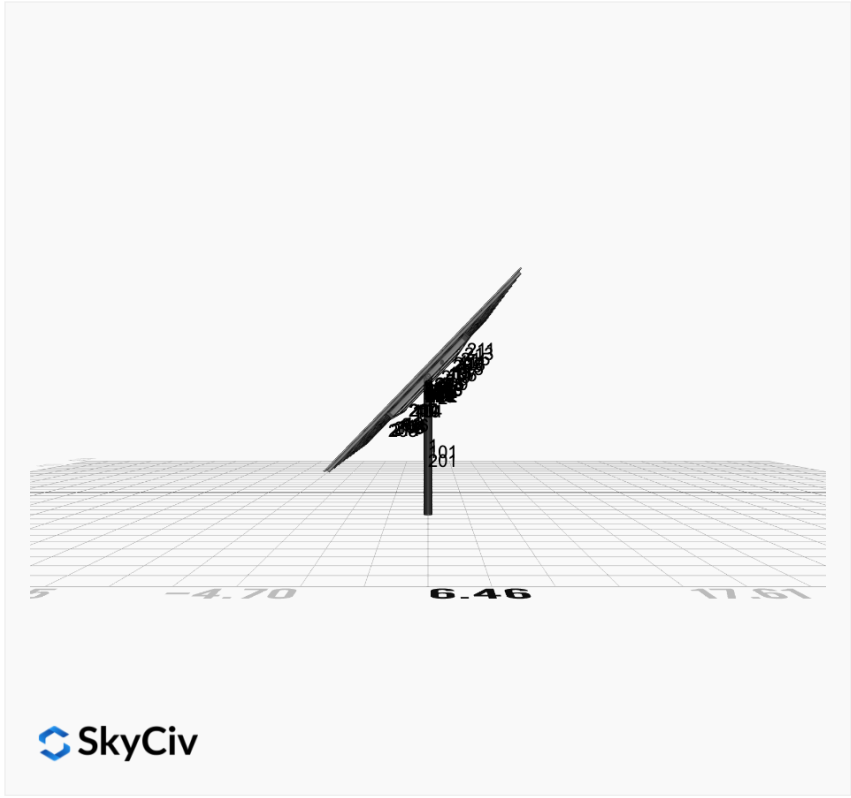
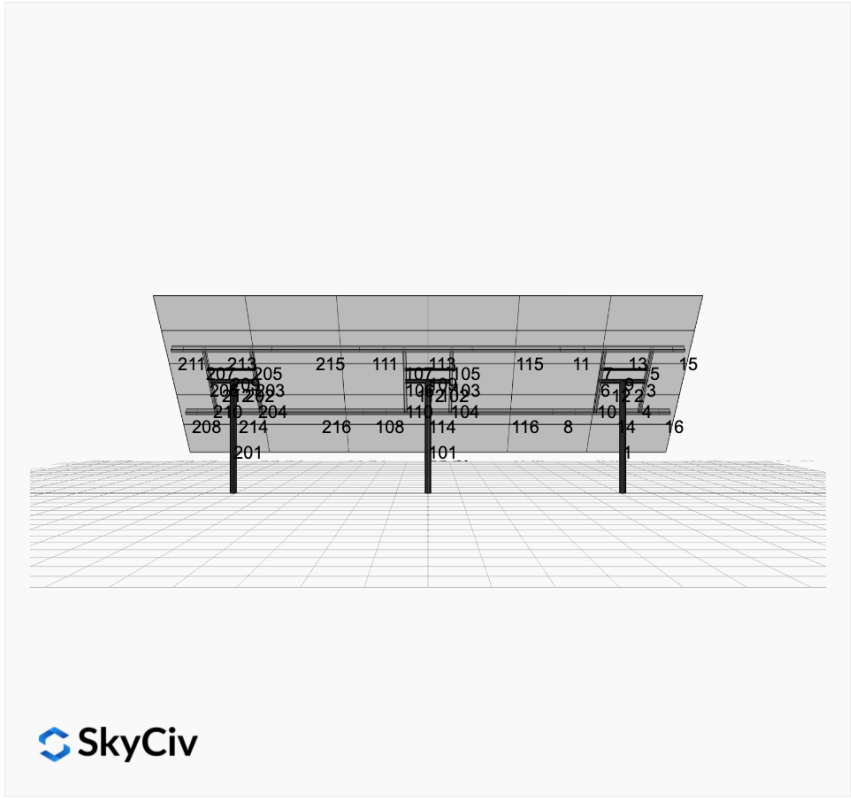
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

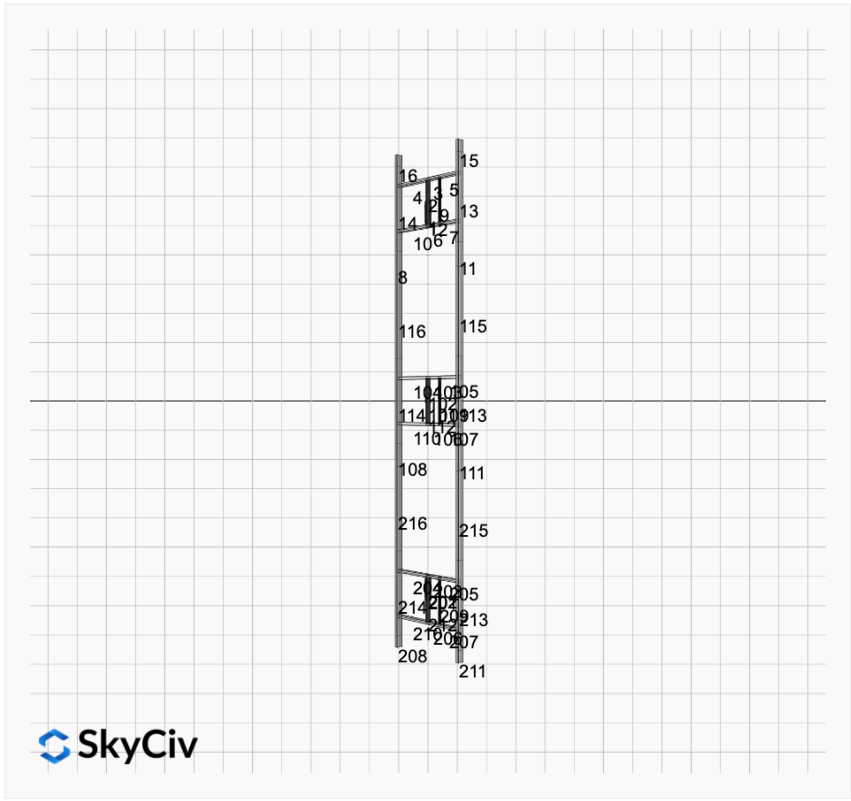
AutoDesigner Input

```
{"product_type": "Beam", "project_id": "WilsonResidence CU C", "site_address": "18680 Gunsmoke Rd, Yucca, AZ 86438, USA", "module_width": 44.625, "module_length": 89.25, "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_L2GA", "pole_spacing": 15, "tilt_angle": 46, "ground_clearance": 3, "risk_category": "I", "exposure_category": "C", "frame_duty_override": "auto", "pole_override": "auto", "soil_type": "sand", "customer_foundation_override": "48_Square", "foundation_type": "Square", "foundation_size": 48, "isolated_foundation_thickness": 18, "check_rails": false, "wind_speed_override": 115, "snow_load_override": 10, "direct_snow_load": false, "add_angle_brace": false}
```

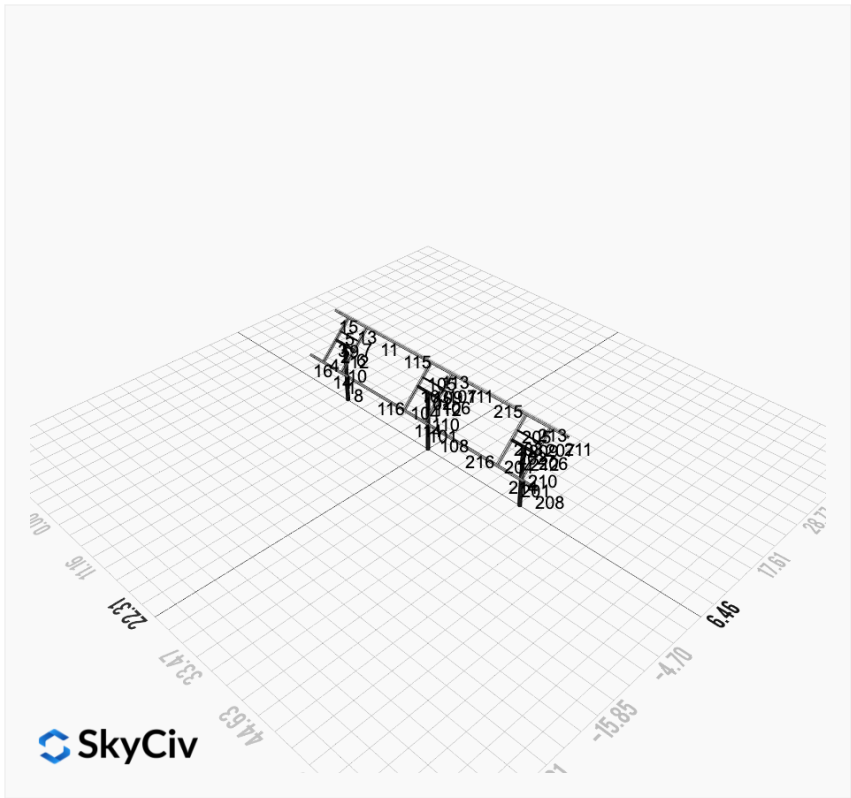
Design Notes:

- 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

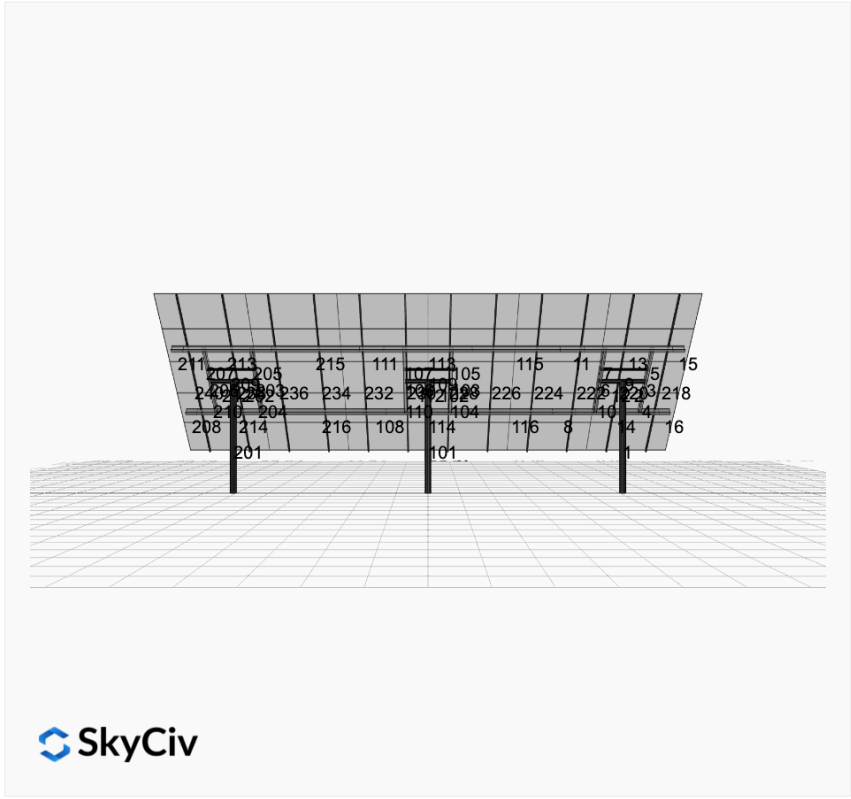




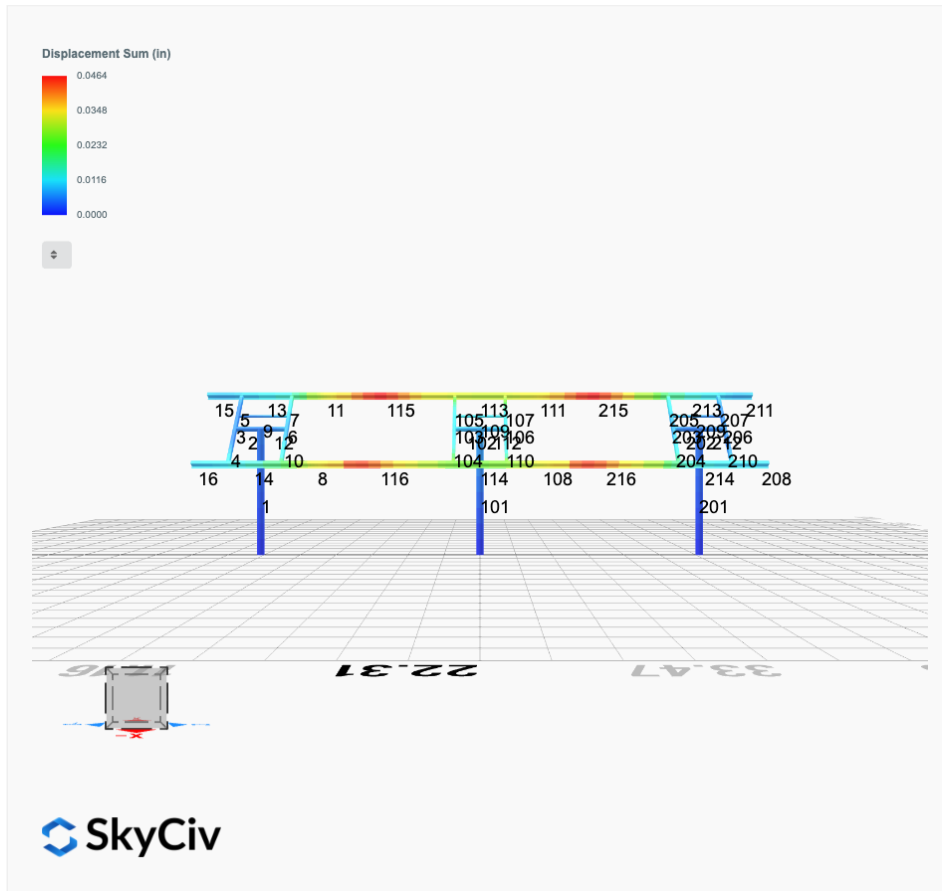
 SkyCiv

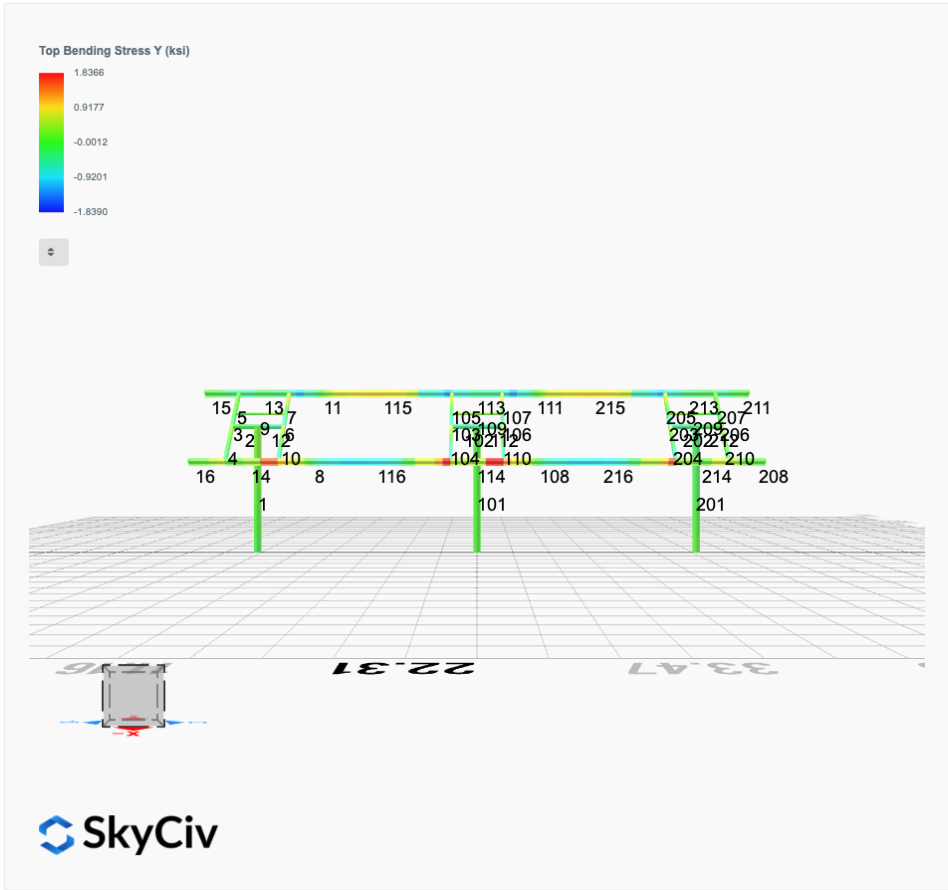
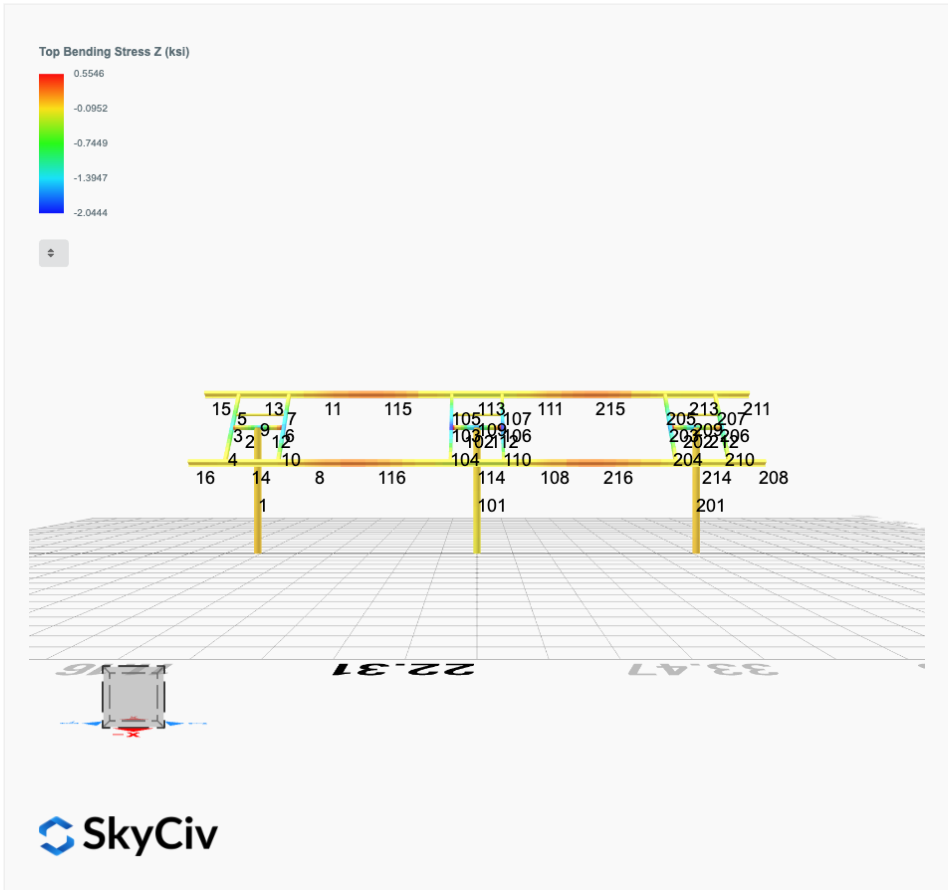


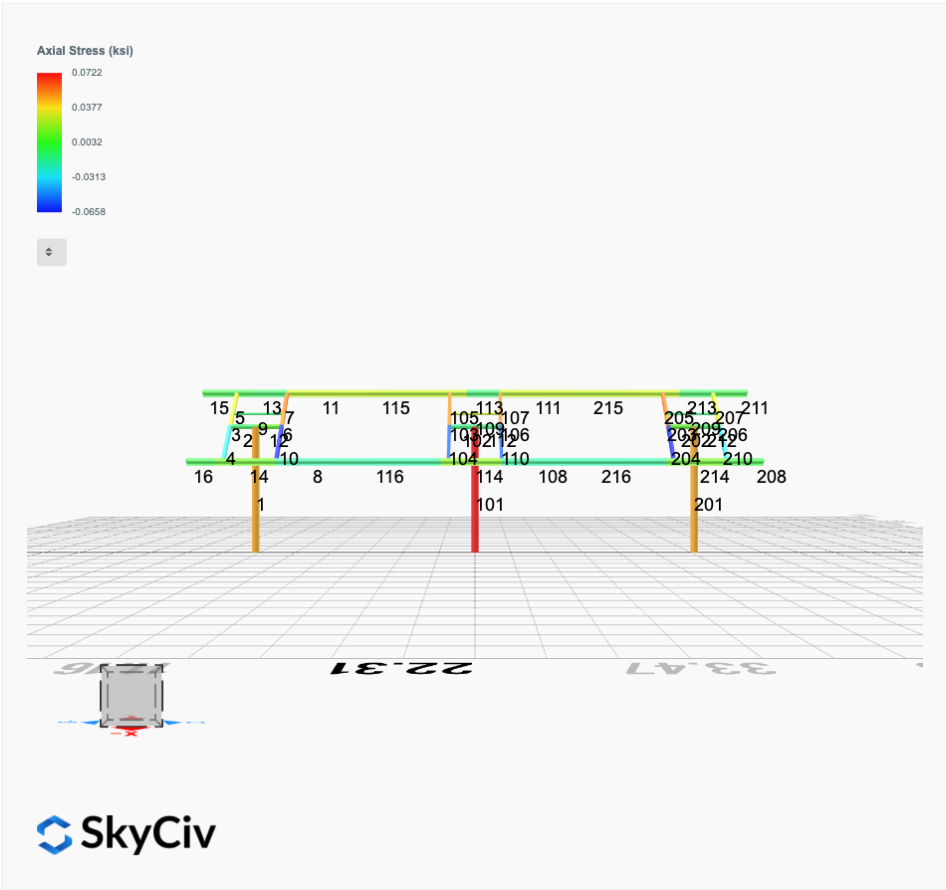
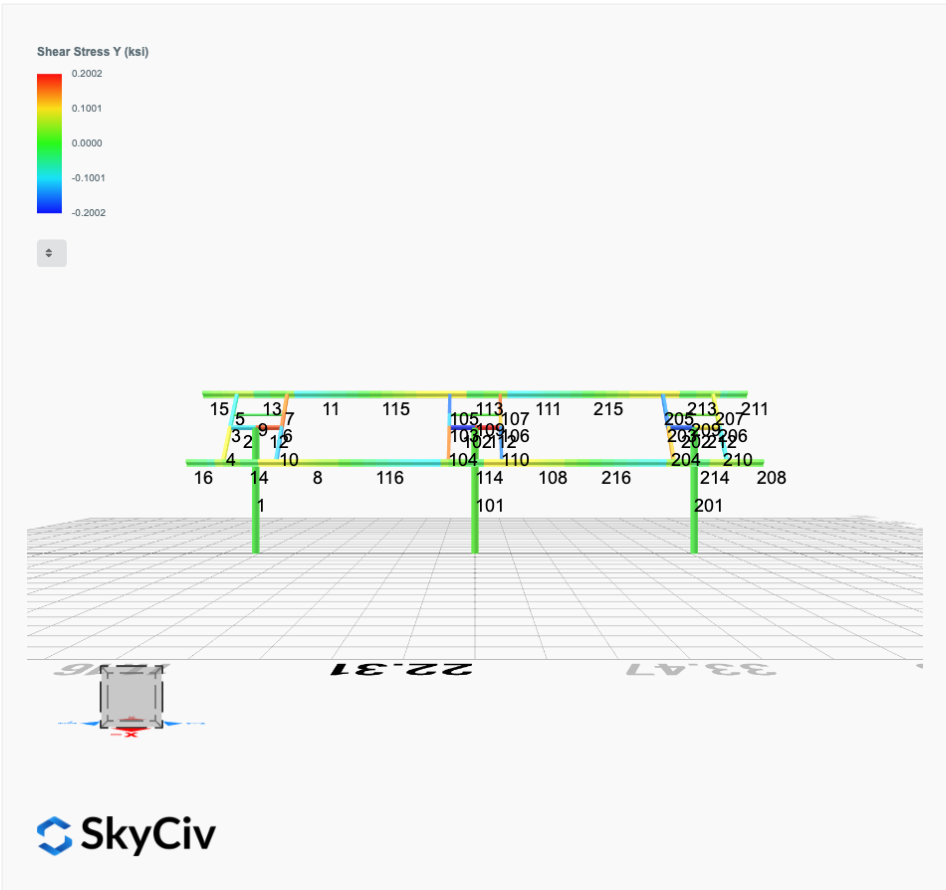
 SkyCiv



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	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 2. D + L	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 3. D + (S or Lr or R)	0.0322	2.2266	0.1101	0.3155	-0.1189	-0.2584
ULS: 3. D + (S or Lr or R)	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0302	2.1150	0.1033	0.2959	-0.1115	-0.2413
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 5b. D + 0.7E	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0302	2.1150	0.1033	0.2959	-0.1115	-0.2413
ULS: 8. 0.6D + 0.7E	0.0145	1.0683	0.0497	0.1423	-0.0536	-0.1142
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5204	4.1516	0.3433	0.9395	-1.0429	25.3710
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5616	-0.5879	-0.1710	-0.4458	0.8399	-24.8784
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8782	3.8935	0.2987	0.8226	-0.8267	18.9296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0302	2.1150	0.1033	0.2959	-0.1115	-0.2413
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9333	0.3388	-0.0871	-0.2163	0.5854	-18.7574
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0302	2.1150	0.1033	0.2959	-0.1115	-0.2413
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8842	3.5588	0.2782	0.7639	-0.8045	18.9807
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9273	0.0042	-0.1076	-0.2750	0.6076	-18.7064
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0242	1.7804	0.0828	0.2372	-0.0894	-0.1903
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5301	3.4395	0.3102	0.8446	-1.0071	25.4472
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0145	1.0683	0.0497	0.1423	-0.0536	-0.1142
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5520	-1.3001	-0.2041	-0.5407	0.8756	-24.8023
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0145	1.0683	0.0497	0.1423	-0.0536	-0.1142

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	6.3126
Shear X	-4.2651
Shear Z	0.5500
Moment X	1.5028
Moment Y (Twist)	1.7211
Moment Z	42.8252

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	4.1516
Shear X	-2.5616
Shear Z	0.3433
Moment X	0.9395
Moment Y (Twist)	1.0429
Moment Z	25.4472

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	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 2. D + L	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 3. D + (S or Lr or R)	-0.0644	2.8756	0.0000	0.0000	0.0000	0.5717
ULS: 3. D + (S or Lr or R)	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0604	2.7238	0.0000	0.0000	0.0000	0.5373
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 5b. D + 0.7E	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0604	2.7238	0.0000	0.0000	0.0000	0.5373
ULS: 8. 0.6D + 0.7E	-0.0290	1.3611	0.0000	0.0000	0.0000	0.2604
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.3563	5.6349	0.0000	-0.0000	0.0000	32.7862
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.2738	-1.1038	0.0000	-0.0000	0.0000	-30.7310
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.5413	5.2487	0.0000	-0.0000	0.0000	24.8014
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0604	2.7238	0.0000	0.0000	0.0000	0.5373
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4312	0.1946	0.0000	-0.0000	0.0000	-22.8365
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0604	2.7238	0.0000	0.0000	0.0000	0.5373
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.5293	4.7933	0.0000	-0.0000	0.0000	24.6981
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4432	-0.2608	0.0000	-0.0000	0.0000	-22.9398
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0484	2.2684	0.0000	0.0000	0.0000	0.4340
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.3369	4.7276	0.0000	-0.0000	0.0000	32.6126
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0290	1.3611	0.0000	0.0000	0.0000	0.2604
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.2931	-2.0112	0.0000	-0.0000	0.0000	-30.9046
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0290	1.3611	0.0000	0.0000	0.0000	0.2604

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	8.6347
Shear X	-5.5723
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	55.1529

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	5.6349
Shear X	-3.3563
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	32.7862

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	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 2. D + L	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 3. D + (S or Lr or R)	0.0322	2.2266	-0.1101	-0.3155	0.1189	-0.2584
ULS: 3. D + (S or Lr or R)	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0302	2.1150	-0.1033	-0.2959	0.1116	-0.2413
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 5b. D + 0.7E	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0302	2.1150	-0.1033	-0.2959	0.1116	-0.2413
ULS: 8. 0.6D + 0.7E	0.0145	1.0683	-0.0497	-0.1423	0.0536	-0.1142
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5204	4.1516	-0.3433	-0.9395	1.0429	25.3711
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5616	-0.5879	0.1710	0.4458	-0.8399	-24.8784
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8782	3.8935	-0.2987	-0.8226	0.8267	18.9296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0302	2.1150	-0.1033	-0.2959	0.1116	-0.2413
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9333	0.3388	0.0871	0.2163	-0.5854	-18.7574
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0302	2.1150	-0.1033	-0.2959	0.1116	-0.2413

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	-1.8842	3.5588	-0.2782	-0.7639	0.8045	18.9807
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9273	0.0042	0.1076	0.2750	-0.6076	-18.7064
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0242	1.7804	-0.0828	-0.2372	0.0894	-0.1903
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5301	3.4395	-0.3102	-0.8446	1.0071	25.4472
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0145	1.0683	-0.0497	-0.1423	0.0536	-0.1142
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5520	-1.3001	0.2041	0.5407	-0.8756	-24.8023
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0145	1.0683	-0.0497	-0.1423	0.0536	-0.1142

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	6.3126
Shear X	-4.2651
Shear Z	-0.5500
Moment X	-1.5028
Moment Y (Twist)	1.7211
Moment Z	42.8260

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	4.1516
Shear X	-2.5616
Shear Z	-0.3433
Moment X	-0.9395
Moment Y (Twist)	1.0429
Moment Z	25.4472

Project Details

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

User Name: sales@mtsolar.us
 Project Name: WilsonResidence CU C
 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)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
8	6in Pipe Sch 80	6.63	0.43				

ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	

ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

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)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31

Member Design Capacity

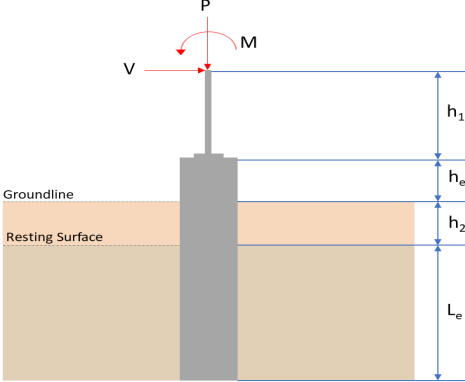
Member ID	$\Phi_c P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	378.22	150.94	62.23	62.23	113.47	113.47
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	21.30	6.45	30.09	45.74
14	120.60	98.23	22.71	6.45	30.09	45.74
15	120.60	113.97	23.36	6.45	30.09	45.74
16	120.60	113.97	23.36	6.45	30.09	45.74
101	378.22	150.94	62.23	62.23	113.47	113.47
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	18.66	6.45	30.09	45.74
114	120.60	98.23	18.49	6.45	30.09	45.74
115	120.60	89.27	18.56	6.45	30.09	45.74
116	120.60	89.27	19.97	6.45	30.09	45.74
201	378.22	150.94	62.23	62.23	113.47	113.47
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61
208	120.60	113.97	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	113.97	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	21.30	6.45	30.09	45.74
214	120.60	98.23	22.71	6.45	30.09	45.74
215	120.60	89.27	18.91	6.45	30.09	45.74
216	120.60	89.27	18.73	6.45	30.09	45.74

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	0.042	0.688	0.062	0.038	0.005	0.733	#13	0.560	Not Required	Pass
2	0.001	0.190	0.189	0.052	0.041	0.379	#13	0.052	Not Required	Pass
3	0.005	0.489	0.037	0.047	0.008	0.513	#13	0.044	Not Required	Pass
4	0.004	0.484	0.127	0.049	0.017	0.612	#13	0.078	Not Required	Pass
5	0.004	0.303	0.025	0.049	0.004	0.325	#13	0.073	Not Required	Pass
6	0.008	0.758	0.126	0.079	0.022	0.868	#13	0.044	Not Required	Pass
7	0.008	0.471	0.153	0.076	0.021	0.493	#13	0.073	Not Required	Pass
8	0.005	0.143	0.062	0.038	0.007	0.205	#13	0.088	Not Required	Pass
9	0.003	0.067	0.078	0.005	0.004	0.145	#13	0.198	Not Required	Pass
10	0.008	0.705	0.207	0.071	0.029	0.754	#13	0.078	Not Required	Pass
11	0.003	0.141	0.057	0.043	0.007	0.196	#13	0.088	Not Required	Pass
12	0.003	0.428	0.296	0.087	0.061	0.725	#13	0.052	Not Required	Pass
13	0.003	0.078	0.152	0.059	0.010	0.193	#15	0.265	Not Required	Pass
14	0.005	0.070	0.154	0.053	0.010	0.176	#15	0.177	Not Required	Pass
15	0.000	0.006	0.005	0.009	0.001	0.010	#13	Not Required	Not Required	Pass
16	0.000	0.006	0.005	0.009	0.001	0.010	#13	Not Required	Not Required	Pass
101	0.057	0.886	0.000	0.049	0.000	0.915	#13	0.560	Not Required	Pass
102	0.002	0.429	0.331	0.097	0.065	0.760	#13	0.052	Not Required	Pass
103	0.008	0.807	0.091	0.081	0.015	0.877	#13	0.044	Not Required	Pass
104	0.008	0.857	0.163	0.086	0.021	0.971	#13	0.078	Not Required	Warn
105	0.008	0.501	0.167	0.081	0.024	0.543	#13	0.073	Not Required	Pass
106	0.008	0.807	0.091	0.081	0.015	0.877	#13	0.044	Not Required	Pass
107	0.008	0.501	0.167	0.080	0.024	0.543	#13	0.073	Not Required	Pass
108	0.005	0.104	0.071	0.049	0.007	0.124	#15	0.088	Not Required	Pass
109	0.007	0.049	0.059	0.001	0.000	0.112	#13	0.198	Not Required	Pass
110	0.008	0.857	0.163	0.086	0.021	0.971	#13	0.078	Not Required	Warn
111	0.003	0.138	0.070	0.044	0.007	0.151	#15	0.088	Not Required	Pass
112	0.002	0.429	0.331	0.097	0.065	0.760	#13	0.052	Not Required	Pass
113	0.003	0.109	0.166	0.059	0.010	0.246	#13	0.265	Not Required	Pass
114	0.007	0.185	0.167	0.065	0.010	0.329	#13	0.265	Not Required	Pass
115	0.003	0.213	0.080	0.044	0.007	0.282	#13	0.321	Not Required	Pass
116	0.005	0.192	0.081	0.049	0.007	0.264	#13	0.321	Not Required	Pass
201	0.042	0.688	0.062	0.038	0.005	0.733	#13	0.560	Not Required	Pass
202	0.003	0.428	0.296	0.087	0.061	0.725	#13	0.052	Not Required	Pass
203	0.008	0.758	0.126	0.079	0.022	0.868	#13	0.044	Not Required	Pass
204	0.008	0.705	0.207	0.071	0.029	0.754	#13	0.078	Not Required	Pass
205	0.008	0.471	0.153	0.076	0.021	0.493	#13	0.073	Not Required	Pass
206	0.005	0.489	0.037	0.047	0.008	0.513	#13	0.044	Not Required	Pass
207	0.004	0.303	0.025	0.049	0.004	0.325	#13	0.073	Not Required	Pass
208	0.000	0.006	0.005	0.009	0.001	0.010	#13	Not Required	Not Required	Pass
209	0.003	0.067	0.078	0.005	0.004	0.145	#13	0.198	Not Required	Pass
210	0.004	0.484	0.127	0.049	0.017	0.611	#13	0.117	Not Required	Pass
211	0.000	0.006	0.005	0.009	0.001	0.010	#13	Not Required	Not Required	Pass
212	0.001	0.190	0.189	0.052	0.041	0.379	#13	0.052	Not Required	Pass
213	0.003	0.078	0.152	0.059	0.010	0.193	#15	0.177	Not Required	Pass
214	0.005	0.070	0.154	0.053	0.010	0.176	#15	0.265	Not Required	Pass
215	0.003	0.225	0.080	0.043	0.007	0.285	#13	0.321	Not Required	Pass
216	0.005	0.209	0.080	0.038	0.007	0.280	#13	0.321	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z, M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ 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="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 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.152</td> <td>6.313</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.562</td> <td>-4.265</td> </tr> <tr> <td>V_z (kip)</td> <td>0.343</td> <td>0.550</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.939</td> <td>1.503</td> </tr> <tr> <td>M_z (kipft)</td> <td>25.447</td> <td>42.825</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)	4.152	6.313	V_x (kip)	-2.562	-4.265	V_z (kip)	0.343	0.550	M_x (kipft)	0.939	1.503	M_z (kipft)	25.447	42.825	
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)	4.152	6.313																										
V_x (kip)	-2.562	-4.265																										
V_z (kip)	0.343	0.550																										
M_x (kipft)	0.939	1.503																										
M_z (kipft)	25.447	42.825																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.562 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.40796 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.447 \text{ kipft}) + ((-2.562 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0521 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.6955 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.343 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.054618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((0.343 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14952 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.7591 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6955 \text{ ft}), (2.7591 \text{ ft})]$$

$$L_{e,req} = 5.695 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.695 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9112$$

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

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

$$q = \frac{(4.152 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.2595 \text{ kip/ft}^2$$

$$q = 0.2099 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.2595 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.12975$$

Status: **PASS**
Ratio: **0.130**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.40796 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.0521 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0521 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.0521 \text{ kipft/ft})) + (4 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3206 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0521 \text{ kipft/ft})) + (3 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.0521 \text{ kipft/ft})) + (2 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.19932 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.0521 \text{ kipft/ft})) + ((-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.85315 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3206 \text{ ft})}{2}$$

$$p_a = 0.32404 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19932 \text{ kip/ft}^2)}{(0.32404 \text{ kip/ft}^2)}$$

$$Ratio = 0.61509$$

p_a - Allowable lateral soil pressure at depth L_e ,

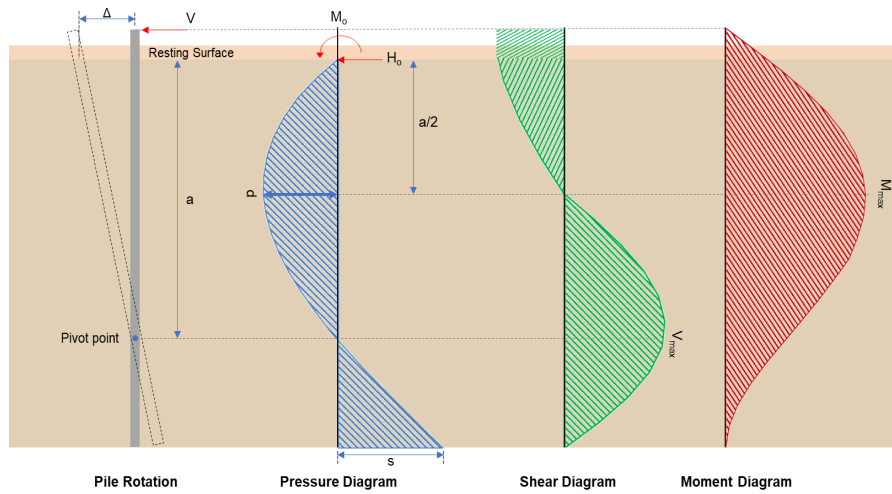
Status: **PASS**
Ratio: **0.620**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.85315 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.91003$	Status: PASS Ratio: 0.910
	<p>Considering z-direction:</p> <p>$H_o = 0.054618 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.14952 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.14952 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.054618 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.14952 \text{ kipft/ft})) + (4 \times (0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.481 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.14952 \text{ kipft/ft})) + (3 \times (0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.14952 \text{ kipft/ft})) + (2 \times (0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = 0.04466 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.14952 \text{ kipft/ft})) + ((0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.098366 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.481 \text{ ft})}{2}$ $p_a = 0.33607 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.04466 \text{ kip/ft}^2)}{(0.33607 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.13289$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.130

$$\text{Ratio} = \frac{(0.098366 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.10492$$

Status: **PASS**
Ratio: **0.100**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.265 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.67914 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.825 \text{ kipft}) + ((-4.265 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8193 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.8193 \text{ kipft/ft})}{(-0.67914 \text{ kip/ft})}$$

$$E = 10.041 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8193 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.67914 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.8193 \text{ kipft/ft})) + (4 \times (-0.67914 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3194 \text{ ft}$$

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

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

$$V_{max} = ((-0.67914 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3194 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3194 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.514 \text{ kip}$$

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

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

$$M_{max} = ((-0.67914 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(10.041 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3194 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3194 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3194 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.191 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.55 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.08758 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.503 \text{ kipft}) + ((0.55 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23933 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.23933 \text{ kipft/ft})}{(0.08758 \text{ kip/ft})}$$

$$E = 2.7327 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23933 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.08758 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.23933 \text{ kipft/ft})) + (4 \times (0.08758 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4812 \text{ ft}$$

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

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

$$V_{max} = ((0.08758 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4812 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4812 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.50493 \text{ kip}$$

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

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

$$M_{max} = ((0.08758 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2.7327 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4812 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4812 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4812 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.383 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.313 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.386 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.386 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(6.313 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0023598$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.313 \text{ kip} \rightarrow 6313 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(6313 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.33 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

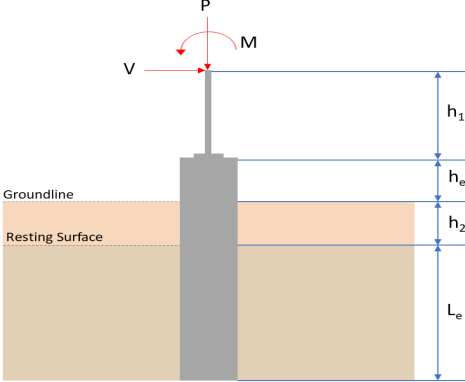
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.33 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.33 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.33 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.64 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.514 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.514 \text{ kip})}{(110.64 \text{ kip})}$ $\text{Ratio} = 0.085988$ <p>Considering z-direction:</p> <p>$V_{max} = 0.50493 \text{ kip}$ - Maximum shear force in the z-direction, <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.50493 \text{ kip})}{(110.64 \text{ kip})}$ $\text{Ratio} = 0.0045635$	<p>Status: PASS Ratio: 0.090</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: $M_{max} = 28.191 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(28.191 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.11295$	<p>Status: PASS Ratio: 0.110</p>
	<p>Considering z-direction: $M_{max} = 1.383 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.383 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0055407$	<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 $L = 6.25$ 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="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>4.152</td> <td>6.313</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.562</td> <td>-4.265</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.343</td> <td>-0.550</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.939</td> <td>-1.503</td> </tr> <tr> <td>M_z (kipft)</td> <td>25.447</td> <td>42.826</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)	4.152	6.313	V_x (kip)	-2.562	-4.265	V_z (kip)	-0.343	-0.550	M_x (kipft)	-0.939	-1.503	M_z (kipft)	25.447	42.826	
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)	4.152	6.313																										
V_x (kip)	-2.562	-4.265																										
V_z (kip)	-0.343	-0.550																										
M_x (kipft)	-0.939	-1.503																										
M_z (kipft)	25.447	42.826																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.562 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.40796 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.447 \text{ kipft}) + ((-2.562 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0521 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.6955 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.343 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.054618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((-0.343 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14952 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.8176 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6955 \text{ ft}), (1.8176 \text{ ft})]$$

$$L_{e,req} = 5.695 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.695 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9112$$

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

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

$$q = \frac{(4.152 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.2595 \text{ kip/ft}^2$$

$$q = 0.2099 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.2595 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.12975$$

Status: **PASS**
Ratio: **0.130**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.40796 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.0521 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0521 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.0521 \text{ kipft/ft})) + (4 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3206 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0521 \text{ kipft/ft})) + (3 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.0521 \text{ kipft/ft})) + (2 \times (-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.19932 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.0521 \text{ kipft/ft})) + ((-0.40796 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.85315 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3206 \text{ ft})}{2}$$

$$p_a = 0.32404 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19932 \text{ kip/ft}^2)}{(0.32404 \text{ kip/ft}^2)}$$

$$Ratio = 0.61509$$

p_a - Allowable lateral soil pressure at depth L_e ,

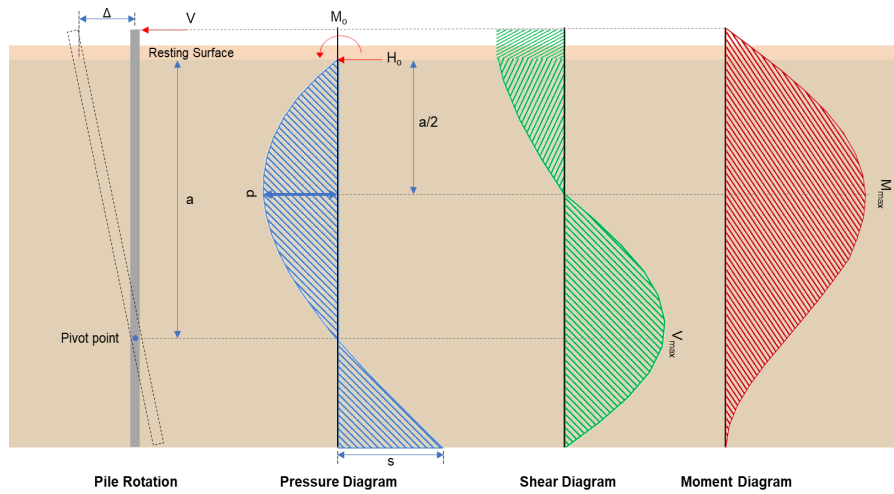
Status: **PASS**
Ratio: **0.620**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.85315 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.91003$	Status: PASS Ratio: 0.910
	<p>Considering z-direction:</p> <p>$H_o = -0.054618 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.14952 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.14952 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.054618 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.14952 \text{ kipft/ft})) + (4 \times (-0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.481 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.14952 \text{ kipft/ft})) + (3 \times (-0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.14952 \text{ kipft/ft})) + (2 \times (-0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = -0.01488 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.14952 \text{ kipft/ft})) + ((-0.054618 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = -0.0065 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.481 \text{ ft})}{2}$ $p_a = 0.33607 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.01488 \text{ kip/ft}^2)}{(0.33607 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.044276$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.040

$$\text{Ratio} = \frac{(-0.0065 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0069332$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.265 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.67914 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.826 \text{ kipft}) + ((-4.265 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8194 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.8194 \text{ kipft/ft})}{(-0.67914 \text{ kip/ft})}$$

$$E = 10.041 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8194 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.67914 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.8194 \text{ kipft/ft})) + (4 \times (-0.67914 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3194 \text{ ft}$$

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

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

$$V_{max} = ((-0.67914 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3194 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3194 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.5142 \text{ kip}$$

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

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

$$M_{max} = ((-0.67914 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(10.041 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3194 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3194 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.041 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3194 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.192 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.55 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.08758 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.503 \text{ kipft}) + ((-0.55 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23933 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.23933 \text{ kipft/ft})}{(-0.08758 \text{ kip/ft})}$$

$$E = 2.7327 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23933 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.08758 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.23933 \text{ kipft/ft})) + (4 \times (-0.08758 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4812 \text{ ft}$$

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

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

$$V_{max} = ((-0.08758 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4812 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4812 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.50493 \text{ kip}$$

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

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

$$M_{max} = ((-0.08758 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2.7327 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4812 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4812 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7327 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4812 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.383 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.313 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.386 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.386 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(6.313 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0023598$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.313 \text{ kip} \rightarrow 6313 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(6313 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.33 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

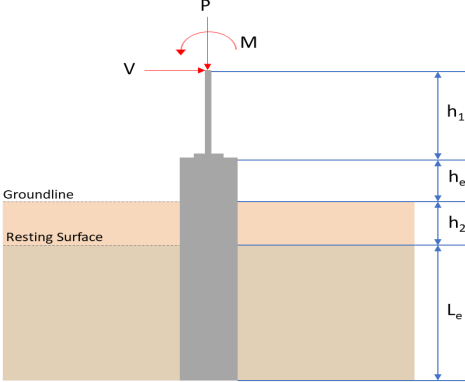
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.33 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.33 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.33 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.64 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.5142 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.5142 \text{ kip})}{(110.64 \text{ kip})}$ $\text{Ratio} = 0.08599$ <p>Considering z-direction:</p> <p>$V_{max} = 0.50493 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.50493 \text{ kip})}{(110.64 \text{ kip})}$ $\text{Ratio} = 0.0045635$	<p>Status: PASS Ratio: 0.090</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: $M_{max} = 28.192 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(28.192 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.11295$	<p>Status: PASS Ratio: 0.110</p>
	<p>Considering z-direction: $M_{max} = 1.383 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.383 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0055407$	<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 $L = 6.75$ 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 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.635</td> <td>8.635</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.356</td> <td>-5.572</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>32.786</td> <td>55.153</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)	5.635	8.635	V_x (kip)	-3.356	-5.572	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	32.786	55.153	
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)	5.635	8.635																										
V_x (kip)	-3.356	-5.572																										
V_z (kip)	0.000	0.000																										
M_x (kipft)	0.000	0.000																										
M_z (kipft)	32.786	55.153																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.356 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.53439 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(32.786 \text{ kipft}) + ((-3.356 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 5.2207 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 6.0657 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.0657 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.066 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.75 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.066 \text{ ft})}{(6.75 \text{ ft})}$ $\text{Ratio} = 0.89867$	<p>Status: PASS Ratio: 0.900</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_v}{A}$ $q = \frac{(5.635 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.35219 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p><i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.35219 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.17609$	<p>Status: PASS Ratio: 0.180</p>
<p>Czerniak</p>	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.53439$ kip/ft - Lateral force per length of pile,

$M_o = 5.2207$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.2207 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.53439 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (5.2207 \text{ kipft/ft})) + (4 \times (-0.53439 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6774 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.2207 \text{ kipft/ft})) + (3 \times (-0.53439 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (5.2207 \text{ kipft/ft})) + (2 \times (-0.53439 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.19725 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.2207 \text{ kipft/ft})) + ((-0.53439 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.89998 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6774 \text{ ft})}{2}$$

$$p_a = 0.3508 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19725 \text{ kip/ft}^2)}{(0.3508 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.56228$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

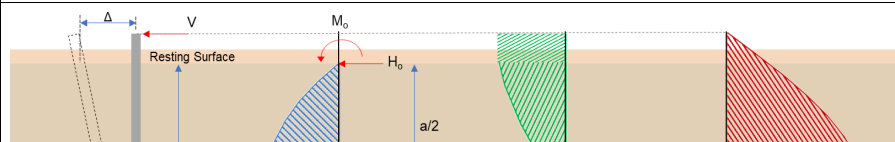
$$\text{Ratio} = \frac{s}{p_s}$$

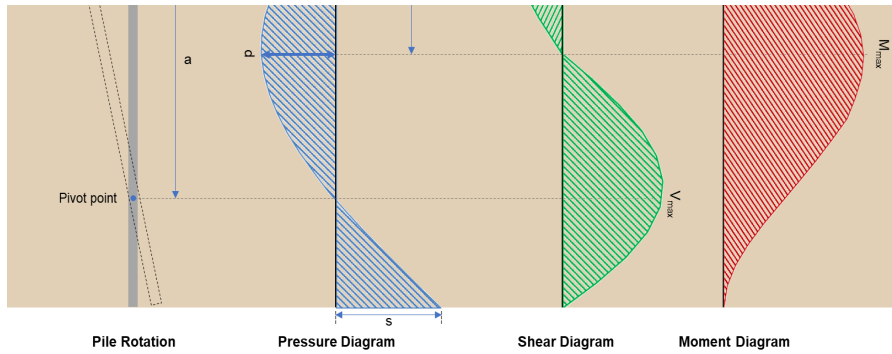
$$\text{Ratio} = \frac{(0.89998 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88887$$

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

Status: **PASS**
Ratio: **0.890**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.572 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.88726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.153 \text{ kipft}) + ((-5.572 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.7823 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.7823 \text{ kipft/ft})}{(-0.88726 \text{ kip/ft})}$$

$$E = 9.8982 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.7823 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.88726 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.7823 \text{ kipft/ft})) + (4 \times (-0.88726 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6758 \text{ ft}$$

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

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

$$V_{max} = ((-0.88726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.8982 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6758 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.8982 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6758 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.549 \text{ kip}$$

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

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

$$M_{max} = ((-0.88726 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(9.8982 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6758 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.8982 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6758 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.8982 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6758 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 36.808 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.635 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.309 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.309 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(8.635 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0032278$$

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

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.635 \text{ kip} \rightarrow 8635 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8635 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.64 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.64 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.64 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.64 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.84 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.549 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.549 \text{ kip})}{(110.84 \text{ kip})}$ $\text{Ratio} = 0.10419$	<p>Status: PASS Ratio: 0.100</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$\phi M_{n,2} = \phi M_{n,1}$

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

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

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:

$M_{max} = 36.808 \text{ kipft}$ - Maximum moment in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(36.808 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.14747$$

Status: **PASS**
Ratio: **0.150**