

Your Project Calculations



Project Name: MTSOLAR_LI147LBJ34K

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_LI147LBJ34K&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=5m8jxS10VpWGOaLK065PJQOIAbEGXgZe7KNuntqx5bOnNfdqR2WEWyzCDdWFj

Array Specification

Product:	Beam
Unique ID:	2P-15-8TOP-SD-24-L-5Hx4W-46BI
Duty Classification:	SD
Module Width:	42.00 in
Module Length:	84.00in
Number of Rows:	5
Number of Columns:	4
Total Number of Modules:	20
Desired Tilt Angle:	65
Front Edge Clearance:	5
Total Array Height at Tilt:	20.95 ft
Total Frame Length:	26.50 ft
Frame Weight:	1911 lbs
Array Dimensions N/S:	17.71 ft
Array Dimensions E/W:	28.33 ft
Rail Length:	212.50 in
Rail Spacing:	3.50 ft
Rail Check:	Not Checked

Support Specifications

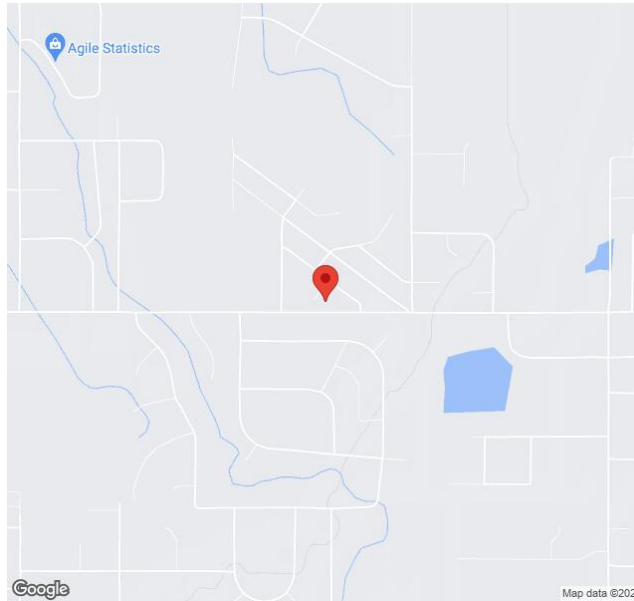
Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	13.02 ft
Number of Poles:	2
Pole Spacing:	15 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.50 ft Pile 2: 7.50 ft
Foundation Volume:	8.889 y ³
Foundation Result:	PASSED
Mount Twist:	0.624769 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1999 Cyprus Ct, North Pole, AK 99705, USA
Wind Speed:	105 mph
Snow Load:	0 psf
Design Uplift Pressure:	0.025824 ksf
Design Downforce Pressure:	-0.025824 ksf
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

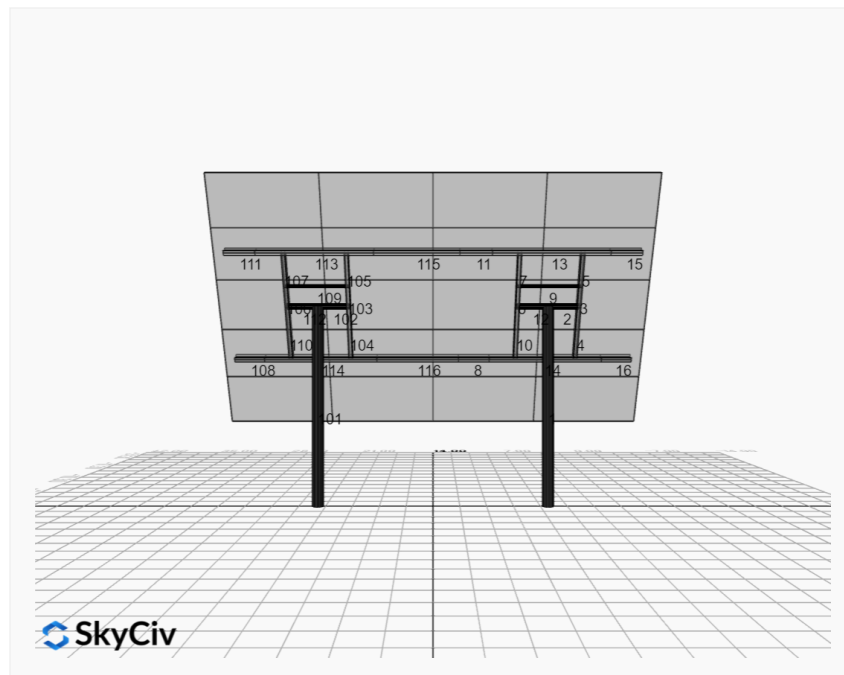
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

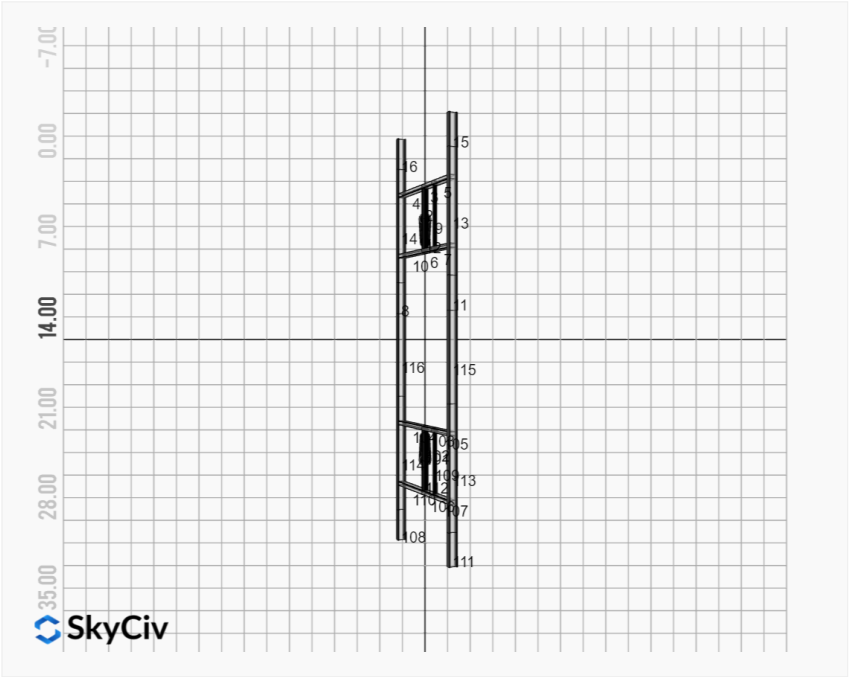
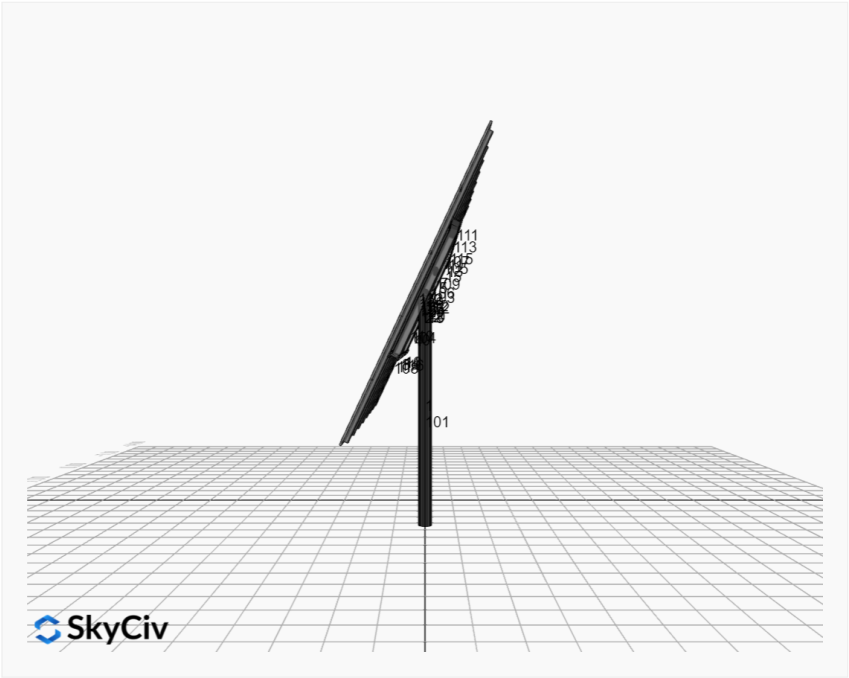
AutoDesigner Input

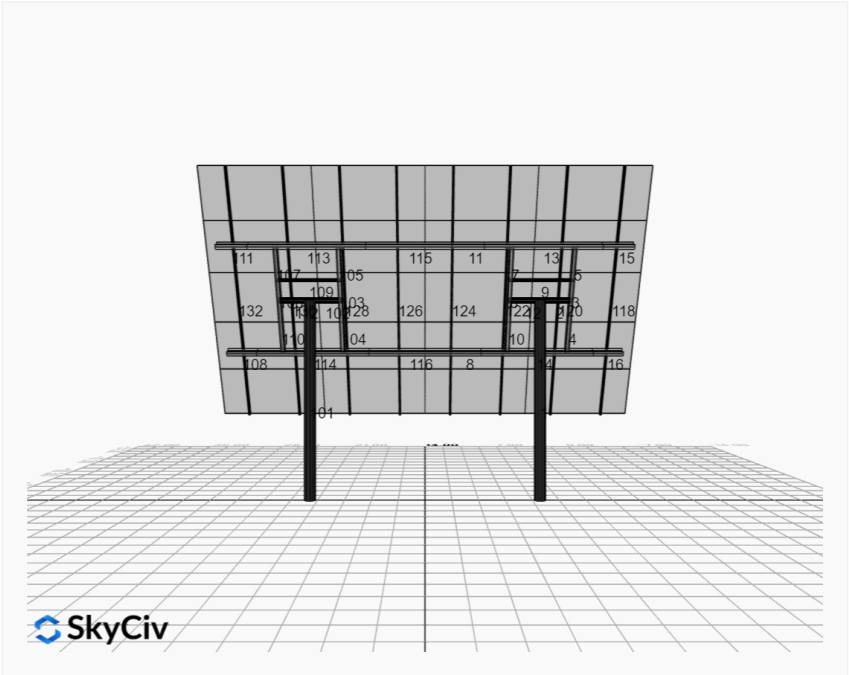
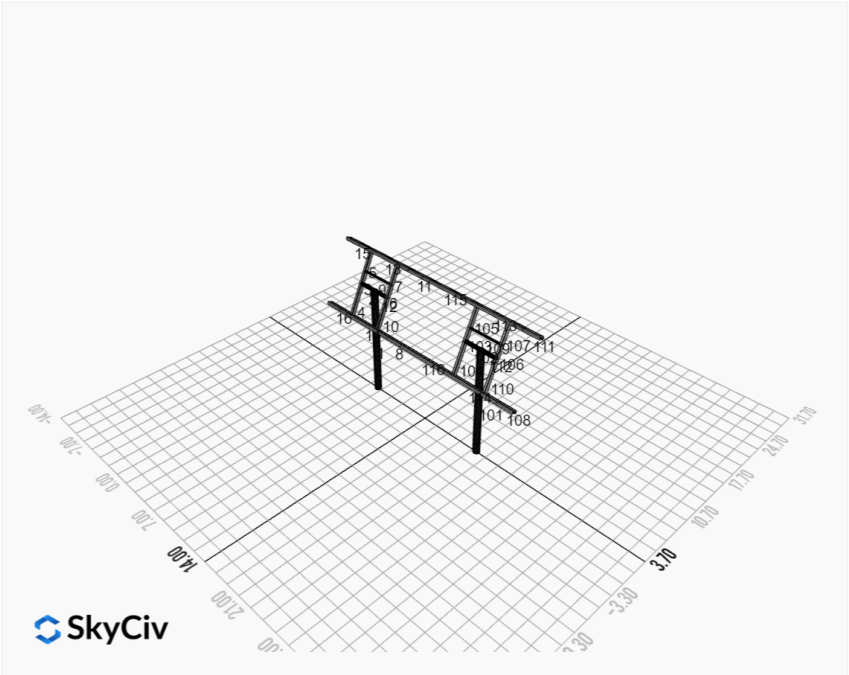
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Design Notes:

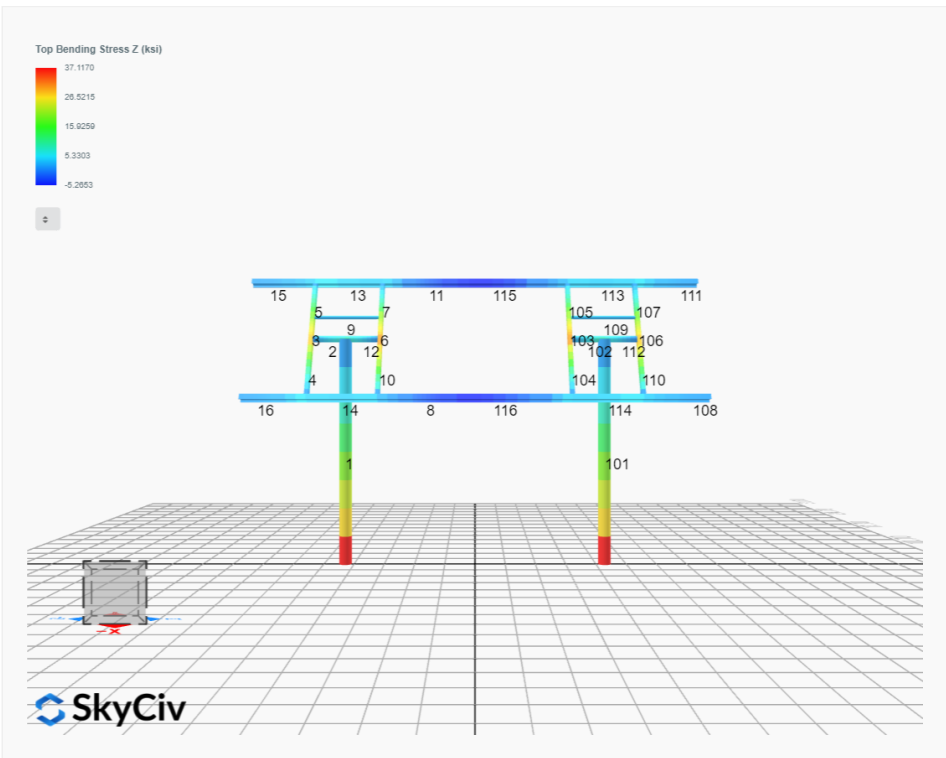
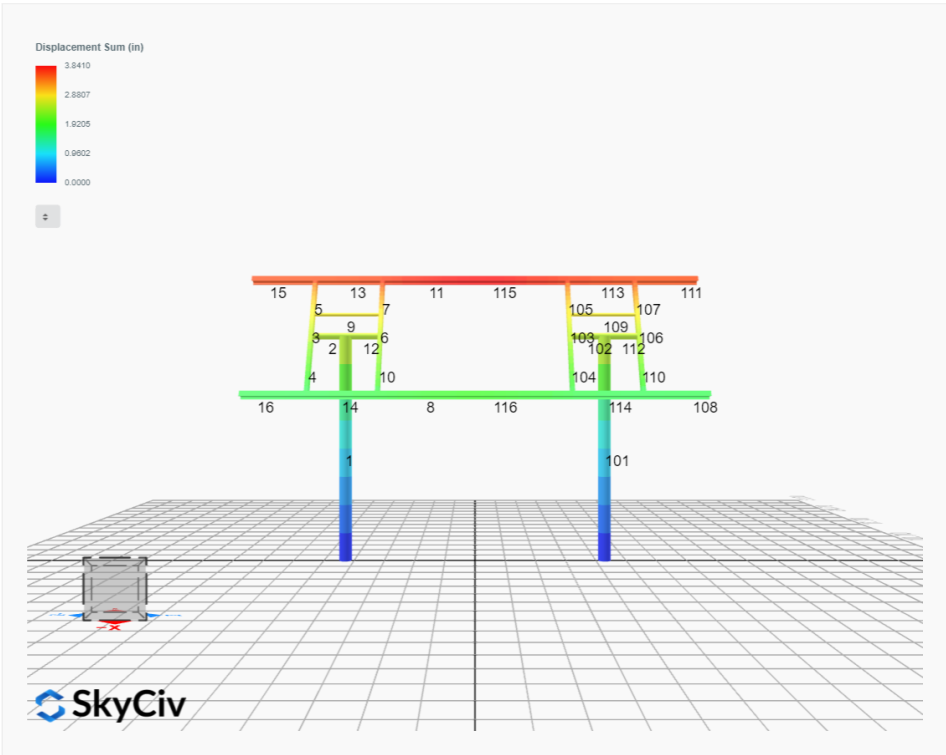
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

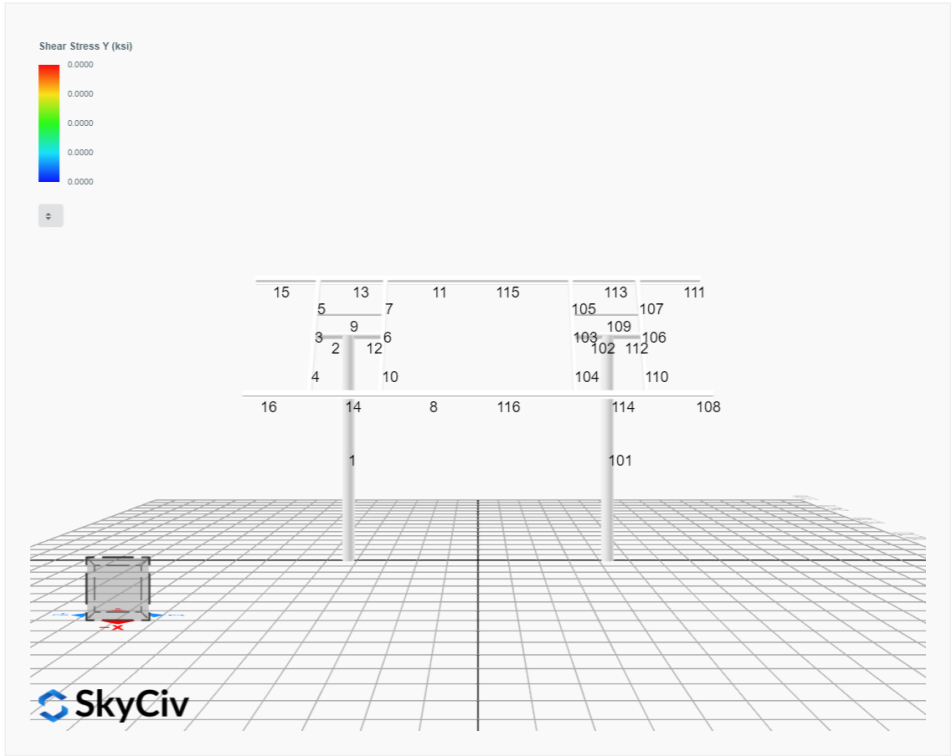
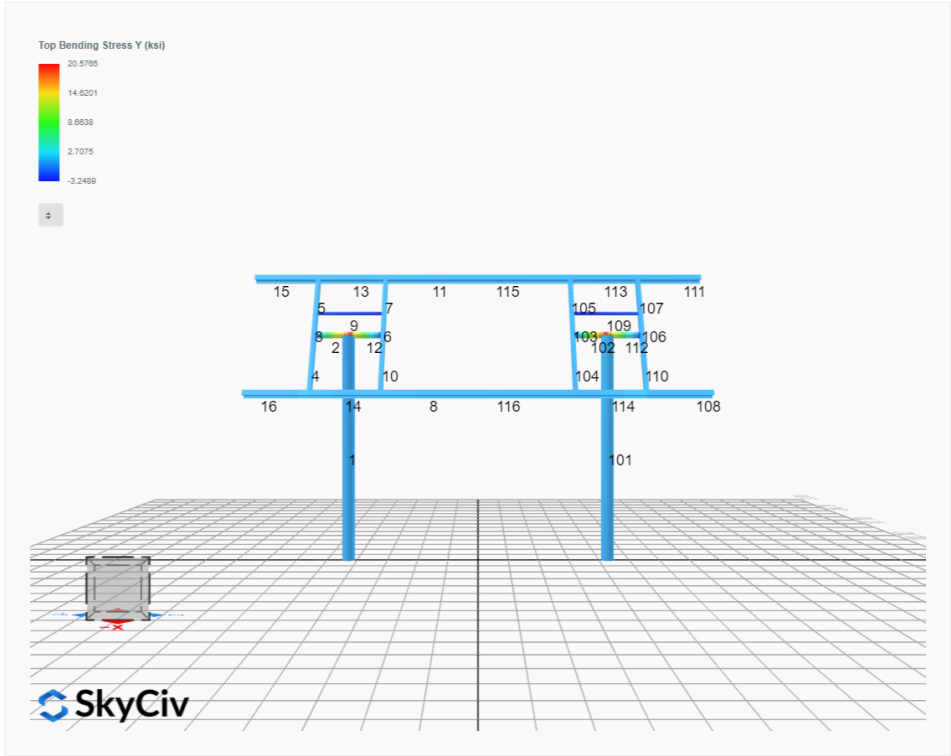


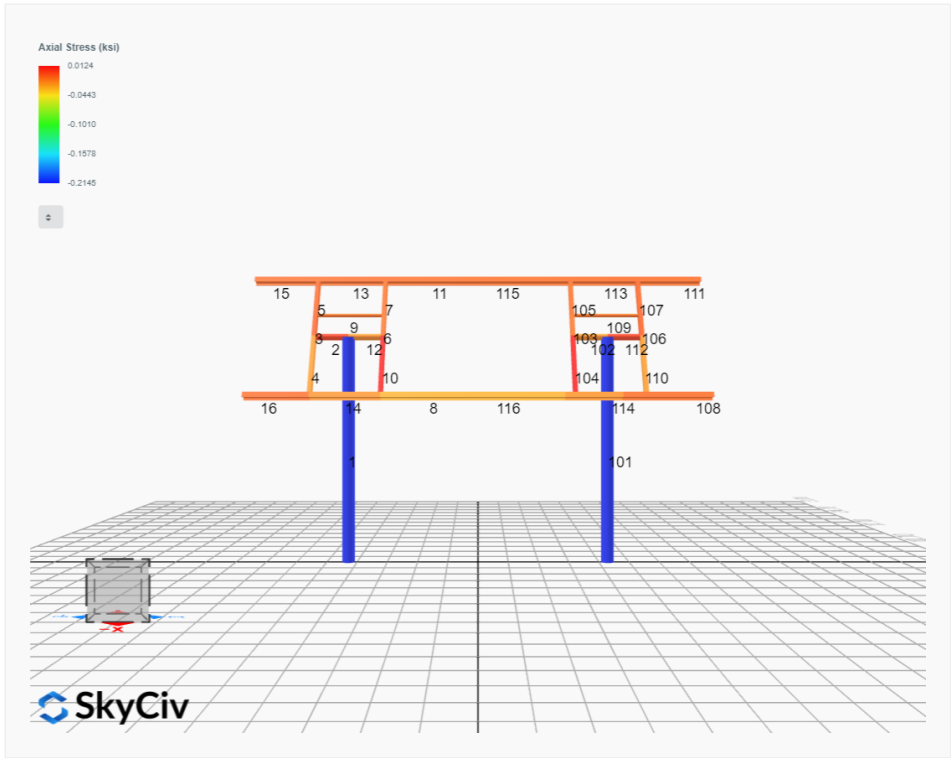




FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 2. D + L	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 3. D + (S or Lr or R)	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 3. D + (S or Lr or R)	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 5b. D + 0.7E	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 8. 0.6D + 0.7E	0.0000	1.2474	0.0124	0.0486	-0.0018	0.0064
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.5228	3.7217	0.0523	0.1928	-0.3768	46.2307
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5228	0.4362	-0.0109	-0.0307	0.3709	-45.5420
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6421	3.3110	0.0444	0.1648	-0.2834	34.6757
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6421	0.8469	-0.0030	-0.0028	0.2774	-34.1538
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6421	3.3110	0.0444	0.1648	-0.2834	34.6757
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6421	0.8469	-0.0030	-0.0028	0.2774	-34.1538
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.0790	0.0207	0.0809	-0.0030	0.0107
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.5228	2.8901	0.0441	0.1604	-0.3756	46.2265
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.2474	0.0124	0.0486	-0.0018	0.0064
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.5228	-0.3954	-0.0192	-0.0630	0.3721	-45.5462
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.2474	0.0124	0.0486	-0.0018	0.0064

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	5.2326
Shear X	-5.8714
Shear Z	0.0775
Moment X	0.2834
Moment Y (Twist)	0.6249
Moment Z	77.6053

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	3.7217
Shear X	-3.5228
Shear Z	0.0523
Moment X	0.1928
Moment Y (Twist)	0.3768
Moment Z	46.2307

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	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 2. D + L	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 3. D + (S or Lr or R)	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 3. D + (S or Lr or R)	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 5b. D + 0.7E	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 8. 0.6D + 0.7E	-0.0000	1.2474	-0.0124	-0.0486	0.0018	0.0064
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.5228	3.7217	-0.0523	-0.1928	0.3768	46.2308
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5228	0.4362	0.0109	0.0307	-0.3709	-45.5420
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6421	3.3110	-0.0444	-0.1648	0.2834	34.6757
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6421	0.8469	0.0030	0.0028	-0.2774	-34.1538
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6421	3.3110	-0.0444	-0.1648	0.2834	34.6757
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6421	0.8469	0.0030	0.0028	-0.2774	-34.1538
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.0790	-0.0207	-0.0809	0.0030	0.0107
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.5228	2.8901	-0.0441	-0.1604	0.3756	46.2265
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.2474	-0.0124	-0.0486	0.0018	0.0064
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.5228	-0.3954	0.0192	0.0630	-0.3721	-45.5462
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.2474	-0.0124	-0.0486	0.0018	0.0064

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	5.2326
Shear X	-5.8714
Shear Z	-0.0775
Moment X	-0.2834
Moment Y (Twist)	0.6248
Moment Z	77.6062

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	3.7217
Shear X	-3.5228
Shear Z	-0.0523
Moment X	-0.1928
Moment Y (Twist)	0.3768
Moment Z	46.2308

Project Details

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

User Name: sales@mtsolar.us
 Project Name: MTSOLAR_LI147LBJ34K
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions								
ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
10	8in Pipe Sch 80	8.63	0.50					
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31

10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

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	10	27.35	27.35	13.02	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1
8	18	1.33	1.33	2.05	1.29,1.29,1.29,1.29,1.29,1.29,1.28,1.29,1.28,1.29,1.28,1.29,1.28,1.29,1.28,1.29,1.28,1.29,1.28,1.29,1.28,1.29,1.28,1.29	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1
11	18	1.33	1.33	2.05	1.30,1.30,1.30,1.30,1.30,1.30,1.28,1.30,1.28,1.30,1.28,1.30,1.28,1.30,1.28,1.30,1.28,1.30,1.28,1.30,1.28,1.30,1.28,1.30	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.23,1.21,1.24,1.21,1.23,1.21,1.24,1.21,1.23,1.21,1.24,1.21,1.23,1.21,1.24,1.21,1.23,1.21	300	200	1
14	18	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.22,1.21,1.23,1.21,1.22,1.21,1.23,1.21,1.22,1.21,1.23,1.21,1.22,1.21,1.23,1.21,1.22,1.21	300	200	1
15	18	4.20	4.20	2.00	2.33,2.33	300	200	1
16	18	4.20	4.20	2.00	2.33,2.33	300	200	1
101	10	27.35	27.35	13.02	-	300	200	1
102	4	1.30	1.30	2.00	-	300	200	1
103	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1
105	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1
106	15	0.92	0.92	1.42	1.18,1.18	300	200	1
107	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1

Member Design Capacity

SkyCiv AutoDesigner Report - MTSOLAR_LI147LBGJ34K
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115	120.60	102.67	22.18	6.45	30.09	45.74
116	120.60	102.67	22.18	6.45	30.09	45.74

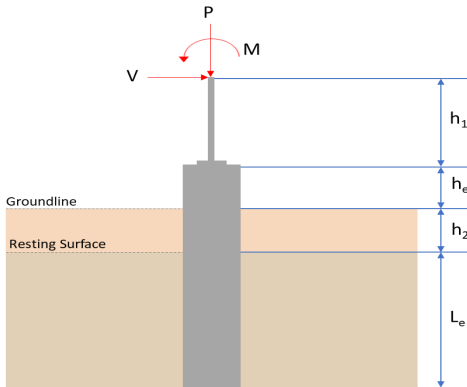
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.024	0.626	0.006	0.034	0.000	0.640	#13	0.570	Not Required	Pass
2	0.000	0.203	0.313	0.049	0.066	0.516	#13	0.052	Not Required	Pass
3	0.006	0.590	0.067	0.059	0.012	0.648	#13	0.044	Not Required	Pass
4	0.005	0.588	0.064	0.059	0.009	0.653	#13	0.078	Not Required	Pass
5	0.005	0.367	0.059	0.059	0.008	0.372	#13	0.073	Not Required	Pass
6	0.007	0.647	0.094	0.065	0.017	0.733	#13	0.044	Not Required	Pass
7	0.007	0.402	0.092	0.065	0.013	0.418	#13	0.073	Not Required	Pass
8	0.001	0.099	0.043	0.034	0.005	0.136	#13	0.088	Not Required	Pass
9	0.001	0.031	0.064	0.002	0.001	0.086	#13	0.132	Not Required	Pass
10	0.007	0.644	0.090	0.065	0.013	0.705	#13	0.078	Not Required	Pass
11	0.001	0.099	0.043	0.034	0.005	0.135	#13	0.088	Not Required	Pass
12	0.001	0.247	0.352	0.057	0.072	0.599	#13	0.052	Not Required	Pass
13	0.001	0.106	0.096	0.049	0.008	0.149	#13	0.265	Not Required	Pass
14	0.002	0.109	0.096	0.049	0.008	0.150	#13	0.177	Not Required	Pass
15	0.000	0.023	0.020	0.018	0.003	0.041	#13	Not Required	Not Required	Pass
16	0.000	0.023	0.020	0.018	0.003	0.041	#13	Not Required	Not Required	Pass
101	0.024	0.626	0.006	0.034	0.000	0.640	#13	0.570	Not Required	Pass
102	0.001	0.247	0.352	0.057	0.072	0.599	#13	0.052	Not Required	Pass
103	0.007	0.647	0.094	0.065	0.017	0.733	#13	0.044	Not Required	Pass
104	0.007	0.644	0.090	0.065	0.013	0.705	#13	0.078	Not Required	Pass
105	0.007	0.402	0.092	0.065	0.013	0.418	#13	0.073	Not Required	Pass
106	0.006	0.590	0.067	0.059	0.012	0.648	#13	0.044	Not Required	Pass
107	0.005	0.367	0.059	0.059	0.008	0.372	#13	0.073	Not Required	Pass
108	0.000	0.023	0.020	0.018	0.003	0.041	#13	Not Required	Not Required	Pass
109	0.001	0.031	0.064	0.002	0.001	0.086	#13	0.132	Not Required	Pass
110	0.005	0.588	0.064	0.059	0.009	0.653	#13	0.078	Not Required	Pass
111	0.000	0.023	0.020	0.018	0.003	0.041	#13	Not Required	Not Required	Pass
112	0.000	0.203	0.313	0.049	0.066	0.516	#13	0.052	Not Required	Pass
113	0.001	0.106	0.096	0.049	0.008	0.149	#13	0.177	Not Required	Pass
114	0.002	0.109	0.096	0.049	0.008	0.150	#13	0.265	Not Required	Pass
115	0.001	0.121	0.057	0.034	0.005	0.169	#13	0.235	Not Required	Pass
116	0.001	0.121	0.057	0.034	0.005	0.171	#13	0.235	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 = 7.5 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>3.722</td><td>5.233</td></tr><tr><td>Vx (kip)</td><td>-3.523</td><td>-5.871</td></tr><tr><td>Vz (kip)</td><td>0.052</td><td>0.077</td></tr><tr><td>Mx (kipft)</td><td>0.193</td><td>0.283</td></tr><tr><td>Mz (kipft)</td><td>46.231</td><td>77.605</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)	3.722	5.233	Vx (kip)	-3.523	-5.871	Vz (kip)	0.052	0.077	Mx (kipft)	0.193	0.283	Mz (kipft)	46.231	77.605	
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	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, $H_o = \frac{V_x}{1.57\text{ }D}$ $H_o = \frac{(-3.523\text{ kip})}{1.57 \times (48\text{ in})}$ $H_o = -0.56099\text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x\text{ }H)}{1.57\text{ }D}$																											

	$M_o = \frac{(46.231 \text{ kipft}) + ((-3.523 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 7.3616 \text{ kipft/ft}$ Required depth of embedment in earth: $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,x} = 7.0567 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.052 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.0082803 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.193 \text{ kipft}) + ((0.052 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.030732 \text{ kipft/ft}$ 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: $L_{e,z} = 1.472 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(7.0567 \text{ ft}), (1.472 \text{ ft})]$ $L_{e,req} = 7.057 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_c - h_2$ $L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 7.5 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(7.057 \text{ ft})}{(7.5 \text{ ft})}$ $Ratio = 0.94093$	Status: PASS Ratio: 0.940
	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 $q = \frac{P_o}{A}$ $q = \frac{(3.722 \text{ kip})}{(16 \text{ ft}^2)}$	

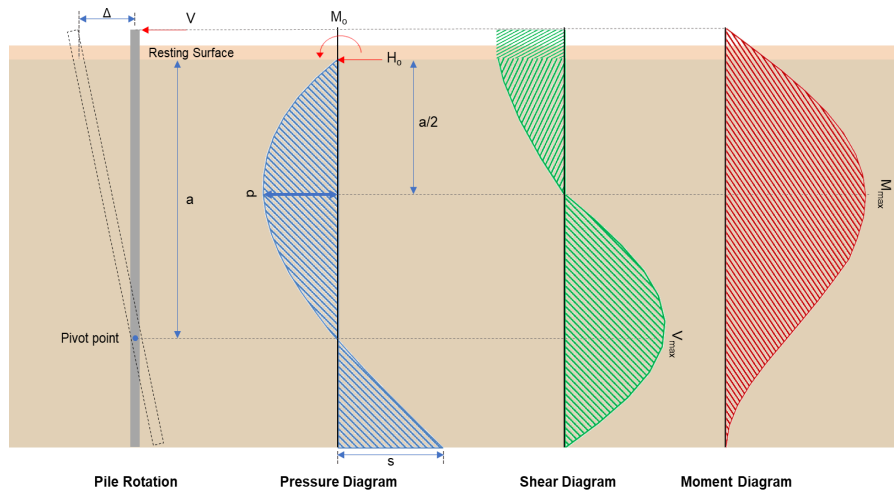
	$q = 0.23262 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.23262 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.11631$	Status: PASS Ratio: 0.120
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.875$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.56099 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 7.3616 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (7.3616 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.3616 \text{ kipft/ft})) + (4 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))}$ $a = 5.1724 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (7.3616 \text{ kipft/ft})) + (3 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^3 \times [(3 \times (7.3616 \text{ kipft/ft})) + (2 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$ $p = 0.27608 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (7.3616 \text{ kipft/ft})) + ((-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$ $s = 1.1217 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.1724 \text{ ft})}{2}$ $p_a = 0.38793 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.27608 \text{ kip/ft}^2)}{(0.38793 \text{ kip/ft}^2)}$ $Ratio = 0.71168$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.710

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$ $p_s = 1.125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.1217 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.99706$	Status: PASS Ratio: 1.000
	Considering z-direction: $H_o = 0.0082803 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.030732 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.030732 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.030732 \text{ kipft/ft})) + (4 \times (0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))}$ $a = 5.3587 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.030732 \text{ kipft/ft})) + (3 \times (0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^3 \times [(3 \times (0.030732 \text{ kipft/ft})) + (2 \times (0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$ $p = 0.0058919 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.030732 \text{ kipft/ft})) + ((0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$ $s = 0.01318 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.3587 \text{ ft})}{2}$ $p_a = 0.4019 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0058919 \text{ kip/ft}^2)}{(0.4019 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.01466$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$ $p_s = 1.125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.010

$$Ratio = \frac{(0.01318 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.011716$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.871 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.93487 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(77.605 \text{ kipft}) + ((-5.871 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.357 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.357 \text{ kipft/ft})}{(-0.93487 \text{ kip/ft})}$$

$$E = 13.218 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.357 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.93487 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (12.357 \text{ kipft/ft})) + (4 \times (-0.93487 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1715 \text{ ft}$$

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

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

$$V_{max} = ((-0.93487 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.218 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1715 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.218 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1715 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.129 \text{ kip}$$

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

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

$$M_{max} = ((-0.93487 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(13.218 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1715 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.218 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1715 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.218 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1715 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 50.436 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.077 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.012261 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.283 \text{ kipft}) + ((0.077 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.045064 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.045064 \text{ kipft/ft})}{(0.012261 \text{ kip/ft})}$$

$$E = 3.6753 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.045064 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.012261 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.045064 \text{ kipft/ft})) + (4 \times (0.012261 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3602 \text{ ft}$$

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

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

$$V_{max} = ((0.012261 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3602 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3602 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.075215 \text{ kip}$$

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

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

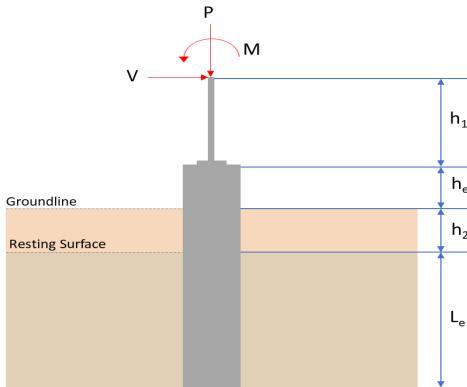
$$M_{max} = ((0.012261 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.6753 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3602 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3602 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3602 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.24926 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(5.233 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.422 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.422 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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 $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(5.233 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0019561$	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.233 \text{ kip} \rightarrow 5233 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(5233 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.18 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (119.18 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.18 \text{ kip}$	

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 $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.18 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.55 \text{ kip}$ Considering x-direction: $V_{max} = 14.129 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(14.129 \text{ kip})}{(110.55 \text{ kip})}$ $Ratio = 0.12781$ Considering z-direction: $V_{max} = 0.075215 \text{ kip}$ - Maximum shear force in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.075215 \text{ kip})}{(110.55 \text{ kip})}$ $Ratio = 0.00068037$	Status: PASS Ratio: 0.130 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: $M_{max} = 50.436 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(50.436 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.20207$	Status: PASS Ratio: 0.200
	Considering z-direction: $M_{max} = 0.24926 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.24926 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.00099862$	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 = 7.5 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>3.722</td><td>5.233</td></tr><tr><td>Vx (kip)</td><td>-3.523</td><td>-5.871</td></tr><tr><td>Vz (kip)</td><td>-0.052</td><td>-0.077</td></tr><tr><td>Mx (kipft)</td><td>-0.193</td><td>-0.283</td></tr><tr><td>Mz (kipft)</td><td>46.231</td><td>77.606</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)	3.722	5.233	Vx (kip)	-3.523	-5.871	Vz (kip)	-0.052	-0.077	Mx (kipft)	-0.193	-0.283	Mz (kipft)	46.231	77.606	
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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Vz (kip)	-0.052	-0.077																										
Mx (kipft)	-0.193	-0.283																										
Mz (kipft)	46.231	77.606																										
	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, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.523\text{ kip})}{1.57 \times (48\text{ in})}$ $H_o = -0.56099\text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(46.231 \text{ kipft}) + ((-3.523 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 7.3616 \text{ kipft/ft}$ Required depth of embedment in earth: $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,x} = 7.0567 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.052 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.0082803 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.193 \text{ kipft}) + ((-0.052 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.030732 \text{ kipft/ft}$ 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: $L_{e,z} = 1.2273 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(7.0567 \text{ ft}), (1.2273 \text{ ft})]$ $L_{e,req} = 7.057 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 7.5 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(7.057 \text{ ft})}{(7.5 \text{ ft})}$ $Ratio = 0.94093$	Status: PASS Ratio: 0.940
	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 $q = \frac{P_o}{A}$ $q = \frac{(3.722 \text{ kip})}{(16 \text{ ft}^2)}$	

$$q = 0.232625 \text{ kips/ft}^2$$

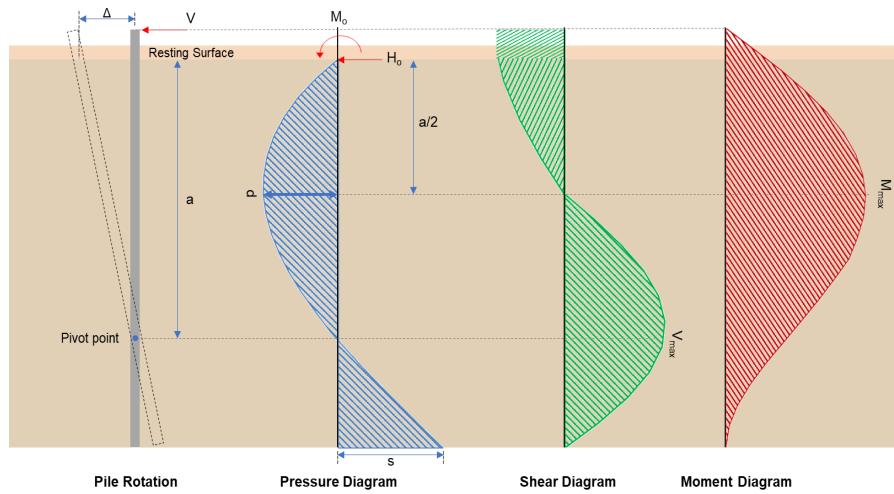
	$q = 0.23262 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.23262 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.11631$	Status: PASS Ratio: 0.120
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.875$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.56099 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 7.3616 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (7.3616 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.3616 \text{ kipft/ft})) + (4 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))}$ $a = 5.1724 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (7.3616 \text{ kipft/ft})) + (3 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^3 \times [(3 \times (7.3616 \text{ kipft/ft})) + (2 \times (-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$ $p = 0.27608 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (7.3616 \text{ kipft/ft})) + ((-0.56099 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$ $s = 1.1217 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.1724 \text{ ft})}{2}$ $p_a = 0.38793 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.27608 \text{ kip/ft}^2)}{(0.38793 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.71168$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.710

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$ $p_s = 1.125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.1217 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.99706$	Status: PASS Ratio: 1.000
	Considering z-direction: $H_o = -0.0082803 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.030732 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.030732 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.030732 \text{ kipft/ft})) + (4 \times (-0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))}$ $a = 5.3587 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.030732 \text{ kipft/ft})) + (3 \times (-0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^3 \times [(3 \times (0.030732 \text{ kipft/ft})) + (2 \times (-0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$ $p = -0.0016732 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.030732 \text{ kipft/ft})) + ((-0.0082803 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$ $s = -0.000067941 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.3587 \text{ ft})}{2}$ $p_a = 0.4019 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0016732 \text{ kip/ft}^2)}{(0.4019 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.0041632$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$ $p_s = 1.125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$Ratio = \frac{(-0.000067941 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = -0.000060392$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.871 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.93487 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(77.606 \text{ kipft}) + ((-5.871 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.358 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.358 \text{ kipft/ft})}{(-0.93487 \text{ kip/ft})}$$

$$E = 13.219 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.358 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.93487 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (12.358 \text{ kipft/ft})) + (4 \times (-0.93487 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1715 \text{ ft}$$

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

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

$$V_{max} = ((-0.93487 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.219 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1715 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.219 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1715 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.129 \text{ kip}$$

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

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

$$M_{max} = ((-0.93487 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(13.219 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1715 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.219 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1715 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.219 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1715 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 50.437 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.077 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.012261 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.283 \text{ kipft}) + ((-0.077 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.045064 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.045064 \text{ kipft/ft})}{(-0.012261 \text{ kip/ft})}$$

$$E = 3.6753 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.045064 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.012261 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.045064 \text{ kipft/ft})) + (4 \times (-0.012261 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3602 \text{ ft}$$

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

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

$$V_{max} = ((-0.012261 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3602 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3602 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.075215 \text{ kip}$$

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

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

$$M_{max} = ((-0.012261 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.6753 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3602 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3602 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.6753 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3602 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.24926 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(5.233 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.422 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.422 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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 $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(5.233 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0019561$	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.233 \text{ kip} \rightarrow 5233 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(5233 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.18 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (119.18 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.18 \text{ kip}$	

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 $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.18 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.55 \text{ kip}$ Considering x-direction: $V_{max} = 14.129 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(14.129 \text{ kip})}{(110.55 \text{ kip})}$ $Ratio = 0.12781$ Considering z-direction: $V_{max} = 0.075215 \text{ kip}$ - Maximum shear force in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.075215 \text{ kip})}{(110.55 \text{ kip})}$ $Ratio = 0.00068037$	Status: PASS Ratio: 0.130 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: $M_{max} = 50.437 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(50.437 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.20207$	Status: PASS Ratio: 0.200
	Considering z-direction: $M_{max} = 0.24926 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.24926 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.00099862$	Status: PASS Ratio: 0.000