

Your Project Calculations



Project Name: CORios

S3D Model Link:

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

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=ZMCB4CXSeapPYv1Cw13R3qJ1nTUNC2oNuQtVo66YWON7PM74EsFcMI20zODfarr

Array Specification

Product:	Beam
Unique ID:	2P-22.5-8TOP-HD-45-L-4Hx6W-B83J
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	75.08in
Number of Rows:	4
Number of Columns:	6
Total Number of Modules:	24
Desired Tilt Angle:	37
Front Edge Clearance:	5
Total Array Height at Tilt:	13.30 ft
Total Frame Length:	37.50 ft
Frame Weight:	2166 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	38.04 ft
Rail Length:	166.40 in
Rail Spacing:	3.13 ft
Rail Check:	Not Checked

Support Specifications

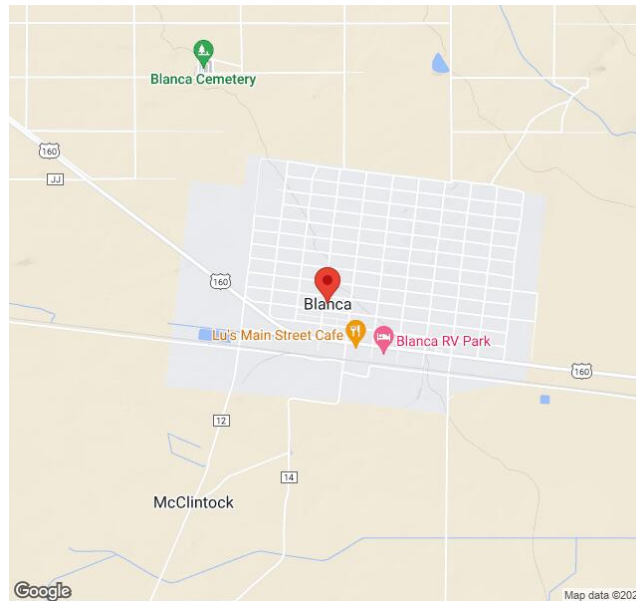
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	9.17 ft
Number of Poles:	2
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.00 ft
Foundation Volume:	7.111 y ³
Foundation Result:	PASSED
Mount Twist:	1.315628 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Blanca, CO 81123, USA
Wind Speed:	102 mph
Snow Load:	35 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.012701 ksf



Design Disclaimer

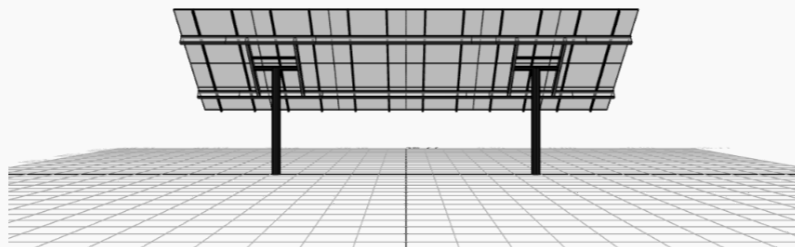
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

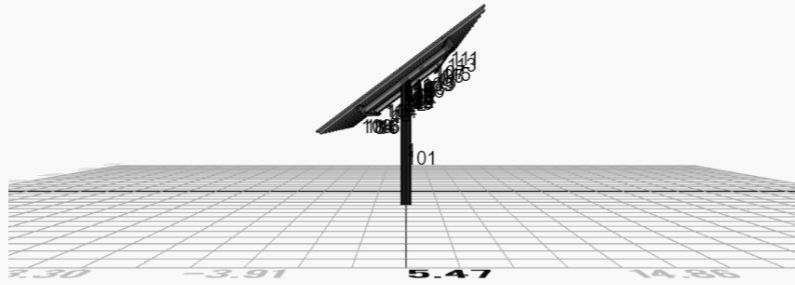
AutoDesigner Input

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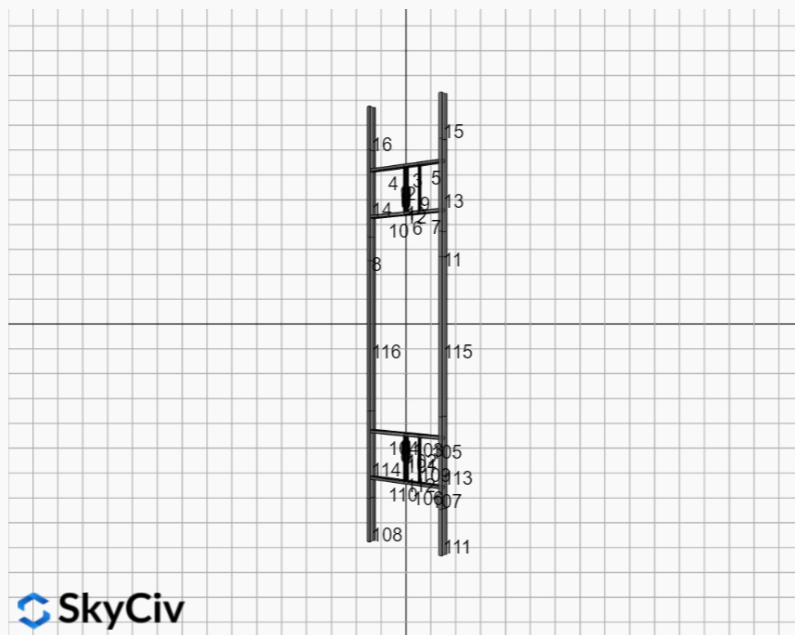
Design Notes:

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

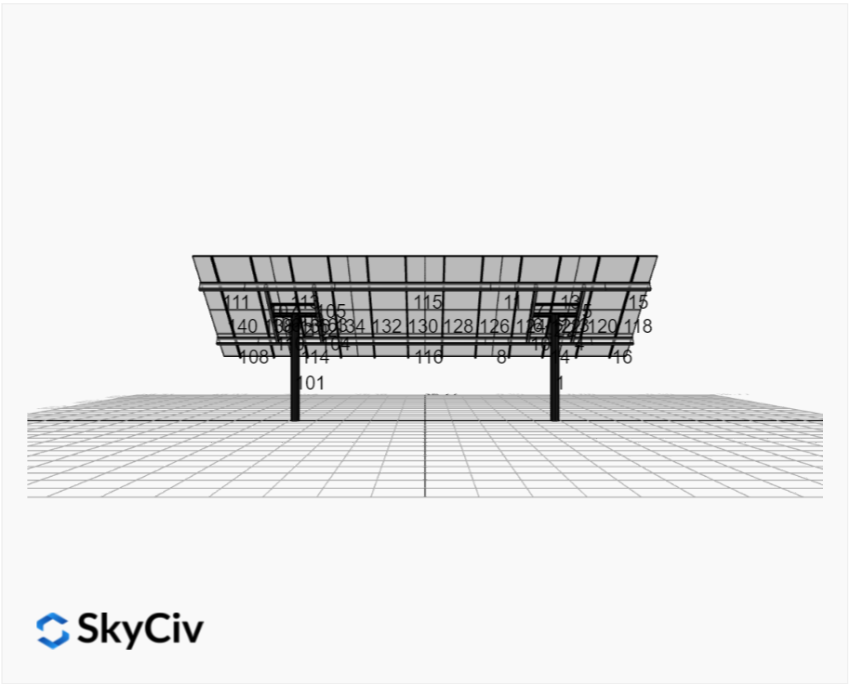
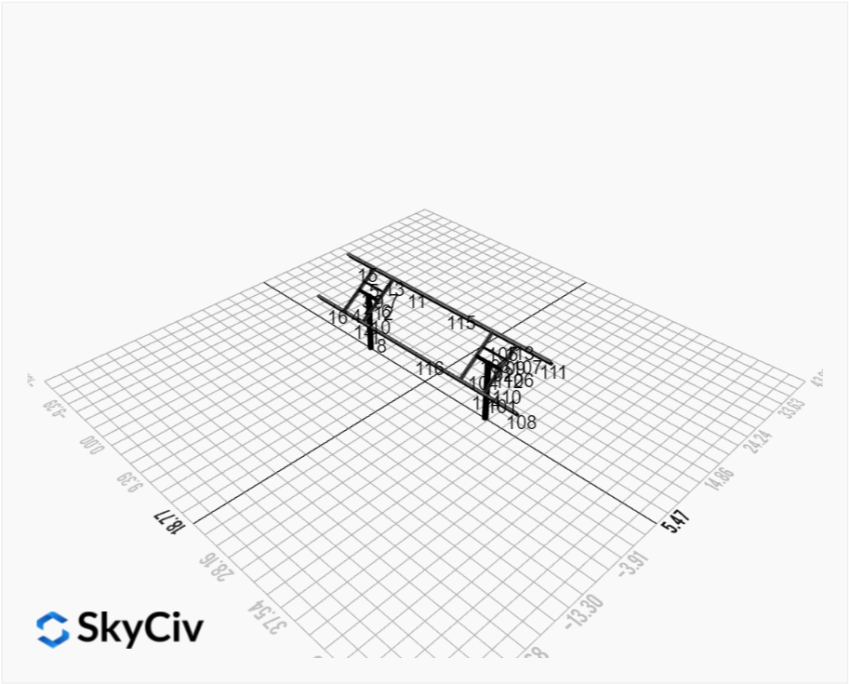




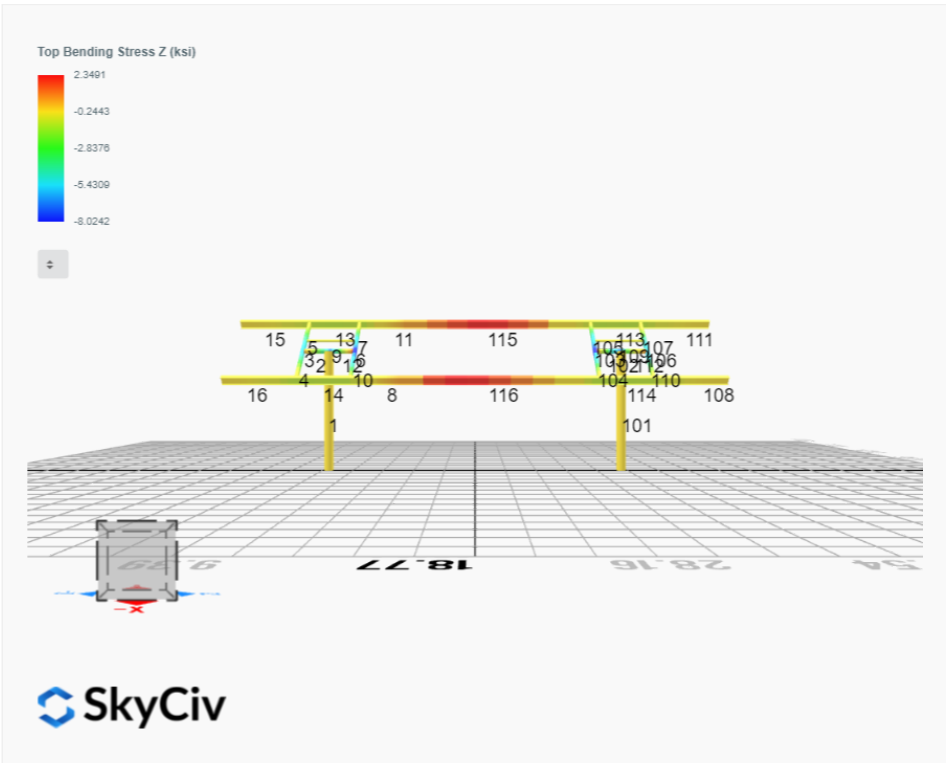
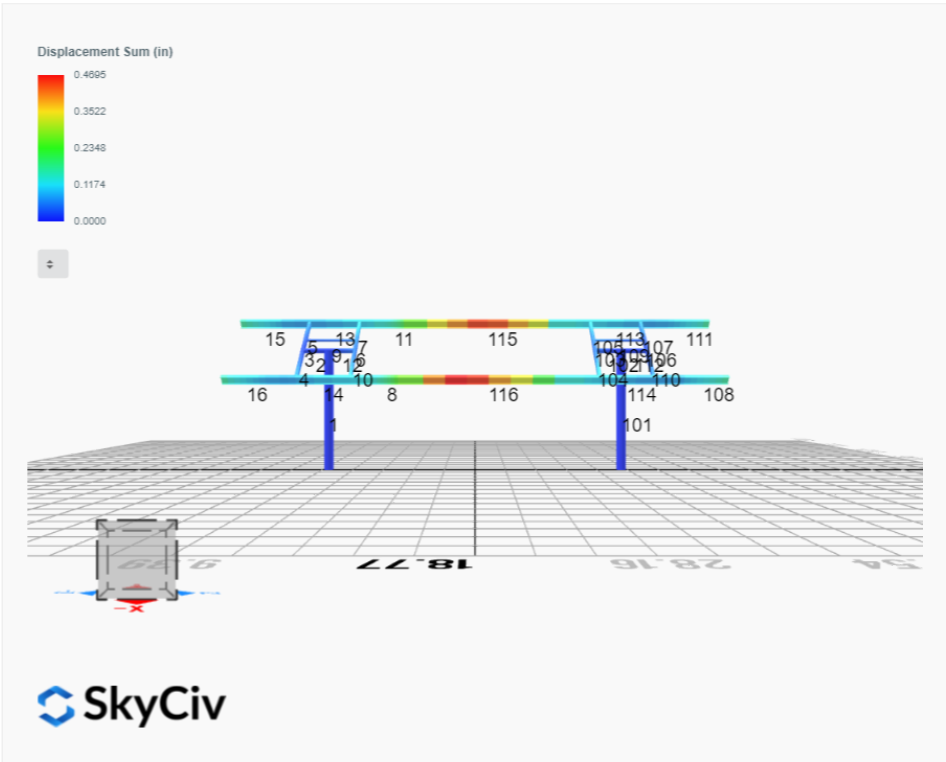
 SkyCiv

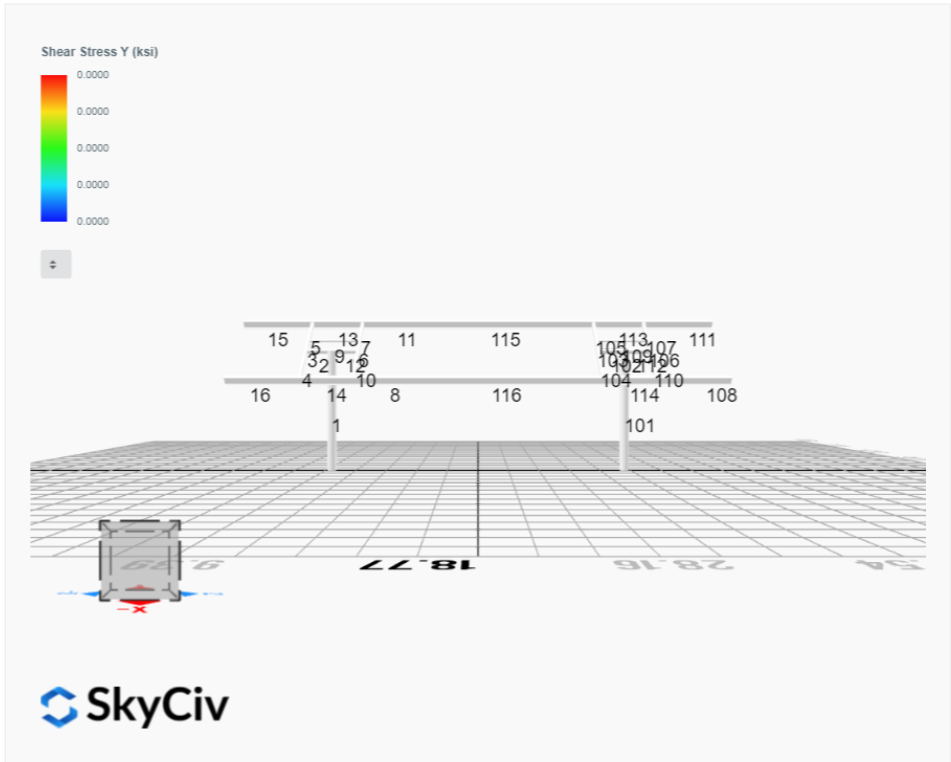
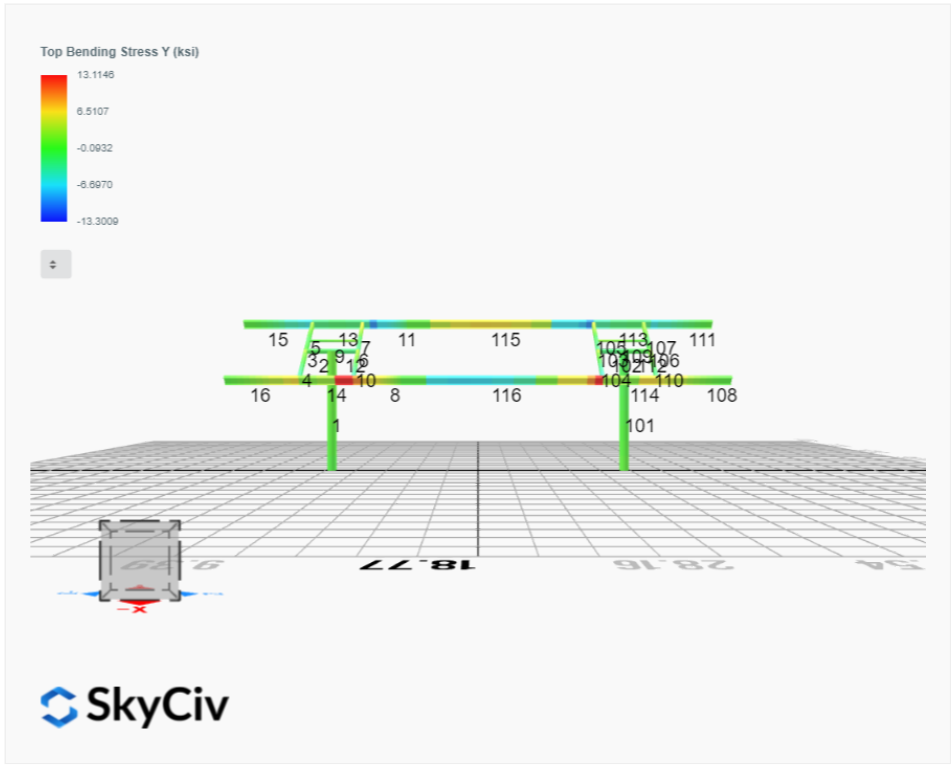


 SkyCiv

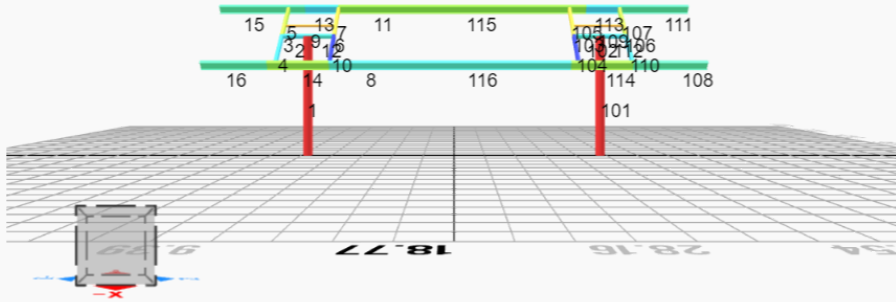
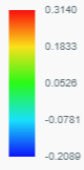


FEM Results (Envelope Worst Case for each member)





Axial Stress (ksi)



Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 2. D + L	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 3. D + (S or Lr or R)	0.0000	4.7239	0.2327	0.5528	-0.2627	0.0339
ULS: 3. D + (S or Lr or R)	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.0646	0.1965	0.4669	-0.2218	0.0317
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 5b. D + 0.7E	0.0000	2.0866	0.0881	0.2090	-0.0994	0.0250
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.0646	0.1965	0.4669	-0.2218	0.0317
ULS: 8. 0.6D + 0.7E	0.0000	1.2520	0.0529	0.1254	-0.0596	0.0150
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5219	5.4334	0.3174	0.7135	-0.7679	23.7564
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5219	5.4334	0.3174	0.7135	-0.7679	23.7564
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1147	-0.7196	-0.1030	-0.2110	0.4593	-19.1994
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7622	-0.2519	-0.0723	-0.1436	0.3695	-22.5828
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8915	6.5746	0.3685	0.8452	-0.7232	17.8302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8915	6.5746	0.3685	0.8452	-0.7232	17.8302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5860	1.9599	0.0532	0.1519	0.1971	-14.3866
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3217	2.3107	0.0762	0.2024	0.1298	-16.9241
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8915	4.5967	0.2601	0.5874	-0.6007	17.8235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8915	4.5967	0.2601	0.5874	-0.6007	17.8235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5860	-0.0181	-0.0552	-0.1060	0.3196	-14.3933
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3217	0.3327	-0.0322	-0.0555	0.2523	-16.9308
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5219	4.5987	0.2821	0.6299	-0.7281	23.7464
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5219	4.5987	0.2821	0.6299	-0.7281	23.7464
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1147	-1.5543	-0.1382	-0.2946	0.4990	-19.2094
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7622	-1.0866	-0.1076	-0.2272	0.4093	-22.5928

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.5126
Shear X	-4.2032
Shear Z	0.5612
Moment X	1.2663
Moment Y (Twist)	1.3148
Moment Z	39.9330

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.5746
Shear X	-2.5219
Shear Z	0.3685
Moment X	0.8452
Moment Y (Twist)	0.7679
Moment Z	23.7564

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	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 2. D + L	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 3. D + (S or Lr or R)	-0.0000	4.7239	-0.2327	-0.5530	0.2629	0.0341
ULS: 3. D + (S or Lr or R)	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.0646	-0.1965	-0.4670	0.2220	0.0318
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250
ULS: 5b. D + 0.7E	-0.0000	2.0866	-0.0881	-0.2091	0.0994	0.0250

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.0646	-0.1965	-0.4670	0.2220	0.0318
ULS: 8. 0.6D + 0.7E	-0.0000	1.2520	-0.0529	-0.1254	0.0597	0.0150
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5220	5.4334	-0.3174	-0.7136	0.7679	23.7564
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5220	5.4334	-0.3174	-0.7136	0.7679	23.7564
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1147	-0.7196	0.1030	0.2110	-0.4592	-19.1994
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7622	-0.2519	0.0723	0.1436	-0.3695	-22.5827
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8915	6.5746	-0.3685	-0.8454	0.7234	17.8304
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8915	6.5746	-0.3685	-0.8454	0.7234	17.8304
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5860	1.9599	-0.0532	-0.1520	-0.1970	-14.3865
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3217	2.3106	-0.0762	-0.2025	-0.1297	-16.9240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8915	4.5967	-0.2601	-0.5874	0.6008	17.8235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8915	4.5967	-0.2601	-0.5874	0.6008	17.8235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5860	-0.0181	0.0552	0.1059	-0.3196	-14.3933
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3217	0.3327	0.0322	0.0554	-0.2523	-16.9308
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5219	4.5987	-0.2821	-0.6299	0.7281	23.7464
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5219	4.5987	-0.2821	-0.6299	0.7281	23.7464
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1147	-1.5543	0.1382	0.2946	-0.4990	-19.2094
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7622	-1.0866	0.1076	0.2272	-0.4093	-22.5927

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.5125
Shear X	-4.2033
Shear Z	-0.5612
Moment X	-1.2669
Moment Y (Twist)	1.3156
Moment Z	39.9341

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.5746
Shear X	-2.5220
Shear Z	-0.3685
Moment X	-0.8454
Moment Y (Twist)	0.7679
Moment Z	23.7564

Project Details

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

User Name: sales@mtsolar.us
Project Name: CORios
Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions								
ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
9	8in Pipe Sch 40	8.63	0.32					
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x16	5.00	3.00	0.17	0.17	0.17		
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D
1	9	19.26	19.26	9.17	-	300	200	1
2	5	4.20	4.20	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.19,1.18,1.18,1.16,1.17,1.17,1.17,1.17,1.17	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.70,1.67,1.67,1.66,1.35,1.67,1.67,1.68,1.68,1.67,1.67,1.64,1.71,1.67,1.67,1.66,1.62	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.69,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1
8	19	1.33	1.33	2.05	1.65,1.65,1.65,1.65,1.65,1.65,1.63,1.63,1.60,2.10,1.63,1.63,1.61,1.34,1.64,1.64,1.70,1.69,1.63,1.63,1.59,1.95,1.62,1.62,1.62,1.45	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.62,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.72,1.67,1.67,1.66,1.65	300	200	1
11	19	1.33	1.33	2.05	1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.64,1.61,1.65,1.64,1.64,1.63,1.66,1.65,1.65,1.72,1.71,1.65,1.65,1.61,1.65,1.64,1.64,1.63,1.66	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.09,1.10,1.14,1.09,1.09,1.09,1.13,1.09,1.09,1.07,1.09,1.09,1.09,1.10,1.15,1.09,1.09,1.09,1.13	300	200	1
14	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.99,1.08,1.08,1.09,1.24,1.08,1.08,1.07,1.11,1.08,1.10,1.39,1.09,1.09,1.09,1.16	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	19.26	19.26	9.17	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
104	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.62,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.72,1.67,1.67,1.66,1.65	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1
106	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.19,1.18,1.18,1.16,1.17,1.17,1.17,1.17,1.17,1.17	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.69,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1

[illegible]

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	240.36	83.29	83.29	113.39	113.39
2	198.33	182.14	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	126.01	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	126.01	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.52	6.12	40.24	43.62
14	133.20	104.94	24.52	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62
101	377.97	240.36	83.29	83.29	113.39	113.39
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	52.83	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	52.83	32.87	6.12	40.24	43.62
112	198.33	182.14	21.95	21.95	59.50	59.50
113	133.20	104.94	24.52	6.12	40.24	43.62
114	133.20	104.94	24.52	6.12	40.24	43.62

115	133.20	46.28	12.02	6.12	40.24	43.62
116	133.20	46.28	11.81	6.12	40.24	43.62

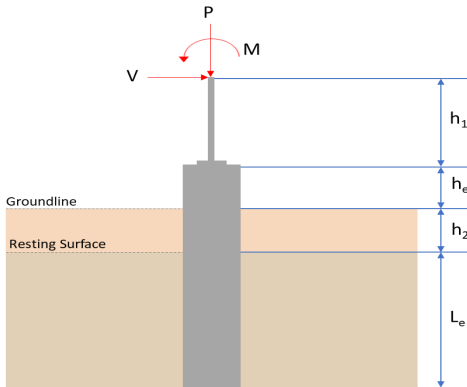
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.040	0.479	0.046	0.037	0.005	0.514	#13	0.393	Not Required	Pass
2	0.004	0.261	0.162	0.062	0.032	0.424	#13	0.114	Not Required	Pass
3	0.007	0.499	0.035	0.049	0.009	0.506	#13	0.045	Not Required	Pass
4	0.006	0.480	0.126	0.048	0.029	0.555	#13	0.080	Not Required	Pass
5	0.006	0.310	0.105	0.050	0.027	0.320	#13	0.074	Not Required	Pass
6	0.010	0.671	0.087	0.069	0.017	0.729	#13	0.045	Not Required	Pass
7	0.010	0.415	0.201	0.067	0.051	0.449	#13	0.074	Not Required	Pass
8	0.003	0.098	0.205	0.047	0.017	0.210	#24	0.095	Not Required	Pass
9	0.014	0.061	0.081	0.002	0.004	0.139	#13	0.204	Not Required	Pass
10	0.012	0.649	0.180	0.065	0.037	0.678	#13	0.080	Not Required	Pass
11	0.005	0.100	0.214	0.048	0.017	0.222	#21	0.095	Not Required	Pass
12	0.003	0.437	0.222	0.092	0.039	0.659	#13	0.053	Not Required	Pass
13	0.006	0.186	0.452	0.060	0.021	0.594	#21	0.286	Not Required	Pass
14	0.006	0.182	0.442	0.058	0.021	0.579	#21	0.190	Not Required	Pass
15	0.000	0.056	0.113	0.024	0.008	0.162	#21	Not Required	Not Required	Pass
16	0.000	0.054	0.113	0.023	0.008	0.161	#21	Not Required	Not Required	Pass
101	0.040	0.479	0.046	0.037	0.005	0.514	#13	0.393	Not Required	Pass
102	0.003	0.437	0.222	0.092	0.039	0.659	#13	0.053	Not Required	Pass
103	0.010	0.671	0.087	0.069	0.017	0.729	#13	0.045	Not Required	Pass
104	0.012	0.649	0.180	0.065	0.037	0.678	#13	0.080	Not Required	Pass
105	0.010	0.415	0.201	0.067	0.051	0.449	#13	0.074	Not Required	Pass
106	0.007	0.499	0.035	0.049	0.009	0.506	#13	0.045	Not Required	Pass
107	0.006	0.310	0.105	0.050	0.027	0.320	#13	0.074	Not Required	Pass
108	0.000	0.054	0.113	0.023	0.008	0.161	#21	Not Required	Not Required	Pass
109	0.014	0.061	0.081	0.002	0.004	0.139	#13	0.204	Not Required	Pass
110	0.006	0.480	0.126	0.048	0.029	0.555	#13	0.080	Not Required	Pass
111	0.000	0.056	0.113	0.024	0.008	0.162	#21	Not Required	Not Required	Pass
112	0.004	0.261	0.162	0.062	0.032	0.424	#13	0.114	Not Required	Pass
113	0.006	0.186	0.451	0.060	0.021	0.594	#21	0.190	Not Required	Pass
114	0.006	0.182	0.442	0.058	0.021	0.579	#21	0.286	Not Required	Pass
115	0.013	0.585	0.245	0.048	0.017	0.763	#21	0.601	Not Required	Pass
116	0.005	0.576	0.245	0.047	0.017	0.751	#21	0.601	Not Required	Pass

Definitions

Φ _t	Safety factor for tensile
Φ _c	Safety factor for compression
Φ _b	Safety factor for flexure
Φ _v	Safety factor for shear
E	Modulus of elasticity
F _y	Specified minimum yield stress
F _u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I _{yp}	Moment of inertia about the Y axes
I _{zp}	Moment of inertia about the Z axes
I _w	Warping constant
S _{yp}	Plastic section modulus about the Y axis
S _{zp}	Plastic section modulus about the Z axis
KL	Effective length
C _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																										
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																											
	Pile Input  Geometry Pile shape: rectangular b = 48 in - Pile width D = 48 in - Pile depth L = 6 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>6.575</td><td>9.513</td></tr><tr><td>Vx (kip)</td><td>-2.522</td><td>-4.203</td></tr><tr><td>Vz (kip)</td><td>0.368</td><td>0.561</td></tr><tr><td>Mx (kipft)</td><td>0.845</td><td>1.266</td></tr><tr><td>Mz (kipft)</td><td>23.756</td><td>39.933</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.575	9.513	Vx (kip)	-2.522	-4.203	Vz (kip)	0.368	0.561	Mx (kipft)	0.845	1.266	Mz (kipft)	23.756	39.933	
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Mz (kipft)	23.756	39.933																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0\text{ ft}) + (0\text{ ft}) + (0\text{ ft})$ $H = 0\text{ ft}$ Considering x-direction: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.522\text{ kip})}{1.57 \times (48\text{ in})}$ $H_o = -0.40159\text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(23.756 \text{ kipft}) + ((-2.522 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.7828 \text{ kipft/ft}$ 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: $L_{e,x} = 5.5323 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.368 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.058599 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.845 \text{ kipft}) + ((0.368 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.13455 \text{ kipft/ft}$ 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: $L_{e,z} = 2.7309 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.5323 \text{ ft}), (2.7309 \text{ ft})]$ $L_{e,req} = 5.532 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.532 \text{ ft})}{(6 \text{ ft})}$ $Ratio = 0.922$	Status: PASS Ratio: 0.920
	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 $q = \frac{P_o}{A}$ $q = \frac{(6.575 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.41094 \text{ kips/ft}^2$	

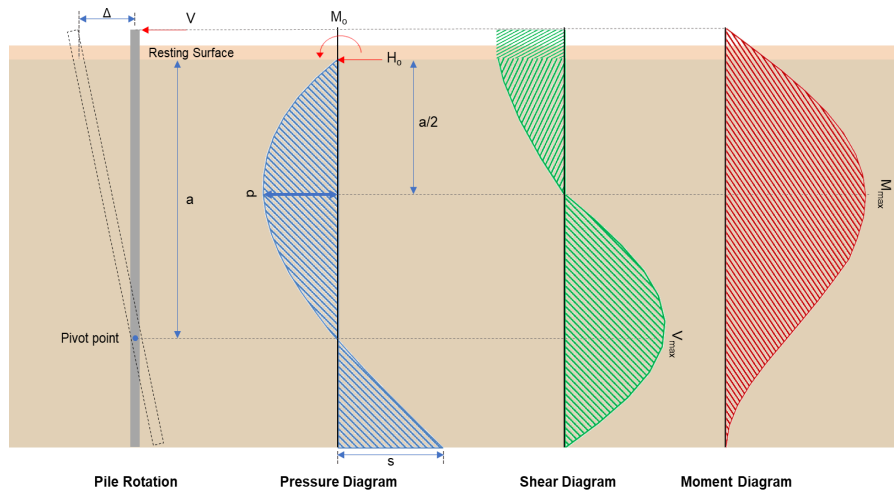
	$q = 0.41094 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.41094 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.20547$	Status: PASS Ratio: 0.210
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.5$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.40159 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 3.7828 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (3.7828 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.7828 \text{ kipft/ft})) + (4 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.149 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (3.7828 \text{ kipft/ft})) + (3 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^3 \times [(3 \times (3.7828 \text{ kipft/ft})) + (2 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = 0.19926 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (3.7828 \text{ kipft/ft})) + ((-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = 0.85934 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.149 \text{ ft})}{2}$ $p_a = 0.31118 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.19926 \text{ kip/ft}^2)}{(0.31118 \text{ kip/ft}^2)}$ $Ratio = 0.64035$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.640

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.85934 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.95482$	Status: PASS Ratio: 0.950
	Considering z-direction: $H_o = 0.058599 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.13455 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.13455 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.058599 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.13455 \text{ kipft/ft})) + (4 \times (0.058599 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.3177 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.13455 \text{ kipft/ft})) + (3 \times (0.058599 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^3 \times [(3 \times (0.13455 \text{ kipft/ft})) + (2 \times (0.058599 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = 0.047764 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.13455 \text{ kipft/ft})) + ((0.058599 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = 0.10345 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.3177 \text{ ft})}{2}$ $p_a = 0.32382 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.047764 \text{ kip/ft}^2)}{(0.32382 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.1475$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.150

$$Ratio = \frac{(0.10345 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.11494$$

Status: **PASS**
Ratio: **0.110**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.203 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.66927 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.933 \text{ kipft}) + ((-4.203 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.3588 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.3588 \text{ kipft/ft})}{(-0.66927 \text{ kip/ft})}$$

$$E = 9.5011 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.3588 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.66927 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.3588 \text{ kipft/ft})) + (4 \times (-0.66927 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1481 \text{ ft}$$

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

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

$$V_{max} = ((-0.66927 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.5011 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1481 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.5011 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1481 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.2665 \text{ kip}$$

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

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

$$M_{max} = ((-0.66927 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(9.5011 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1481 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.5011 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1481 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.5011 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1481 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 26.343 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.561 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.089331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.266 \text{ kipft}) + ((0.561 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20159 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20159 \text{ kipft/ft})}{(0.089331 \text{ kip/ft})}$$

$$E = 2.2567 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20159 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.089331 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.20159 \text{ kipft/ft})) + (4 \times (0.089331 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3197 \text{ ft}$$

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

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

$$V_{max} = ((0.089331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.2567 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3197 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.2567 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3197 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.47694 \text{ kip}$$

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

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

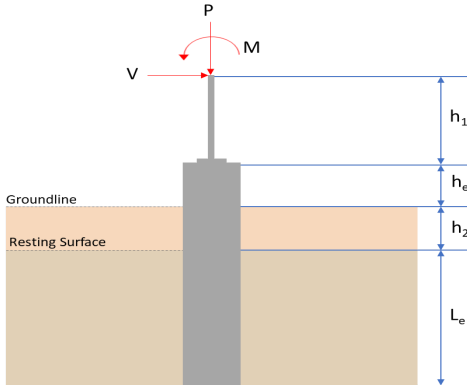
$$M_{max} = ((0.089331 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.2567 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3197 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.2567 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3197 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.2567 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3197 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 1.2403 \text{ kipft}$	
		Minimum Reinforcement Check (LRFD) Parameters: $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength, $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength, $\phi = 0.65$ - Reduction factor for axial strength, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(9.513 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.28 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.28 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(9.513 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.003556$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.513 \text{ kip} \rightarrow 9513 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9513 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.75 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (119.75 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.75 \text{ kip}$	

22.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ A_v - Ties rebar area, $A_v = \frac{\pi d_{tes}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ 22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b) $V_{s,b} = \frac{2 A_v f_{ysk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.75 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.92 \text{ kip}$ Considering x-direction: $V_{max} = 9.2665 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(9.2665 \text{ kip})}{(110.92 \text{ kip})}$ $Ratio = 0.083541$ Considering z-direction: $V_{max} = 0.47694 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.47694 \text{ kip})}{(110.92 \text{ kip})}$ $Ratio = 0.0042998$	Status: PASS Ratio: 0.080 Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

14.5.2.1b	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 26.343 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(26.343 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.10554$	Status: PASS Ratio: 0.110
	Considering z-direction: $M_{max} = 1.2403 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.2403 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0049691$	Status: PASS Ratio: 0.000

REFERENCES	CALCULATIONS	RESULTS																										
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																											
	Pile Input  Geometry Pile shape: rectangular b = 48 in - Pile width D = 48 in - Pile depth L = 6 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><thead><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr></thead><tbody><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></tbody></table> Tabulation of Loads <table><thead><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr></thead><tbody><tr><td>P (kip)</td><td>6.575</td><td>9.512</td></tr><tr><td>Vx (kip)</td><td>-2.522</td><td>-4.203</td></tr><tr><td>Vz (kip)</td><td>-0.368</td><td>-0.561</td></tr><tr><td>Mx (kipft)</td><td>-0.845</td><td>-1.267</td></tr><tr><td>Mz (kipft)</td><td>23.756</td><td>39.934</td></tr></tbody></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.575	9.512	Vx (kip)	-2.522	-4.203	Vz (kip)	-0.368	-0.561	Mx (kipft)	-0.845	-1.267	Mz (kipft)	23.756	39.934	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
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Vx (kip)	-2.522	-4.203																										
Vz (kip)	-0.368	-0.561																										
Mx (kipft)	-0.845	-1.267																										
Mz (kipft)	23.756	39.934																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ Considering x-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.522 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.40159 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(23.756 \text{ kipft}) + ((-2.522 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.7828 \text{ kipft/ft}$ 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: $L_{e,x} = 5.5323 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.368 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.058599 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.845 \text{ kipft}) + ((-0.368 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.13455 \text{ kipft/ft}$ 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: $L_{e,z} = 1.6896 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.5323 \text{ ft}), (1.6896 \text{ ft})]$ $L_{e,req} = 5.532 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.532 \text{ ft})}{(6 \text{ ft})}$ $Ratio = 0.922$	Status: PASS Ratio: 0.920
	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 $q = \frac{P_o}{A}$ $q = \frac{(6.575 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.41094 \text{ kips/ft}^2$	

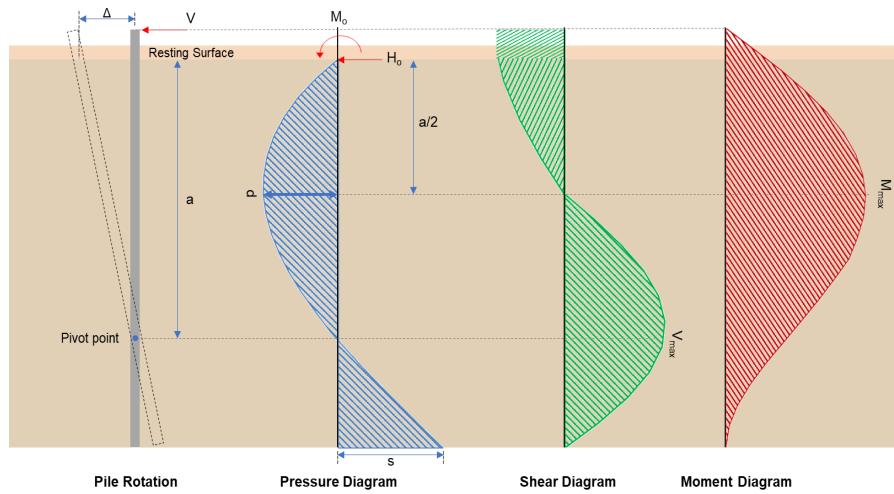
	$q = 0.41094 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.41094 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.20547$	Status: PASS Ratio: 0.210
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.5$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.40159 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 3.7828 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (3.7828 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.7828 \text{ kipft/ft})) + (4 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.149 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (3.7828 \text{ kipft/ft})) + (3 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^3 \times [(3 \times (3.7828 \text{ kipft/ft})) + (2 \times (-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = 0.19926 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (3.7828 \text{ kipft/ft})) + ((-0.40159 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = 0.85934 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.149 \text{ ft})}{2}$ $p_a = 0.31118 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.19926 \text{ kip/ft}^2)}{(0.31118 \text{ kip/ft}^2)}$ $Ratio = 0.64035$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.640

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.85934 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.95482$	Status: PASS Ratio: 0.950
	Considering z-direction: $H_o = -0.058599 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.13455 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.13455 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.13455 \text{ kipft/ft})) + (4 \times (-0.058599 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.3177 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.13455 \text{ kipft/ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^3 \times [(3 \times (0.13455 \text{ kipft/ft})) + (2 \times (-0.058599 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = -0.01856 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.13455 \text{ kipft/ft})) + ((-0.058599 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = -0.013747 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.3177 \text{ ft})}{2}$ $p_a = 0.32382 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.01856 \text{ kip/ft}^2)}{(0.32382 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.057314$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.060

$$Ratio = \frac{(-0.013747 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = -0.015275$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.203 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.66927 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.934 \text{ kipft}) + ((-4.203 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.3589 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.3589 \text{ kipft/ft})}{(-0.66927 \text{ kip/ft})}$$

$$E = 9.5013 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.3589 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.66927 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.3589 \text{ kipft/ft})) + (4 \times (-0.66927 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1481 \text{ ft}$$

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

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

$$V_{max} = ((-0.66927 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.5013 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1481 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.5013 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1481 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.2667 \text{ kip}$$

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

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

$$M_{max} = ((-0.66927 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(9.5013 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1481 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.5013 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1481 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.5013 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1481 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 26.343 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.561 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.089331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.267 \text{ kipft}) + ((-0.561 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20175 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20175 \text{ kipft/ft})}{(-0.089331 \text{ kip/ft})}$$

$$E = 2.2585 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20175 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.089331 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.20175 \text{ kipft/ft})) + (4 \times (-0.089331 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3196 \text{ ft}$$

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

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

$$V_{max} = ((-0.089331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.2585 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3196 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.2585 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3196 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.47712 \text{ kip}$$

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

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

$$M_{max} = ((-0.089331 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.2585 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3196 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.2585 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3196 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.2585 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3196 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 1.2408 \text{ kipft}$	
		Minimum Reinforcement Check (LRFD) Parameters: $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength, $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength, $\phi = 0.65$ - Reduction factor for axial strength, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(9.512 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.28 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.28 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(9.512 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0035556$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.512 \text{ kip} \rightarrow 9512 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9512 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.75 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (119.75 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.75 \text{ kip}$	

22.5.1.2 22.5.8.5.3 22.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ A_v - Ties rebar area, $A_v = \frac{\pi d_{tes}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ $V_{s,b}$ - Shear strength of steel (b) $V_{s,b} = \frac{2 A_v f_{ysk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.75 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.92 \text{ kip}$ Considering x-direction: $V_{max} = 9.2667 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(9.2667 \text{ kip})}{(110.92 \text{ kip})}$ $Ratio = 0.083543$ Considering z-direction: $V_{max} = 0.47712 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.47712 \text{ kip})}{(110.92 \text{ kip})}$ $Ratio = 0.0043015$	Status: PASS Ratio: 0.080 Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

14.5.2.1b	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 26.343 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(26.343 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.10554$	Status: PASS Ratio: 0.110
	Considering z-direction: $M_{max} = 1.2408 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.2408 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0049713$	Status: PASS Ratio: 0.000