

Your Project Calculations



Project Name: Thompson_Kessler_OGS

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Thompson_Kessler_OGS&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=YfZ04tBBIzXkYVhEuMvjkn4mUW1EBOYUMEx2wEg58E4hzfaC7saVOssKfCDxjvk

Array Specification

Product:	Beam
Unique ID:	3P-19.75-8TOP-HD-45-L-4Hx9W-EKLB
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	74.00in
Number of Rows:	4
Number of Columns:	9
Total Number of Modules:	36
Desired Tilt Angle:	45
Front Edge Clearance:	5
Total Array Height at Tilt:	14.75 ft
Total Frame Length:	54.50 ft
Frame Weight:	3023 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	56.25 ft
Rail Length:	166.40 in
Rail Spacing:	3.08 ft
Rail Check:	Not Checked

Support Specifications

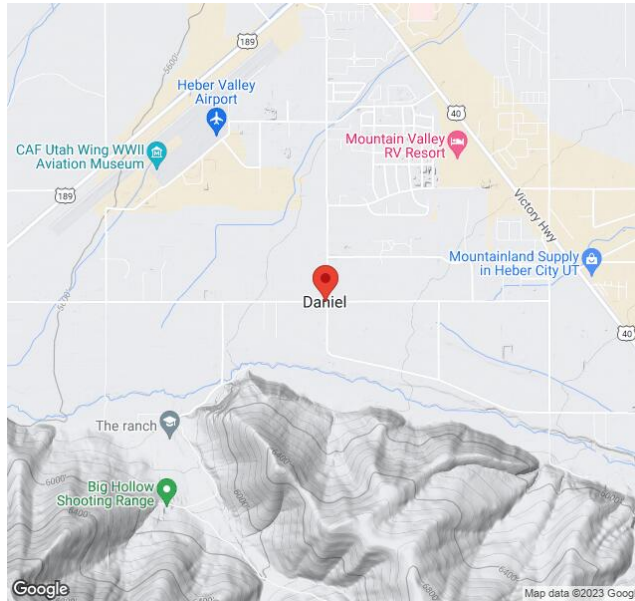
Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	9.90 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 999999.00 ft Pile 2: 999999.00 ft Pile 3: 15.00 ft
Foundation Volume:	3.927 y ³
Foundation Result:	FAILED Try increasing foundation size and/or type and re-running foundation design check on right panel.
Mount Twist:	1.019086 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Daniel, UT 84032, USA
Wind Speed:	115 mph
Snow Load:	85 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.023367 ksf



Design Disclaimer

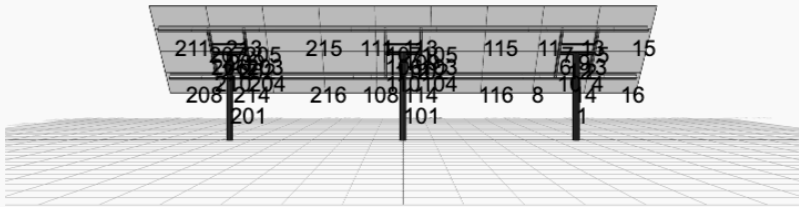
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

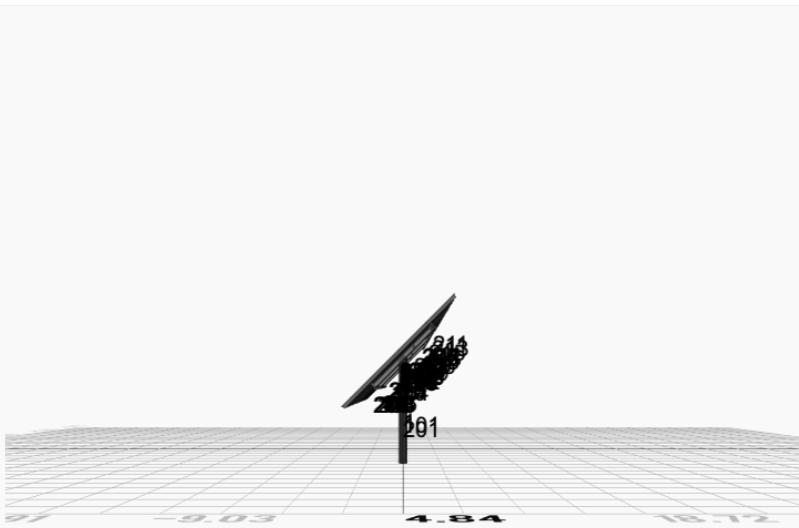
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Design Notes:

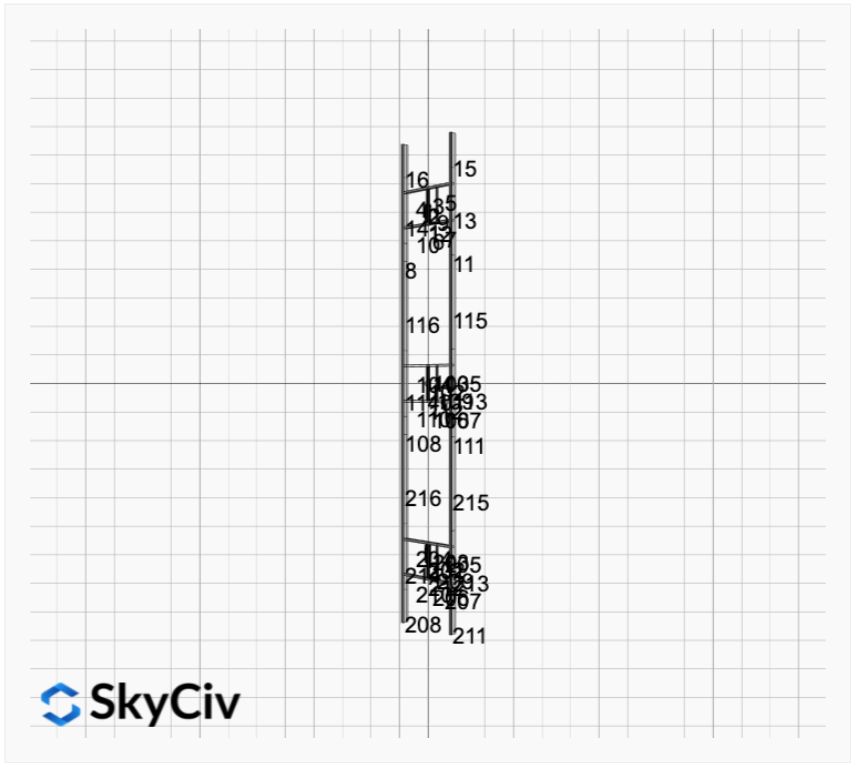
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesigned are all estimates, proper geotechnical reports are required to confirm soil profiles
- Foundation Design and Sizing is approximate only



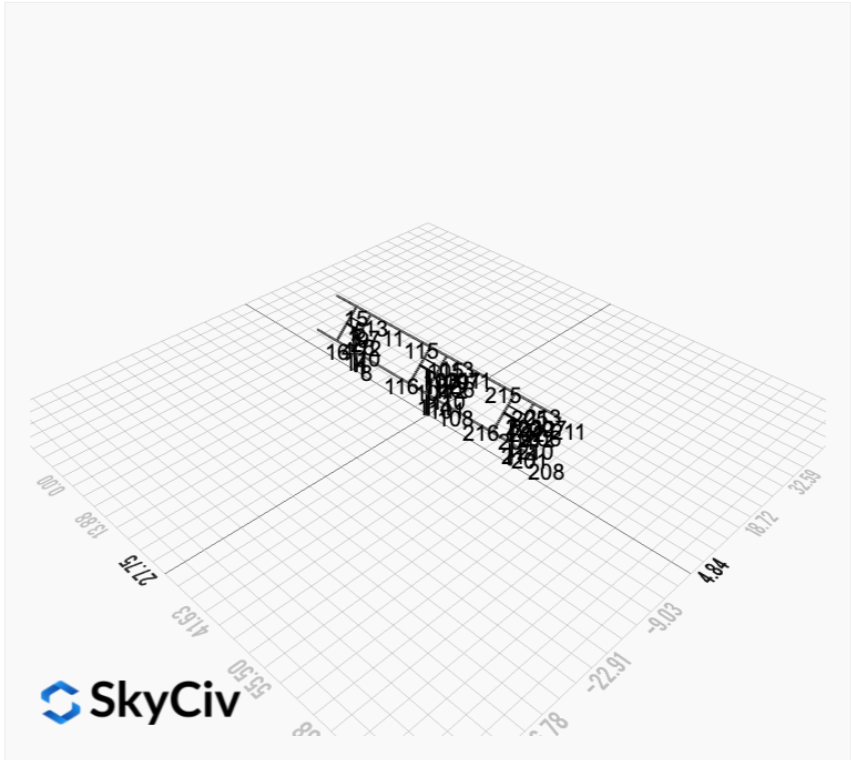
 SkyCiv



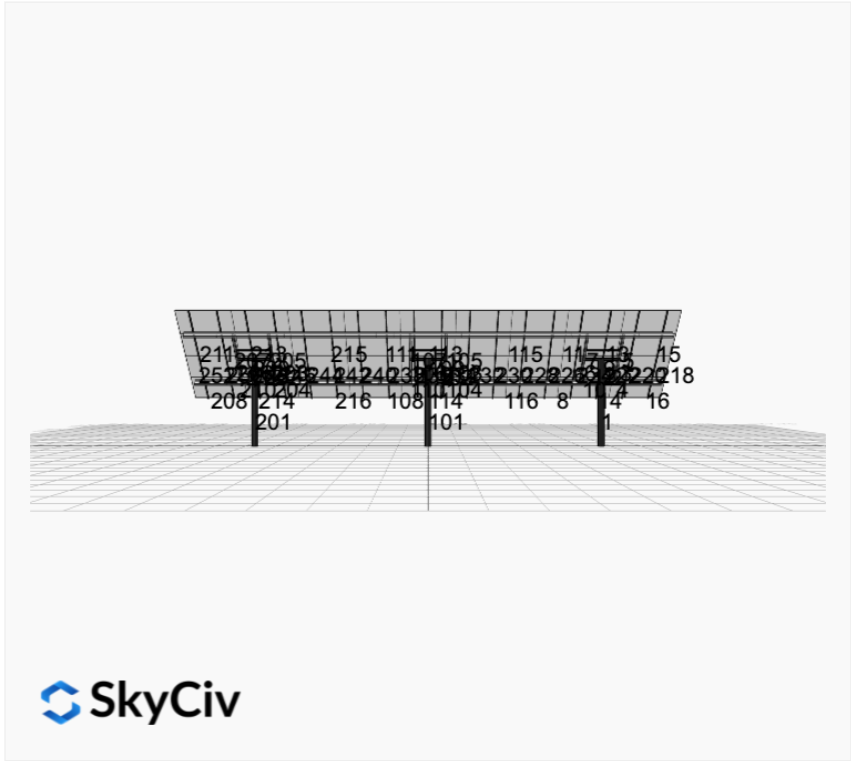
 SkyCiv



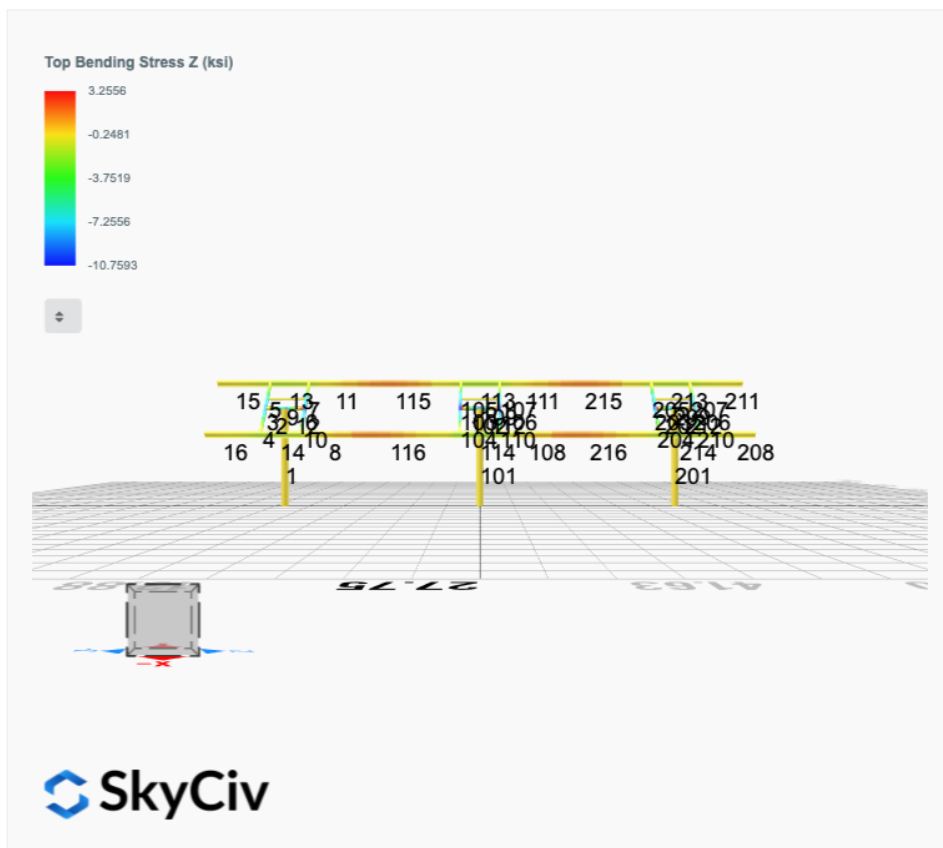
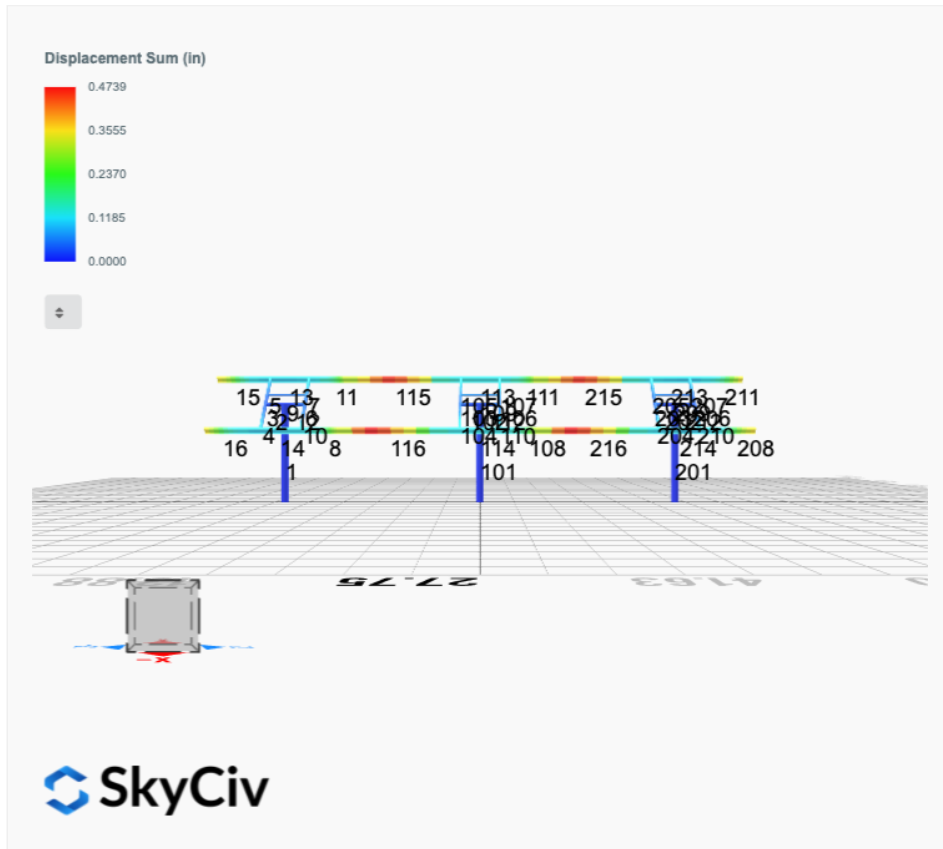
 SkyCiv

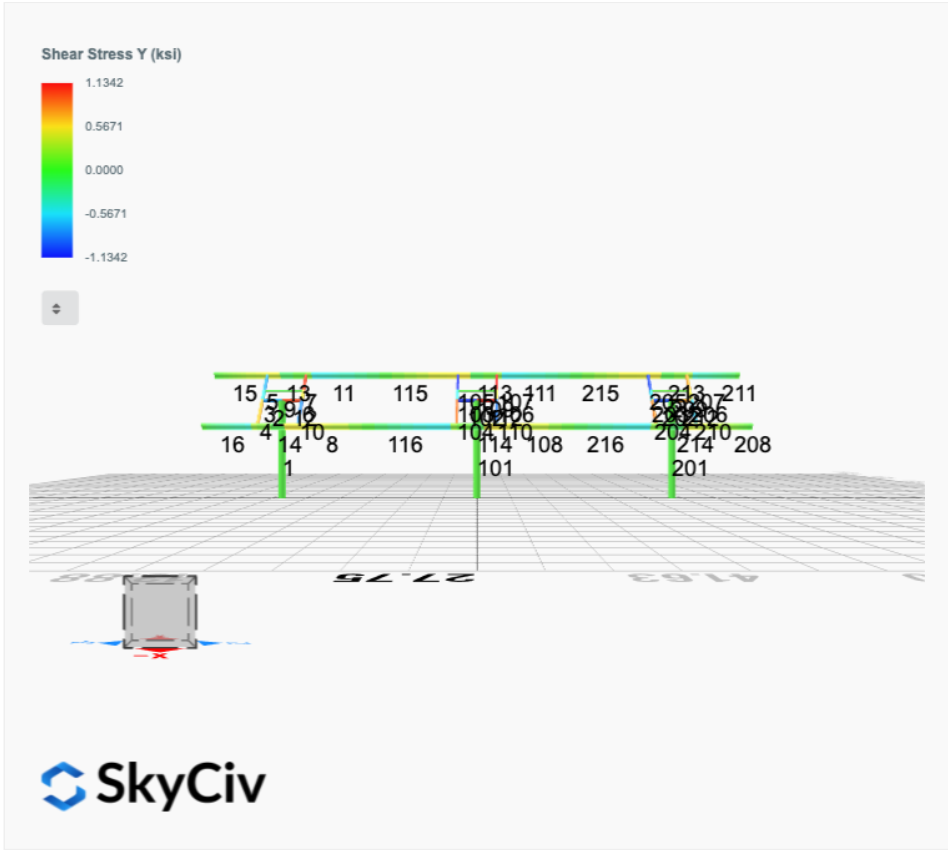
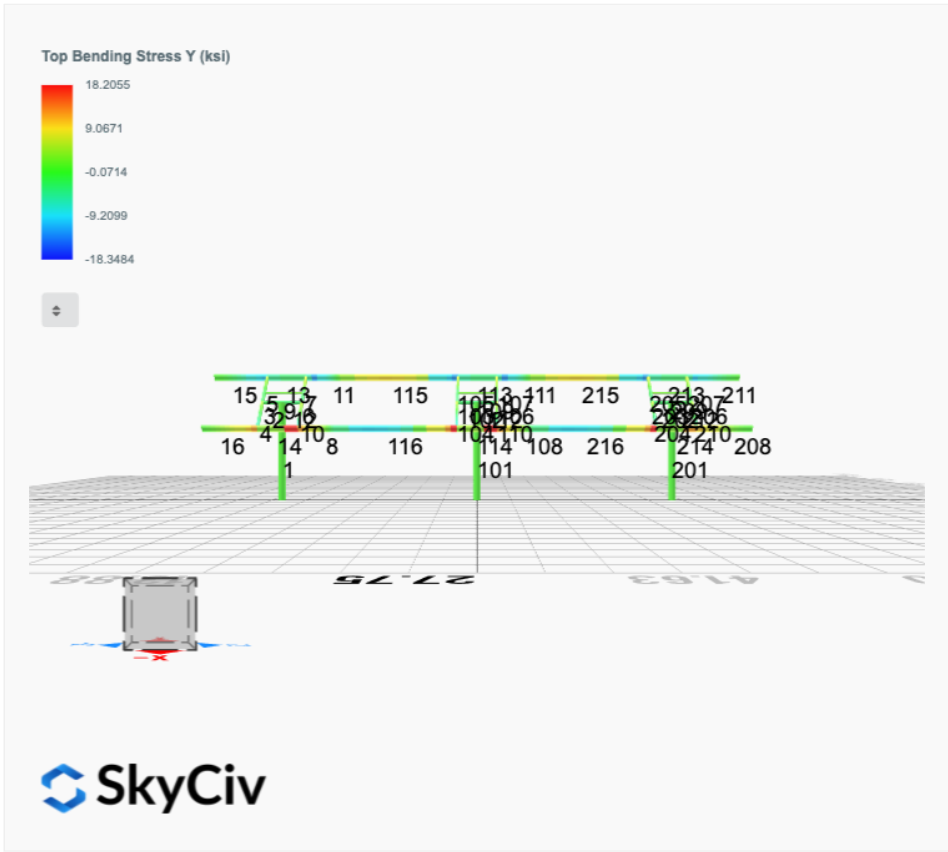


 SkyCiv

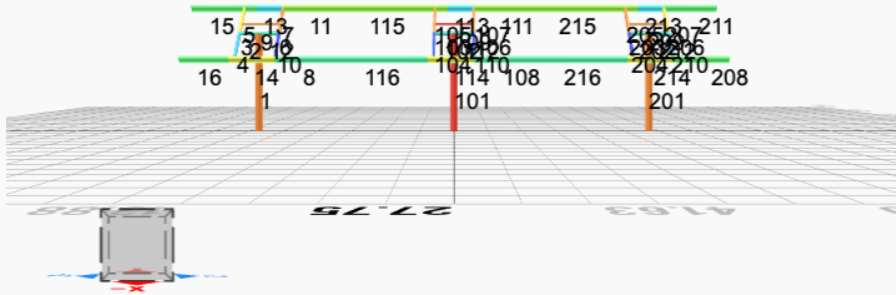
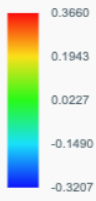


FEM Results (Envelope Worst Case for each member)





Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0080	2.1499	0.0310	0.0797	-0.0071	-0.0519
ULS: 2. D + L	0.0080	2.1499	0.0310	0.0797	-0.0071	-0.0519
ULS: 3. D + (S or Lr or R)	0.0291	6.1076	0.1128	0.2915	-0.0274	-0.2275
ULS: 3. D + (S or Lr or R)	0.0080	2.1499	0.0310	0.0797	-0.0071	-0.0519
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0238	5.1182	0.0924	0.2386	-0.0223	-0.1836
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0080	2.1499	0.0310	0.0797	-0.0071	-0.0519
ULS: 5b. D + 0.7E	0.0080	2.1499	0.0310	0.0797	-0.0071	-0.0519
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0238	5.1182	0.0924	0.2386	-0.0223	-0.1836
ULS: 8. 0.6D + 0.7E	0.0048	1.2899	0.0186	0.0478	-0.0043	-0.0312
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.1667	6.3054	0.1527	0.3465	-0.6049	43.3116
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.1667	6.3054	0.1527	0.3465	-0.6049	43.3116
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.0241	-0.8538	-0.0535	-0.1055	0.4078	-28.8481
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.6806	-0.5073	-0.0527	-0.1033	0.4075	-33.9126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1072	8.2349	0.1836	0.4387	-0.4707	32.3391
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1072	8.2349	0.1836	0.4387	-0.4707	32.3391
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2859	2.8654	0.0290	0.0997	0.2888	-21.7807
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0283	3.1253	0.0296	0.1013	0.2886	-25.5791
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1230	5.2666	0.1222	0.2798	-0.4555	32.4707
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1230	5.2666	0.1222	0.2798	-0.4555	32.4707
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2701	-0.1029	-0.0324	-0.0592	0.3040	-21.6490
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0125	0.1570	-0.0318	-0.0576	0.3038	-25.4474
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.1699	5.4455	0.1403	0.3146	-0.6021	43.3324
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.1699	5.4455	0.1403	0.3146	-0.6021	43.3324
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.0209	-1.7138	-0.0659	-0.1373	0.4106	-28.8273
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.6774	-1.3673	-0.0651	-0.1352	0.4103	-33.8918

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	12.3758
Shear X	-6.9577
Shear Z	0.2821
Moment X	0.6627
Moment Y (Twist)	1.0187
Moment Z	72.7437

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	8.2349
Shear X	-4.1699
Shear Z	0.1836
Moment X	0.4387
Moment Y (Twist)	0.6049
Moment Z	43.3324

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	-0.0160	2.3836	0.0000	0.0000	0.0000	0.1710
ULS: 2. D + L	-0.0160	2.3836	0.0000	0.0000	0.0000	0.1710
ULS: 3. D + (S or Lr or R)	-0.0582	6.9552	0.0000	-0.0000	0.0001	0.5902
ULS: 3. D + (S or Lr or R)	-0.0160	2.3836	0.0000	0.0000	0.0000	0.1710
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0476	5.8123	0.0000	-0.0000	0.0000	0.4854
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0160	2.3836	0.0000	0.0000	0.0000	0.1710
ULS: 5b. D + 0.7E	-0.0160	2.3836	0.0000	0.0000	0.0000	0.1710

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0476	5.8123	0.0000	-0.0000	0.0000	0.4854
ULS: 8. 0.6D + 0.7E	-0.0096	1.4302	0.0000	0.0000	0.0000	0.1026
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.7975	7.2033	0.0000	0.0000	0.0000	49.4766
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.7975	7.2033	0.0000	0.0000	0.0000	49.4766
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.4507	-1.1078	0.0000	0.0000	0.0000	-32.6149
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.0201	-0.6833	0.0000	0.0000	0.0000	-37.9679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.6338	9.4271	0.0000	-0.0000	0.0000	37.4646
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.6338	9.4271	0.0000	-0.0000	0.0000	37.4646
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5524	3.1938	0.0000	-0.0000	0.0000	-24.1040
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2294	3.5121	0.0000	-0.0000	0.0000	-28.1187
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.6021	5.9984	0.0000	0.0000	0.0000	37.1502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.6021	5.9984	0.0000	0.0000	0.0000	37.1502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5840	-0.2350	0.0000	0.0000	0.0000	-24.4184
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2611	0.0834	0.0000	0.0000	0.0000	-28.4332
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.7911	6.2499	0.0000	0.0000	0.0000	49.4082
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.7911	6.2499	0.0000	0.0000	0.0000	49.4082
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.4571	-2.0612	0.0000	0.0000	0.0000	-32.6833
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.0265	-1.6367	0.0000	0.0000	0.0000	-38.0362

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.1903
Shear X	-8.0072
Shear Z	0.0000
Moment X	0.0002
Moment Y (Twist)	0.0005
Moment Z	83.3435

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.4271
Shear X	-4.7975
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0001
Moment Z	49.4766

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0080	2.1499	-0.0310	-0.0797	0.0072	-0.0519
ULS: 2. D + L	0.0080	2.1499	-0.0310	-0.0797	0.0072	-0.0519
ULS: 3. D + (S or Lr or R)	0.0291	6.1076	-0.1128	-0.2917	0.0275	-0.2274
ULS: 3. D + (S or Lr or R)	0.0080	2.1499	-0.0310	-0.0797	0.0072	-0.0519
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0238	5.1182	-0.0924	-0.2387	0.0224	-0.1836
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0080	2.1499	-0.0310	-0.0797	0.0072	-0.0519
ULS: 5b. D + 0.7E	0.0080	2.1499	-0.0310	-0.0797	0.0072	-0.0519
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0238	5.1182	-0.0924	-0.2387	0.0224	-0.1836
ULS: 8. 0.6D + 0.7E	0.0048	1.2899	-0.0186	-0.0478	0.0043	-0.0312
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.1667	6.3054	-0.1527	-0.3465	0.6049	43.3116
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.1667	6.3054	-0.1527	-0.3465	0.6049	43.3116
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.0241	-0.8538	0.0535	0.1055	-0.4077	-28.8481
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.6806	-0.5073	0.0527	0.1033	-0.4075	-33.9126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1072	8.2349	-0.1837	-0.4388	0.4707	32.3391
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1072	8.2349	-0.1837	-0.4388	0.4707	32.3391
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2859	2.8654	-0.0290	-0.0998	-0.2888	-21.7807
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0283	3.1253	-0.0296	-0.1014	-0.2886	-25.5790

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1230	5.2666	-0.1222	-0.2798	0.4555	32.4707
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1230	5.2666	-0.1222	-0.2798	0.4555	32.4707
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2701	-0.1029	0.0324	0.0592	-0.3040	-21.6490
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0125	0.1570	0.0318	0.0576	-0.3038	-25.4474
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.1699	5.4455	-0.1403	-0.3146	0.6021	43.3324
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.1699	5.4455	-0.1403	-0.3146	0.6021	43.3324
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.0209	-1.7138	0.0659	0.1373	-0.4106	-28.8273
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.6774	-1.3673	0.0651	0.1352	-0.4103	-33.8918

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	12.3757
Shear X	-6.9577
Shear Z	-0.2821
Moment X	-0.6632
Moment Y (Twist)	1.0191
Moment Z	72.7447

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	8.2349
Shear X	-4.1699
Shear Z	-0.1837
Moment X	-0.4388
Moment Y (Twist)	0.6049
Moment Z	43.3324

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 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				
10	8in Pipe Sch 80	8.63	0.50				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05

113	133.20	104.94	23.36	6.12	40.24	43.62
114	133.20	104.94	23.18	6.12	40.24	43.62
115	133.20	69.16	17.32	6.12	40.24	43.62
116	133.20	69.16	17.80	6.12	40.24	43.62
201	574.32	331.46	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	52.83	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	52.83	32.87	6.12	40.24	43.62
212	198.33	182.14	21.95	21.95	59.50	59.50
213	133.20	104.94	24.65	6.12	40.24	43.62
214	133.20	104.94	23.78	6.12	40.24	43.62
215	133.20	69.16	17.29	6.12	40.24	43.62
216	133.20	69.16	16.79	6.12	40.24	43.62

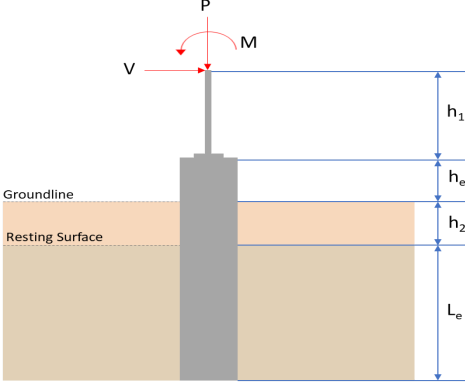
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.037	0.587	0.017	0.040	0.002	0.610	#13	0.434	Not Required	Pass
2	0.004	0.378	0.283	0.090	0.056	0.652	#13	0.114	Not Required	Pass
3	0.011	0.737	0.054	0.073	0.004	0.762	#13	0.045	Not Required	Pass
4	0.011	0.661	0.179	0.066	0.039	0.736	#13	0.080	Not Required	Pass
5	0.011	0.457	0.173	0.073	0.043	0.474	#13	0.074	Not Required	Pass
6	0.014	0.846	0.106	0.085	0.021	0.904	#13	0.045	Not Required	Pass
7	0.015	0.525	0.239	0.084	0.060	0.556	#13	0.074	Not Required	Pass
8	0.002	0.091	0.208	0.050	0.023	0.216	#24	0.095	Not Required	Pass
9	0.016	0.059	0.083	0.002	0.003	0.143	#13	0.204	Not Required	Pass
10	0.015	0.749	0.227	0.075	0.048	0.803	#13	0.080	Not Required	Pass
11	0.003	0.098	0.213	0.058	0.023	0.223	#21	0.095	Not Required	Pass
12	0.003	0.470	0.329	0.109	0.062	0.795	#13	0.053	Not Required	Pass
13	0.008	0.256	0.549	0.074	0.030	0.714	#21	0.286	Not Required	Pass
14	0.009	0.232	0.542	0.065	0.030	0.685	#21	0.190	Not Required	Pass
15	0.000	0.081	0.191	0.035	0.014	0.259	#21	Not Required	Not Required	Pass
16	0.000	0.073	0.191	0.032	0.014	0.256	#21	Not Required	Not Required	Pass
101	0.043	0.672	0.000	0.046	0.000	0.692	#13	0.434	Not Required	Pass
102	0.004	0.489	0.353	0.115	0.067	0.834	#13	0.035	Not Required	Pass
103	0.015	0.902	0.088	0.090	0.013	0.947	#13	0.045	Not Required	Pass
104	0.014	0.828	0.231	0.083	0.049	0.907	#13	0.080	Not Required	Pass
105	0.015	0.560	0.240	0.090	0.060	0.591	#13	0.074	Not Required	Pass
106	0.015	0.902	0.088	0.090	0.013	0.947	#13	0.045	Not Required	Pass
107	0.015	0.560	0.240	0.090	0.060	0.591	#13	0.074	Not Required	Pass
108	0.002	0.074	0.213	0.054	0.024	0.250	#21	0.095	Not Required	Pass
109	0.019	0.059	0.068	0.001	0.000	0.131	#13	0.204	Not Required	Pass
110	0.014	0.828	0.231	0.083	0.049	0.907	#13	0.080	Not Required	Pass
111	0.003	0.097	0.217	0.058	0.024	0.246	#21	0.095	Not Required	Pass
112	0.004	0.489	0.353	0.115	0.067	0.834	#13	0.035	Not Required	Pass
113	0.008	0.255	0.554	0.074	0.030	0.757	#21	0.286	Not Required	Pass

114	0.010	0.268	0.549	0.069	0.030	0.759	#21	0.286	Not Required	Pass
115	0.006	0.356	0.303	0.058	0.024	0.603	#21	0.473	Not Required	Pass
116	0.002	0.311	0.304	0.054	0.024	0.575	#21	0.473	Not Required	Pass
201	0.037	0.587	0.017	0.040	0.002	0.610	#13	0.434	Not Required	Pass
202	0.003	0.470	0.329	0.109	0.062	0.795	#13	0.053	Not Required	Pass
203	0.014	0.846	0.106	0.085	0.021	0.904	#13	0.045	Not Required	Pass
204	0.015	0.749	0.227	0.075	0.048	0.803	#13	0.080	Not Required	Pass
205	0.015	0.525	0.239	0.084	0.060	0.556	#13	0.074	Not Required	Pass
206	0.011	0.737	0.054	0.073	0.004	0.762	#13	0.045	Not Required	Pass
207	0.011	0.457	0.173	0.073	0.043	0.474	#13	0.074	Not Required	Pass
208	0.000	0.073	0.191	0.032	0.014	0.256	#21	Not Required	Not Required	Pass
209	0.016	0.059	0.083	0.002	0.003	0.143	#13	0.204	Not Required	Pass
210	0.011	0.661	0.179	0.066	0.039	0.736	#13	0.080	Not Required	Pass
211	0.000	0.081	0.191	0.035	0.014	0.259	#21	Not Required	Not Required	Pass
212	0.004	0.378	0.283	0.090	0.056	0.652	#13	0.114	Not Required	Pass
213	0.008	0.256	0.549	0.074	0.030	0.714	#21	0.190	Not Required	Pass
214	0.009	0.232	0.542	0.065	0.030	0.685	#21	0.286	Not Required	Pass
215	0.006	0.364	0.304	0.058	0.023	0.605	#21	0.473	Not Required	Pass
216	0.002	0.319	0.303	0.050	0.023	0.580	#21	0.473	Not Required	Pass

Definitions

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 15$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.235</td> <td>12.376</td> </tr> <tr> <td>V_x (kip)</td> <td>-4.170</td> <td>-6.958</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.184</td> <td>-0.282</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.439</td> <td>-0.663</td> </tr> <tr> <td>M_z (kipft)</td> <td>43.332</td> <td>72.745</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	8.235	12.376	V_x (kip)	-4.170	-6.958	V_z (kip)	-0.184	-0.282	M_x (kipft)	-0.439	-0.663	M_z (kipft)	43.332	72.745	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-4.17 \text{ kip})}{(36 \text{ in})}$ $H_o = -1.39 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(43.332 \text{ kipft}) + ((-4.17 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 14.444 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.7464 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.184 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.061333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.439 \text{ kipft}) + ((-0.184 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.14633 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9325 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.7464 \text{ ft}), (1.9325 \text{ ft})]$$

$$L_{e,req} = 8.746 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (15 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 15 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.746 \text{ ft})}{(15 \text{ ft})}$$

$$\text{Ratio} = 0.58307$$

Status: **PASS**
Ratio: **0.580**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.235 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.165 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.165 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.58251$$

Status: **PASS**
Ratio: **0.580**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(15 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.39 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 14.444 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.444 \text{ kipft/ft}) \times (15 \text{ ft})) + (3 \times (-1.39 \text{ kip/ft}) \times (15 \text{ ft})^2)}{(6 \times (14.444 \text{ kipft/ft})) + (4 \times (-1.39 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10.613 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (14.444 \text{ kipft/ft})) + (3 \times (-1.39 \text{ kip/ft}) \times (15 \text{ ft}))]^2}{(15 \text{ ft})^2 \times [(3 \times (14.444 \text{ kipft/ft})) + (2 \times (-1.39 \text{ kip/ft}) \times (15 \text{ ft}))]}$$

$$p = 0.073115 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (14.444 \text{ kipft/ft})) + ((-1.39 \text{ kip/ft}) \times (15 \text{ ft}))]}{(15 \text{ ft})^2}$$

$$s = 0.3367 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(10.613 \text{ ft})}{2}$$

$$p_a = 0.79598 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.073115 \text{ kip/ft}^2)}{(0.79598 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.091856$$

Status: **PASS**
Ratio: **0.090**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (15 \text{ ft})$$

$$p_s = 2.25 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.3367 \text{ kip/ft}^2)}{(2.25 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.14965$$

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

Considering z-direction:

$H_o = -0.061333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.14633 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14633 \text{ kipft/ft}) \times (15 \text{ ft})) + (3 \times (-0.061333 \text{ kip/ft}) \times (15 \text{ ft})^2)}{(6 \times (0.14633 \text{ kipft/ft})) + (4 \times (-0.061333 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 11.009 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.14633 \text{ kipft/ft})) + (3 \times (-0.061333 \text{ kip/ft}) \times (15 \text{ ft}))]^2}{(15 \text{ ft})^2 \times [(3 \times (0.14633 \text{ kipft/ft})) + (2 \times (-0.061333 \text{ kip/ft}) \times (15 \text{ ft}))]}$$

$$p = -0.017673 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.14633 \text{ kipft/ft})) + ((-0.061333 \text{ kip/ft}) \times (15 \text{ ft}))]}{(15 \text{ ft})^2}$$

$$s = -0.026278 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(11.009 \text{ ft})}{2}$$

$$p_a = 0.82569 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.017673 \text{ kip/ft}^2)}{(0.82569 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.021404$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (15 \text{ ft})$$

$$p_s = 2.25 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

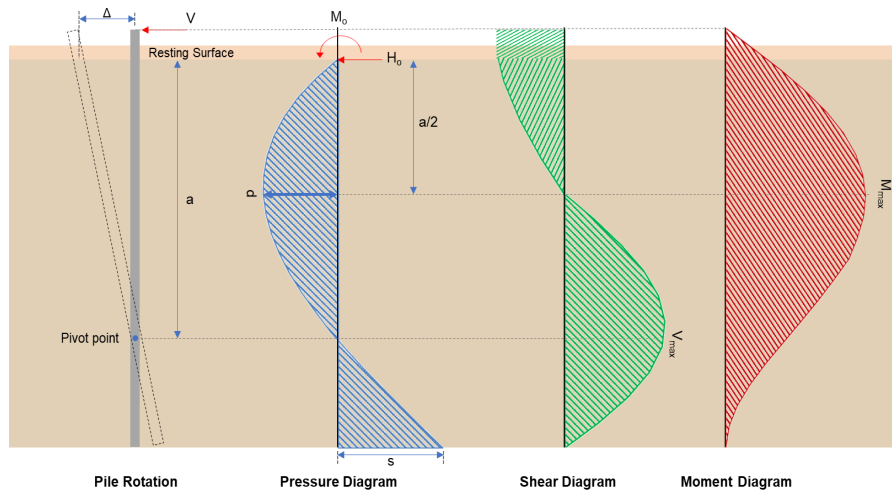
Status: **PASS**
Ratio: **-0.020**

$$ratio = \frac{-}{p_s}$$

$$Ratio = \frac{(-0.026278 \text{ kip/ft}^2)}{(2.25 \text{ kip/ft}^2)}$$

$$Ratio = -0.011679$$

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



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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-6.958 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -2.3193 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(72.745 \text{ kipft}) + ((-6.958 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 24.248 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(24.248 \text{ kipft/ft})}{(-2.3193 \text{ kip/ft})}$$

$$E = 10.455 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (24.248 \text{ kipft/ft}) \times (15 \text{ ft})) + (3 \times (-2.3193 \text{ kip/ft}) \times (15 \text{ ft})^2)}{(6 \times (24.248 \text{ kipft/ft})) + (4 \times (-2.3193 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10.611 \text{ ft}$$

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

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

$$V_{max} = ((-2.3193 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.455 \text{ ft})}{(15 \text{ ft})} + 3 \right) \times \left(\frac{(10.611 \text{ ft})}{(15 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (10.455 \text{ ft})}{(15 \text{ ft})} + 2 \right) \times \left(\frac{(10.611 \text{ ft})}{(15 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.195 \text{ kip}$$

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

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

$$M_{max} = ((-2.3193 \text{ kip/ft}) \times (36 \text{ in}) \times (15 \text{ ft})) \times \left[\left(\frac{(10.455 \text{ ft})}{(15 \text{ ft})} + \frac{(10.611 \text{ ft})}{2 \times (15 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.455 \text{ ft})}{(15 \text{ ft})} + 3 \right) \times \left(\frac{(10.611 \text{ ft})}{2 \times (15 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (10.455 \text{ ft})}{(15 \text{ ft})} + 2 \right) \times \left(\frac{(10.611 \text{ ft})}{2 \times (15 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 89.613 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.282 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.094 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.663 \text{ kipft}) + ((-0.282 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.221 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.221 \text{ kipft/ft})}{(-0.094 \text{ kip/ft})}$$

$$E = 2.3511 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.221 \text{ kipft/ft}) \times (15 \text{ ft})) + (3 \times (-0.094 \text{ kip/ft}) \times (15 \text{ ft})^2)}{(6 \times (0.221 \text{ kipft/ft})) + (4 \times (-0.094 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 11.012 \text{ ft}$$

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

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

$$V_{max} = ((-0.094 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.3511 \text{ ft})}{(15 \text{ ft})} + 3 \right) \times \left(\frac{(11.012 \text{ ft})}{(15 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (2.3511 \text{ ft})}{(15 \text{ ft})} + 2 \right) \times \left(\frac{(11.012 \text{ ft})}{(15 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.26925 \text{ kip}$$

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

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

$$M_{max} = ((-0.094 \text{ kip/ft}) \times (36 \text{ in}) \times (15 \text{ ft})) \times \left[\left(\frac{(2.3511 \text{ ft})}{(15 \text{ ft})} + \frac{(11.012 \text{ ft})}{2 \times (15 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.3511 \text{ ft})}{(15 \text{ ft})} + 3 \right) \times \left(\frac{(11.012 \text{ ft})}{2 \times (15 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (2.3511 \text{ ft})}{(15 \text{ ft})} + 2 \right) \times \left(\frac{(11.012 \text{ ft})}{2 \times (15 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 1.6466 \text{ kipft}$$

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,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.376 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.986 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.986 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$$

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

$$\text{Ratio} = \frac{A_{min}}{A_{st}}$$

$$\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

$$s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$$

$$s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$$

$$s_{rebar} = 1.5 \text{ in}$$

Ties:

25.7.2.2

Since longitudinal reinforcement is \leq No. 10 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

$$s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), D]$$

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

$$s_{ties} = 10 \text{ in}$$

Summary:

Status: **PASS**
Ratio: **1.000**

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.376 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.00987$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.

22.5.5.1.1

$V_{c,max}$ - Max shear strength of concrete

$$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$$

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 186.09 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.376 \text{ kip} \rightarrow 12376 \text{ lbf}$.

22.5.5.1.1(a)

$V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(12376 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.539 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.

22.5.5.1.2

$V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (76.539 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 76.539 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((76.539 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.561 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 13.195 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(13.195 \text{ kip})}{(74.561 \text{ kip})}$ $Ratio = 0.17697$ <p>Considering z-direction:</p> <p>$V_{max} = 0.26925 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.26925 \text{ kip})}{(74.561 \text{ kip})}$ $Ratio = 0.0036111$	<p>Status: PASS Ratio: 0.180</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 89.613 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(89.613 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 1.4447$	<p>Status: FAIL Ratio: 1.440</p>
	<p>Considering z-direction: $M_{max} = 1.6466 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.6466 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.026547$	<p>Status: PASS Ratio: 0.030</p>