

Your Project Calculations



Project Name: South Pole Solar 30 A2

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=South%20Pole%20Solar%2030%20A2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=xjPs2fXHsGgqdKN2ahKFnIQmUIKmATRAioFdzxOOqWsqfKmtAZD4RpoRn6ACpnLs

Array Specification

Product:	Beam
Unique ID:	2P-17-6TOP-HD-57-L-5Hx6W-63KG
Duty Classification:	HD
Module Width:	44.64 in
Module Length:	67.87in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Desired Tilt Angle:	15
Front Edge Clearance:	5
Total Array Height at Tilt:	9.84 ft
Total Frame Length:	34.00 ft
Frame Weight:	1391 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	34.44 ft
Rail Length:	225.70 in
Rail Spacing:	2.83 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	7.43 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.00 ft Pile 2: 5.00 ft
Foundation Volume:	5.926 y ³
Foundation Result:	PASSED
Mount Twist:	0.260914 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	2245 Sean Ln, West Harrison, IN 47060, USA
Wind Speed:	100 mph
Snow Load:	20 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.012096 ksf



Design Disclaimer

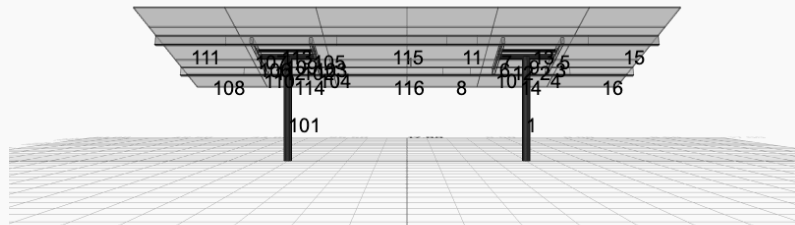
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

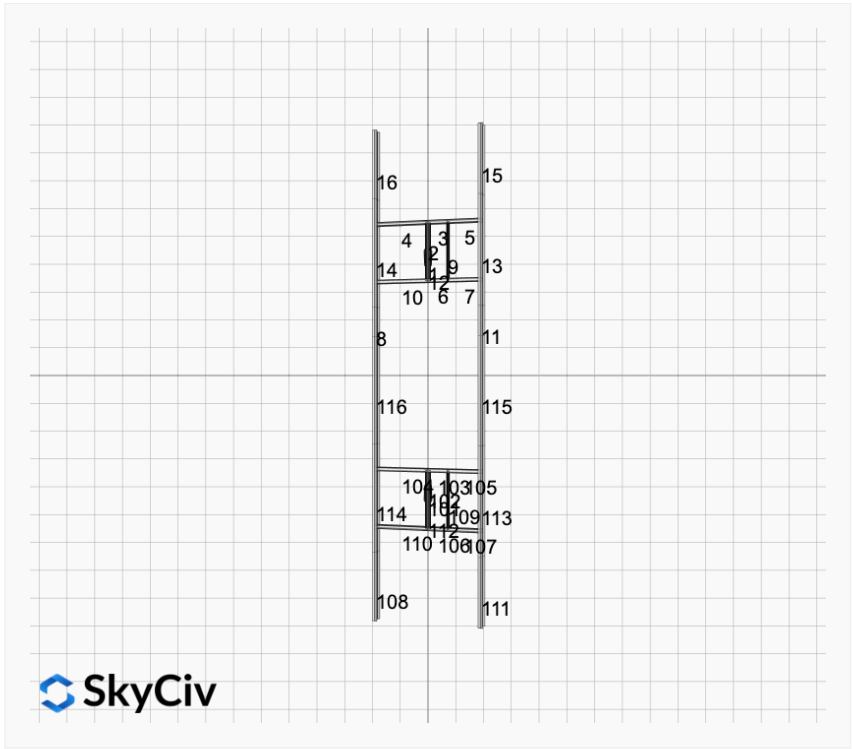
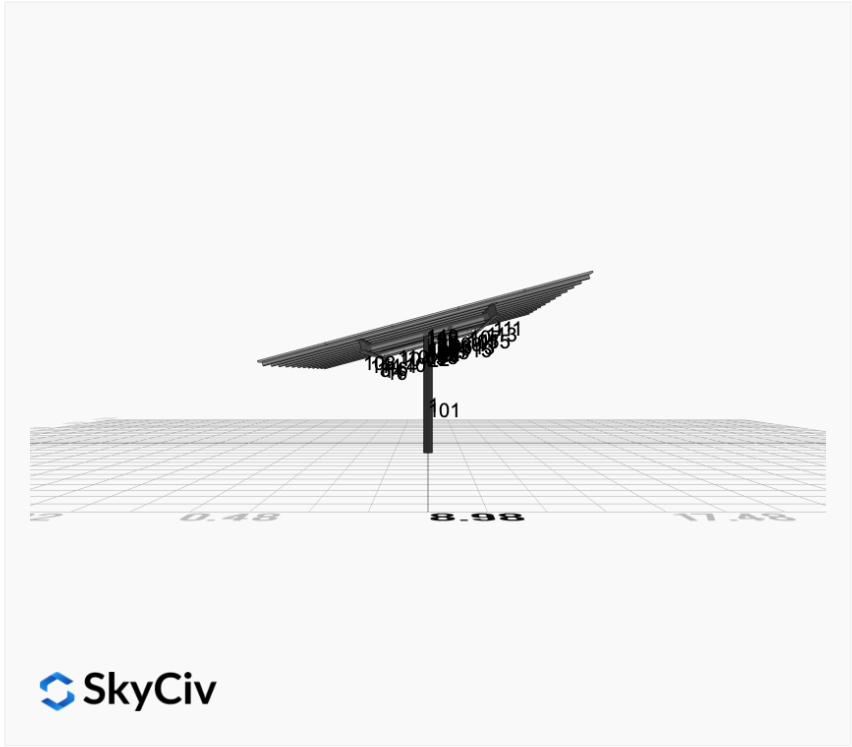
AutoDesigner Input

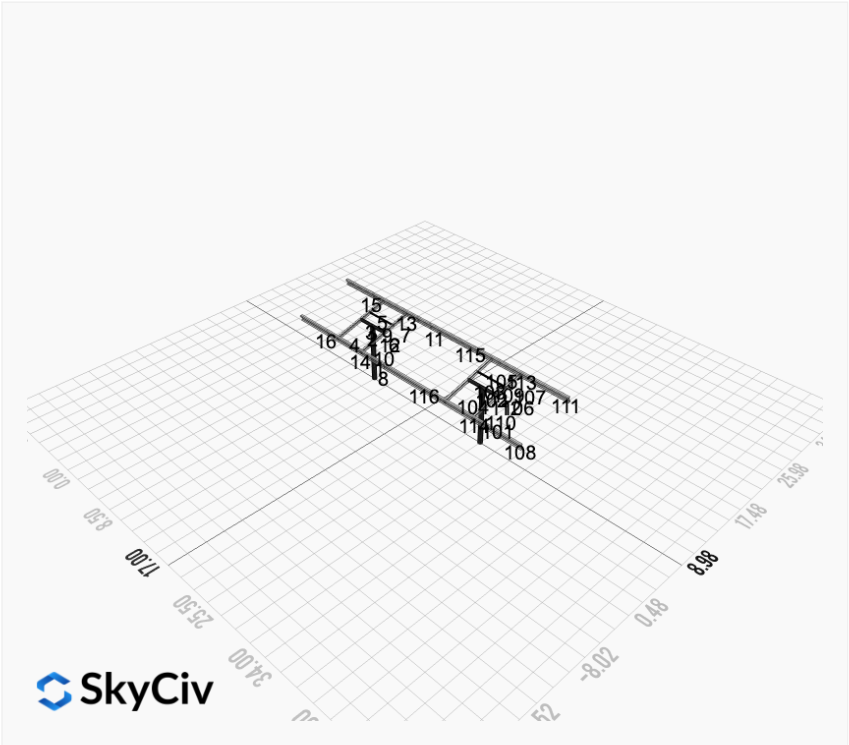
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Design Notes:

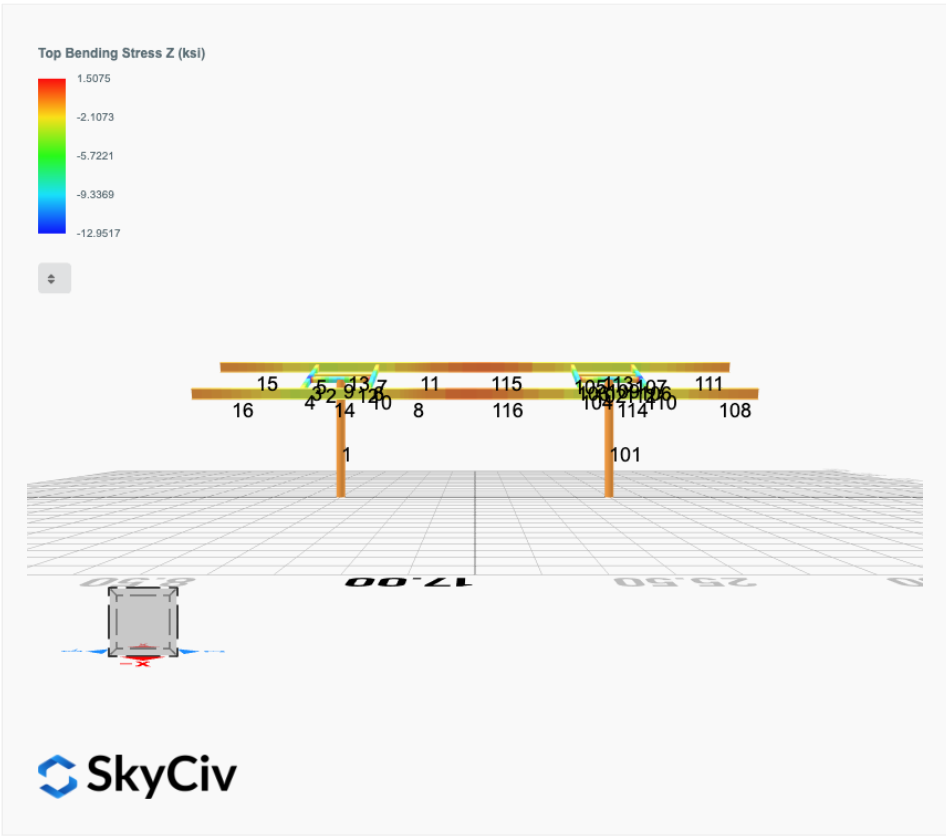
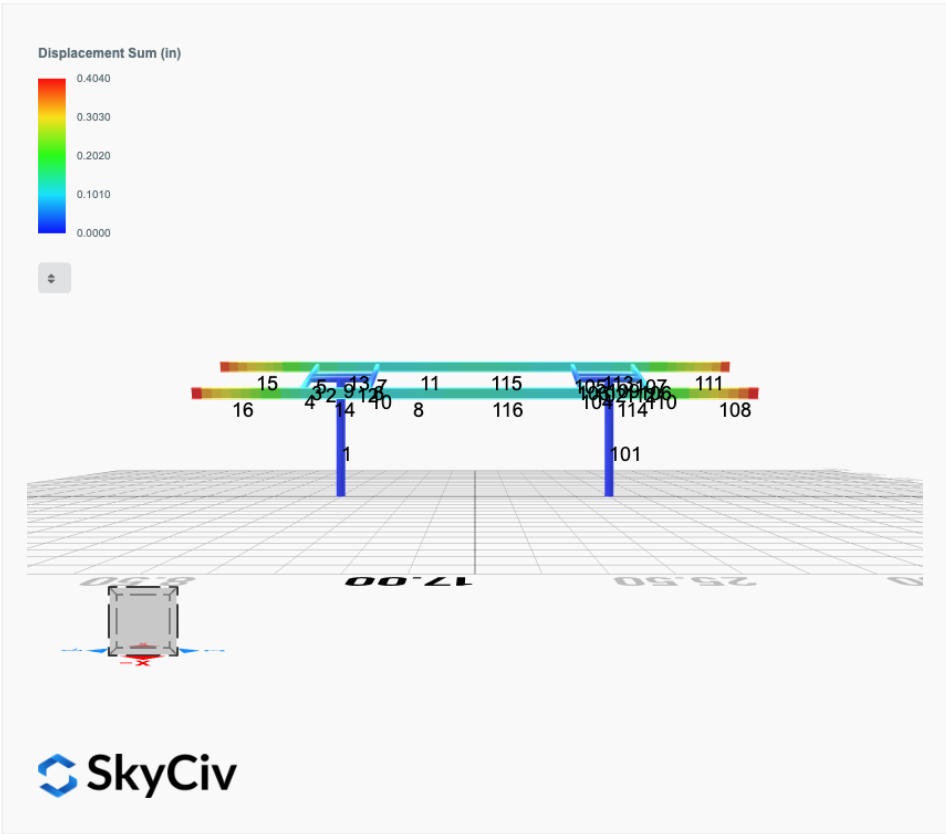
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only



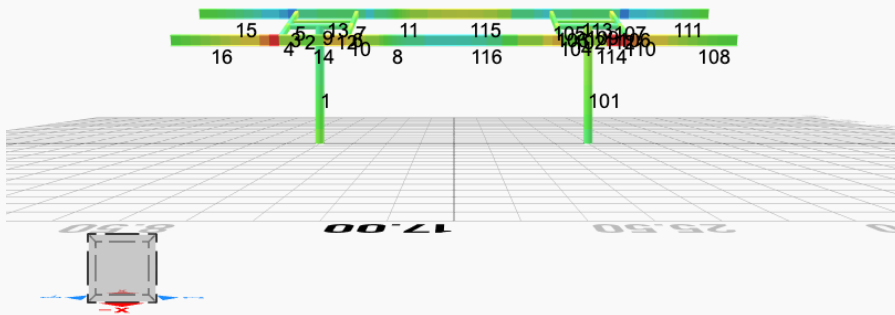
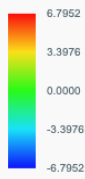




FEM Results (Envelope Worst Case for each member)

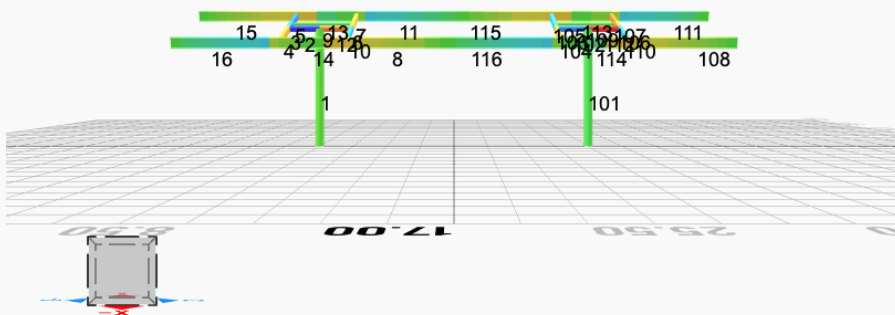


Top Bending Stress Y (ksi)

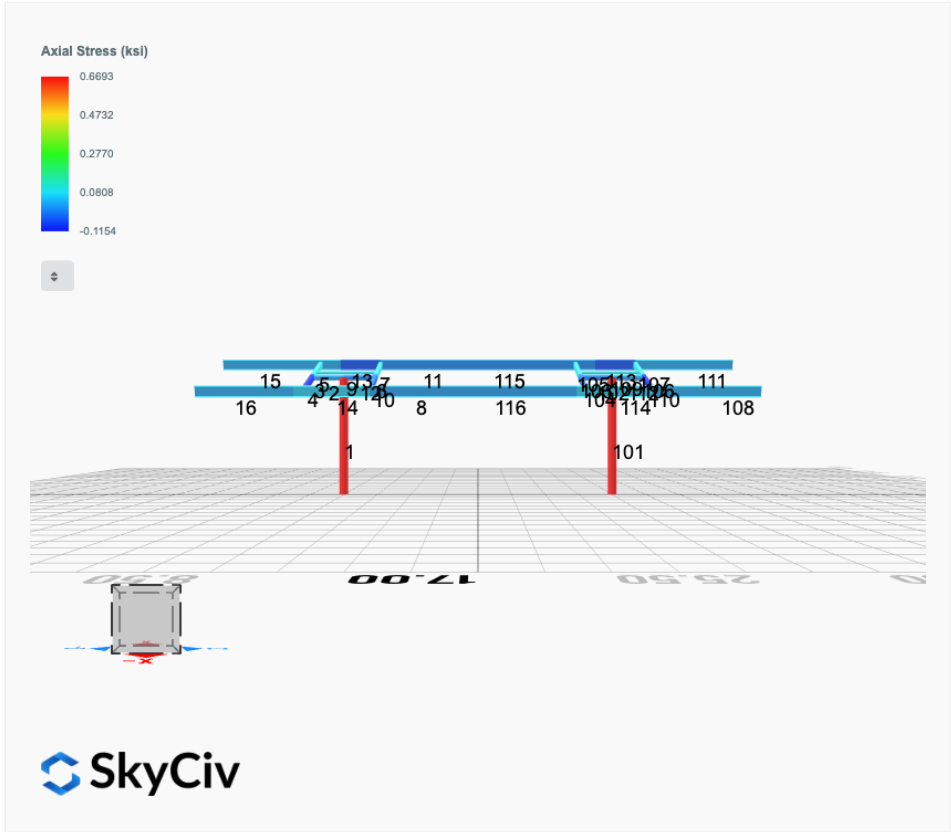


SkyCiv

Shear Stress Y (ksi)



SkyCiv



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	2.1708	-0.0797	-0.1603	0.0427	0.0291
ULS: 2. D + L	-0.0000	2.1708	-0.0797	-0.1603	0.0427	0.0291
ULS: 3. D + (S or Lr or R)	-0.0000	5.9066	-0.2430	-0.4891	0.1301	0.0383
ULS: 3. D + (S or Lr or R)	-0.0000	2.1708	-0.0797	-0.1603	0.0427	0.0291
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.9726	-0.2022	-0.4069	0.1082	0.0360
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.1708	-0.0797	-0.1603	0.0427	0.0291
ULS: 5b. D + 0.7E	-0.0000	2.1708	-0.0797	-0.1603	0.0427	0.0291
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.9726	-0.2022	-0.4069	0.1082	0.0360
ULS: 8. 0.6D + 0.7E	-0.0000	1.3025	-0.0478	-0.0962	0.0256	0.0175
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.7504	4.9712	-0.2094	-0.4195	0.1263	6.8445
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.7504	4.9712	-0.2094	-0.4195	0.1263	6.8445
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5693	0.0463	0.0181	0.0346	-0.0204	-2.6805
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4916	0.3360	0.0058	0.0102	-0.0125	-10.6081
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5628	7.0730	-0.2995	-0.6013	0.1709	5.1475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5628	7.0730	-0.2995	-0.6013	0.1709	5.1475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4269	3.3792	-0.1289	-0.2607	0.0609	-1.9962
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3687	3.5965	-0.1381	-0.2790	0.0668	-7.9419
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5628	4.2711	-0.1770	-0.3547	0.1054	5.1406
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5628	4.2711	-0.1770	-0.3547	0.1054	5.1406
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4269	0.5774	-0.0064	-0.0141	-0.0047	-2.0031
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3687	0.7947	-0.0156	-0.0325	0.0013	-7.9488
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.7504	4.1029	-0.1775	-0.3554	0.1092	6.8328
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.7504	4.1029	-0.1775	-0.3554	0.1092	6.8328
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5693	-0.8220	0.0499	0.0988	-0.0375	-2.6922
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4916	-0.5323	0.0377	0.0743	-0.0296	-10.6198

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	10.9159
Shear X	-1.2506
Shear Z	-0.4666
Moment X	-0.9390
Moment Y (Twist)	0.2609
Moment Z	18.0384

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.0730
Shear X	-0.7504
Shear Z	-0.2995
Moment X	-0.6013
Moment Y (Twist)	0.1709
Moment Z	10.6198

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	2.1708	0.0797	0.1603	-0.0427	0.0291
ULS: 2. D + L	0.0000	2.1708	0.0797	0.1603	-0.0427	0.0291
ULS: 3. D + (S or Lr or R)	0.0000	5.9066	0.2430	0.4891	-0.1301	0.0383
ULS: 3. D + (S or Lr or R)	0.0000	2.1708	0.0797	0.1603	-0.0427	0.0291
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.9726	0.2022	0.4069	-0.1082	0.0360
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.1708	0.0797	0.1603	-0.0427	0.0291
ULS: 5b. D + 0.7E	0.0000	2.1708	0.0797	0.1603	-0.0427	0.0291

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.9726	0.2022	0.4069	-0.1082	0.0360
ULS: 8. 0.6D + 0.7E	0.0000	1.3025	0.0478	0.0962	-0.0256	0.0175
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.7504	4.9712	0.2094	0.4195	-0.1263	6.8445
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.7504	4.9712	0.2094	0.4195	-0.1263	6.8445
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5693	0.0463	-0.0181	-0.0346	0.0204	-2.6805
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4916	0.3360	-0.0058	-0.0102	0.0125	-10.6081
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5628	7.0730	0.2995	0.6013	-0.1709	5.1475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5628	7.0730	0.2995	0.6013	-0.1709	5.1475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4269	3.3792	0.1289	0.2607	-0.0609	-1.9962
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3687	3.5965	0.1381	0.2790	-0.0668	-7.9419
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5628	4.2711	0.1770	0.3547	-0.1054	5.1406
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5628	4.2711	0.1770	0.3547	-0.1054	5.1406
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4269	0.5774	0.0064	0.0141	0.0047	-2.0031
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3687	0.7947	0.0156	0.0325	-0.0013	-7.9488
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.7504	4.1029	0.1775	0.3554	-0.1092	6.8328
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.7504	4.1029	0.1775	0.3554	-0.1092	6.8328
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5693	-0.8220	-0.0499	-0.0988	0.0375	-2.6922
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4916	-0.5323	-0.0377	-0.0743	0.0296	-10.6198

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	10.9159
Shear X	-1.2506
Shear Z	0.4666
Moment X	0.9391
Moment Y (Twist)	0.2609
Moment Z	18.0388

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.0730
Shear X	-0.7504
Shear Z	0.2995
Moment X	0.6013
Moment Y (Twist)	0.1709
Moment Z	10.6198

Project Details

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

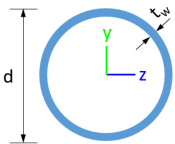
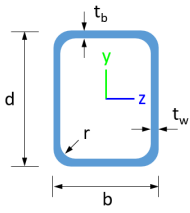
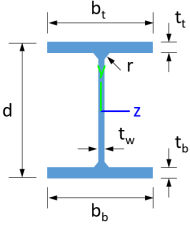
User Name: sales@mtsolar.us
 Project Name: South Pole Solar 30 A2
 Unit System: imperial



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					
7	6in Pipe Sch 40	6.63	0.28					
								
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

7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D
1	7	15.61	15.61	7.43	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.20,1.17,1.18,1.18,1.16,1.18,1.18,1.18,1.18,1.18,1.18,1.21,1.16,1.18,1.18,1.17,1.18	300	200	1
4	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.57,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.67,2.71,1.68,1.67,1.67,1.66,1.68	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.69,1.66,1.67,1.67,1.64,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.65,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.17,1.18,1.18,1.15,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.16,1.18,1.18,1.16,1.17	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.69,1.66,1.67,1.67,1.64,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.64,1.67,1.67,1.65,1.66	300	200	1
8	19	1.33	1.33	2.05	1.91,1.91,1.91,1.91,1.91,1.91,1.90,1.90,1.96,1.98,1.89,1.89,1.91,2.05,1.90,1.90,1.90,1.92,1.90,1.90,2.03,1.98,1.89,1.89,1.91,2.10	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.57,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.16,1.68,1.67,1.67,1.66,1.68	300	200	1
11	19	1.33	1.33	2.05	1.92,1.92,1.92,1.92,1.92,1.92,1.92,1.92,2.02,2.03,1.92,1.92,1.84,1.98,1.92,1.92,1.93,1.89,1.92,1.92,2.00,2.04,1.92,1.92,1.87,1.97	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.14,1.23,1.09,1.09,1.10,1.16,1.09,1.09,1.09,1.10,1.09,1.09,1.11,1.26,1.09,1.09,1.10,1.15	300	200	1
14	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.31,1.15,1.09,1.09,1.10,1.24,1.09,1.09,1.09,1.09,1.09,1.09,3.30,1.14,1.09,1.09,1.09,1.32	300	200	1
15	19	9.97	9.97	4.75	2.33,2.33	300	200	1
16	19	9.97	9.97	4.75	2.33,2.33	300	200	1
101	7	15.61	15.61	7.43	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.17,1.18,1.18,1.15,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.16,1.18,1.18,1.16,1.17	300	200	1
104	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.57,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.16,1.68,1.67,1.67,1.66,1.68	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.69,1.66,1.67,1.67,1.64,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.64,1.67,1.67,1.65,1.66	300	200	1
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.20,1.17,1.18,1.18,1.16,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.21,1.16,1.18,1.18,1.17,1.18	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.69,1.66,1.67,1.67,1.64,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.65,1.67,1.67,1.66,1.66	300	200	1

Member Design Capacity

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115	133.20	93.89	32.87	6.12	40.24	43.62
116	133.20	93.89	32.87	6.12	40.24	43.62

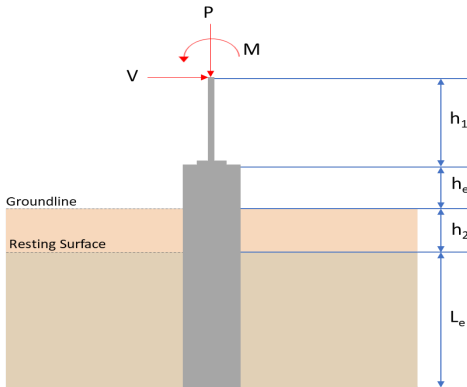
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.072	0.426	0.060	0.017	0.006	0.433	#16	0.417	Not Required	Pass
2	0.001	0.488	0.071	0.099	0.011	0.548	#21	0.053	Not Required	Pass
3	0.005	0.688	0.052	0.070	0.011	0.741	#21	0.045	Not Required	Pass
4	0.006	0.654	0.076	0.066	0.015	0.685	#21	0.080	Not Required	Pass
5	0.005	0.426	0.097	0.069	0.025	0.454	#21	0.074	Not Required	Pass
6	0.005	0.579	0.014	0.058	0.002	0.587	#21	0.045	Not Required	Pass
7	0.005	0.360	0.056	0.058	0.014	0.370	#21	0.074	Not Required	Pass
8	0.001	0.065	0.040	0.035	0.007	0.103	#21	0.095	Not Required	Pass
9	0.006	0.085	0.045	0.002	0.002	0.132	#21	0.204	Not Required	Pass
10	0.004	0.548	0.078	0.055	0.019	0.620	#21	0.080	Not Required	Pass
11	0.002	0.067	0.043	0.037	0.007	0.102	#21	0.095	Not Required	Pass
12	0.002	0.373	0.060	0.082	0.010	0.422	#21	0.035	Not Required	Pass
13	0.004	0.268	0.214	0.050	0.009	0.477	#21	0.286	Not Required	Pass
14	0.003	0.262	0.214	0.048	0.009	0.464	#21	0.190	Not Required	Pass
15	0.000	0.106	0.114	0.037	0.007	0.221	#21	Not Required	Not Required	Pass
16	0.000	0.101	0.114	0.035	0.007	0.216	#21	Not Required	Not Required	Pass
101	0.072	0.426	0.060	0.017	0.006	0.433	#16	0.417	Not Required	Pass
102	0.002	0.373	0.060	0.082	0.010	0.422	#21	0.035	Not Required	Pass
103	0.005	0.579	0.014	0.058	0.002	0.587	#21	0.045	Not Required	Pass
104	0.004	0.548	0.078	0.055	0.019	0.620	#21	0.080	Not Required	Pass
105	0.005	0.360	0.056	0.058	0.014	0.370	#21	0.074	Not Required	Pass
106	0.005	0.688	0.052	0.070	0.011	0.741	#21	0.045	Not Required	Pass
107	0.005	0.426	0.097	0.069	0.025	0.454	#21	0.074	Not Required	Pass
108	0.000	0.101	0.114	0.035	0.007	0.216	#21	Not Required	Not Required	Pass
109	0.006	0.085	0.045	0.002	0.002	0.132	#21	0.204	Not Required	Pass
110	0.006	0.654	0.076	0.066	0.015	0.685	#21	0.080	Not Required	Pass
111	0.000	0.106	0.114	0.037	0.007	0.221	#21	Not Required	Not Required	Pass
112	0.001	0.488	0.071	0.099	0.011	0.548	#21	0.053	Not Required	Pass
113	0.004	0.268	0.214	0.050	0.009	0.477	#21	0.190	Not Required	Pass
114	0.003	0.262	0.214	0.048	0.009	0.464	#21	0.286	Not Required	Pass
115	0.002	0.067	0.080	0.037	0.007	0.120	#21	0.346	Not Required	Pass
116	0.001	0.065	0.077	0.035	0.007	0.114	#21	0.346	Not Required	Pass

Definitions

Φ _t	Safety factor for tensile
Φ _c	Safety factor for compression
Φ _b	Safety factor for flexure
Φ _v	Safety factor for shear
E	Modulus of elasticity
F _y	Specified minimum yield stress
F _u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I _{yp}	Moment of inertia about the Y axes
I _{zp}	Moment of inertia about the Z axes
I _w	Warping constant
S _{yp}	Plastic section modulus about the Y axis
S _{zp}	Plastic section modulus about the Z axis
KL	Effective length
C _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 = 5 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>7.073</td><td>10.916</td></tr><tr><td>Vx (kip)</td><td>-0.750</td><td>-1.251</td></tr><tr><td>Vz (kip)</td><td>-0.299</td><td>-0.467</td></tr><tr><td>Mx (kipft)</td><td>-0.601</td><td>-0.939</td></tr><tr><td>Mz (kipft)</td><td>10.620</td><td>18.038</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)	7.073	10.916	Vx (kip)	-0.750	-1.251	Vz (kip)	-0.299	-0.467	Mx (kipft)	-0.601	-0.939	Mz (kipft)	10.620	18.038	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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Mx (kipft)	-0.601	-0.939																										
Mz (kipft)	10.620	18.038																										
	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, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.75 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.11943 \text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(10.62 \text{ kipft}) + ((-0.75 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 1.6911 \text{ kipft/ft}$ 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: $L_{e,x} = 4.6696 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.299 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.047611 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.601 \text{ kipft}) + ((-0.299 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.095701 \text{ kipft/ft}$ 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: $L_{e,z} = 1.4996 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(4.6696 \text{ ft}), (1.4996 \text{ ft})]$ $L_{e,req} = 4.67 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_c - h_2$ $L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5 \text{ ft}$ <i>Ratio</i> - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(4.67 \text{ ft})}{(5 \text{ ft})}$ $Ratio = 0.934$	Status: PASS Ratio: 0.930
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(7.073 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.44206 \text{ kips/ft}^2$	

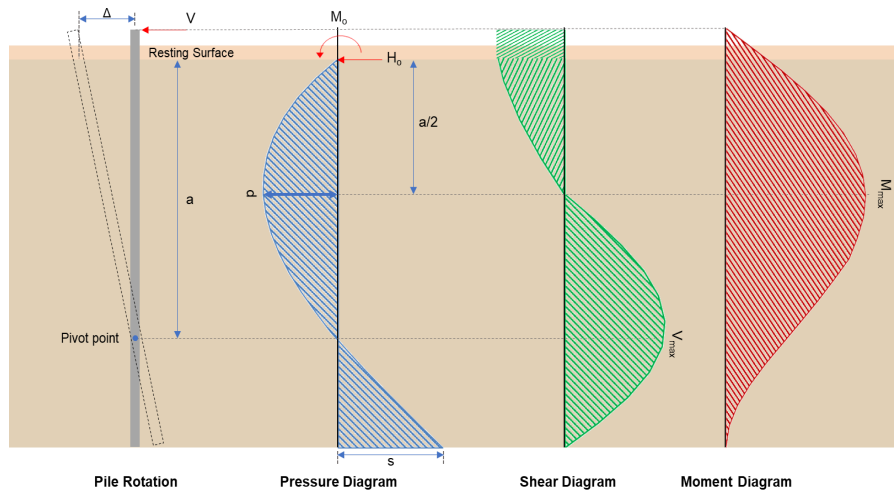
	$q = 0.44206 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.44206 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.22103$	Status: PASS Ratio: 0.220
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.25$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.11943 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 1.6911 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (1.6911 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.6911 \text{ kipft/ft})) + (4 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.4127 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (1.6911 \text{ kipft/ft})) + (3 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^3 \times [(3 \times (1.6911 \text{ kipft/ft})) + (2 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.19126 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (1.6911 \text{ kipft/ft})) + ((-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.66841 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.4127 \text{ ft})}{2}$ $p_a = 0.25595 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.19126 \text{ kip/ft}^2)}{(0.25595 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.74725$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.750

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.66841 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.89121$	Status: PASS Ratio: 0.890
	Considering z-direction: $H_o = -0.047611 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.095701 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.095701 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.047611 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.095701 \text{ kipft/ft})) + (4 \times (-0.047611 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.5933 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.095701 \text{ kipft/ft})) + (3 \times (-0.047611 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^3 \times [(3 \times (0.095701 \text{ kipft/ft})) + (2 \times (-0.047611 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = -0.017428 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.095701 \text{ kipft/ft})) + ((-0.047611 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = -0.011197 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.5933 \text{ ft})}{2}$ $p_a = 0.26949 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.017428 \text{ kip/ft}^2)}{(0.26949 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.06467$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.060

$$Ratio = \frac{(-0.011197 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = -0.01493$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.251 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.1992 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.038 \text{ kipft}) + ((-1.251 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.8723 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.8723 \text{ kipft/ft})}{(-0.1992 \text{ kip/ft})}$$

$$E = 14.419 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.8723 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.1992 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.8723 \text{ kipft/ft})) + (4 \times (-0.1992 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4116 \text{ ft}$$

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

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

$$V_{max} = ((-0.1992 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.419 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4116 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.419 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4116 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.5951 \text{ kip}$$

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

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

$$M_{max} = ((-0.1992 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(14.419 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4116 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.419 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4116 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.419 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4116 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.124 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.467 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.074363 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((-0.467 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14952 \text{ kipft/ft})}{(-0.074363 \text{ kip/ft})}$$

$$E = 2.0107 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14952 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.074363 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.14952 \text{ kipft/ft})) + (4 \times (-0.074363 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5932 \text{ ft}$$

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

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

$$V_{max} = ((-0.074363 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5932 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5932 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.41051 \text{ kip}$$

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

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

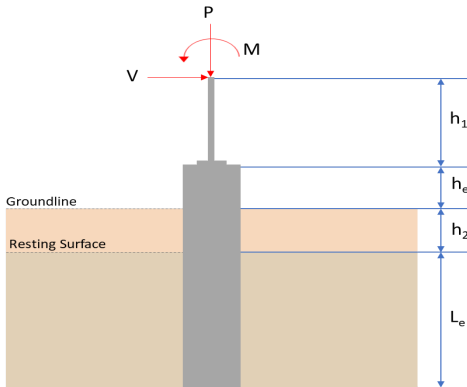
$$M_{max} = ((-0.074363 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(2.0107 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5932 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5932 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5932 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.89401 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(10.916 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.233 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.233 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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 $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.916 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0040805$	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 $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.916 \text{ kip} \rightarrow 10916 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(10916 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.94 \text{ kip}$	
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 $V_c = Min [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = Min [(296.21 \text{ kip}), (119.94 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.94 \text{ kip}$	

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 $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.94 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.04 \text{ kip}$ Considering x-direction: $V_{max} = 4.5951 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.5951 \text{ kip})}{(111.04 \text{ kip})}$ $Ratio = 0.041382$ Considering z-direction: $V_{max} = 0.41051 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.41051 \text{ kip})}{(111.04 \text{ kip})}$ $Ratio = 0.0036969$	Status: PASS Ratio: 0.040 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: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ 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: $M_{max} = 11.124 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.124 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.044567$	Status: PASS Ratio: 0.040
	Considering z-direction: $M_{max} = 0.89401 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.89401 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0035818$	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 = 5 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>7.073</td><td>10.916</td></tr><tr><td>Vx (kip)</td><td>-0.750</td><td>-1.251</td></tr><tr><td>Vz (kip)</td><td>0.299</td><td>0.467</td></tr><tr><td>Mx (kipft)</td><td>0.601</td><td>0.939</td></tr><tr><td>Mz (kipft)</td><td>10.620</td><td>18.039</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)	7.073	10.916	Vx (kip)	-0.750	-1.251	Vz (kip)	0.299	0.467	Mx (kipft)	0.601	0.939	Mz (kipft)	10.620	18.039	
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Mx (kipft)	0.601	0.939																										
Mz (kipft)	10.620	18.039																										
	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, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.75 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.11943 \text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(10.62 \text{ kipft}) + ((-0.75 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 1.6911 \text{ kipft/ft}$ 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: $L_{e,x} = 4.6696 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.299 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.047611 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.601 \text{ kipft}) + ((0.299 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.095701 \text{ kipft/ft}$ 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: $L_{e,z} = 2.4466 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(4.6696 \text{ ft}), (2.4466 \text{ ft})]$ $L_{e,req} = 4.67 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(4.67 \text{ ft})}{(5 \text{ ft})}$ $\text{Ratio} = 0.934$	Status: PASS Ratio: 0.930
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(7.073 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.44206 \text{ kips/ft}^2$	

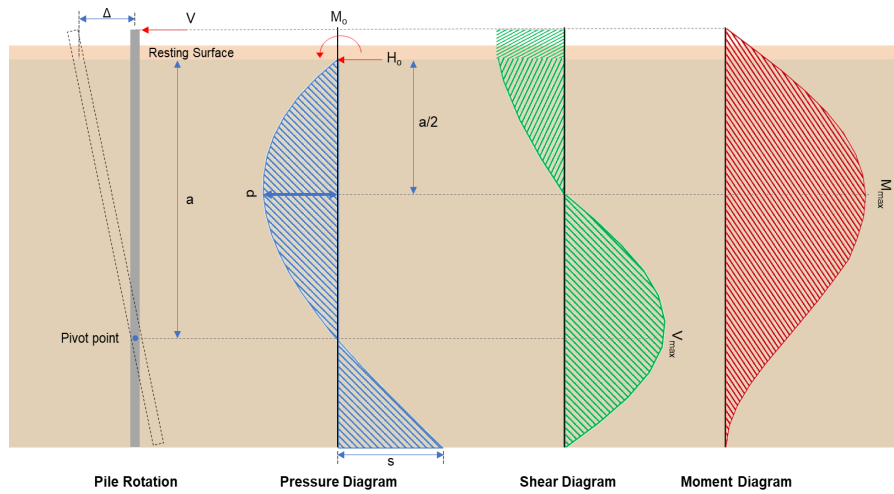
	$q = 0.44206 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.44206 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.22103$	Status: PASS Ratio: 0.220
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.25$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.11943 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 1.6911 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (1.6911 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.6911 \text{ kipft/ft})) + (4 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.4127 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (1.6911 \text{ kipft/ft})) + (3 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^3 \times [(3 \times (1.6911 \text{ kipft/ft})) + (2 \times (-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.19126 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (1.6911 \text{ kipft/ft})) + ((-0.11943 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.66841 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.4127 \text{ ft})}{2}$ $p_a = 0.25595 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.19126 \text{ kip/ft}^2)}{(0.25595 \text{ kip/ft}^2)}$ $Ratio = 0.74725$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.750

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.66841 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.89121$	Status: PASS Ratio: 0.890
	Considering z-direction: $H_o = 0.047611 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.095701 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.095701 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.047611 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.095701 \text{ kipft/ft})) + (4 \times (0.047611 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.5933 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.095701 \text{ kipft/ft})) + (3 \times (0.047611 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^3 \times [(3 \times (0.095701 \text{ kipft/ft})) + (2 \times (0.047611 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.047301 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.095701 \text{ kipft/ft})) + ((0.047611 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.10307 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.5933 \text{ ft})}{2}$ $p_a = 0.26949 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.047301 \text{ kip/ft}^2)}{(0.26949 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.17552$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.180

$$Ratio = \frac{(0.10307 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.13743$$

Status: **PASS**
Ratio: **0.140**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.251 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.1992 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.039 \text{ kipft}) + ((-1.251 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.8725 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.8725 \text{ kipft/ft})}{(-0.1992 \text{ kip/ft})}$$

$$E = 14.42 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.8725 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.1992 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.8725 \text{ kipft/ft})) + (4 \times (-0.1992 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4116 \text{ ft}$$

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

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

$$V_{max} = ((-0.1992 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.42 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4116 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.42 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4116 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.5953 \text{ kip}$$

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

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

$$M_{max} = ((-0.1992 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(14.42 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4116 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.42 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4116 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.42 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4116 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.124 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.467 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.074363 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((0.467 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14952 \text{ kipft/ft})}{(0.074363 \text{ kip/ft})}$$

$$E = 2.0107 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14952 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.074363 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.14952 \text{ kipft/ft})) + (4 \times (0.074363 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5932 \text{ ft}$$

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

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

$$V_{max} = ((0.074363 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5932 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5932 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.41051 \text{ kip}$$

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

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

$$M_{max} = ((0.074363 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(2.0107 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5932 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5932 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0107 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5932 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.89401 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(10.916 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.233 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.233 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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 $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(10.916 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0040805$	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.916 \text{ kip} \rightarrow 10916 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(10916 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.94 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (119.94 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.94 \text{ kip}$	

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 $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.94 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.04 \text{ kip}$ Considering x-direction: $V_{max} = 4.5953 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.5953 \text{ kip})}{(111.04 \text{ kip})}$ $Ratio = 0.041384$ Considering z-direction: $V_{max} = 0.41051 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.41051 \text{ kip})}{(111.04 \text{ kip})}$ $Ratio = 0.0036969$	Status: PASS Ratio: 0.040 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: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ 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: $M_{max} = 11.124 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.124 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.044569$	Status: PASS Ratio: 0.040
	Considering z-direction: $M_{max} = 0.89401 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.89401 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0035818$	Status: PASS Ratio: 0.000