Your Project Calculations

Project Name: BLR_Whitehouse_QCells_475

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=BLR_Whitehouse_QCells_475&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=BxI0diZOlqKA7XI8ngmbLL3DeVJp1ntS8Lr0y6jNWal4ThssprJ2sN3LemidWO55

Array Specification

Product:	Beam
Unique ID:	3P-17-8TOP-XD-12-L-5Hx6W-58LI
Duty Classification:	XD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Desired Tilt Angle:	40
Front Edge Clearance:	3
Total Array Height at Tilt:	14.07 ft
Total Frame Length:	43.50 ft
Frame Weight:	2595 lbs
Array Dimensions N/S:	17.33 ft
Array Dimensions E/W:	44.10 ft
Rail Length:	208.00 in
Rail Spacing:	3.63 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.57 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.75 ft Pile 3: 6.25 ft
Foundation Volume:	11.407 y^3
Foundation Result:	PASSED
Mount Twist:	1.643424 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	16 Beulah Land Rd, Blandford, MA 01008, USA
Wind Speed:	105 mph
Snow Load:	88 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.029030 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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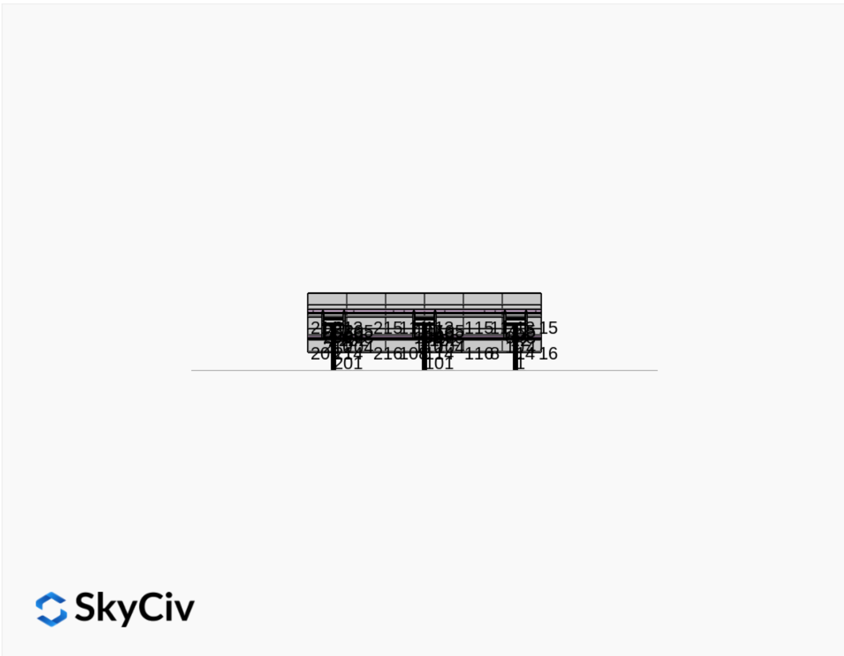
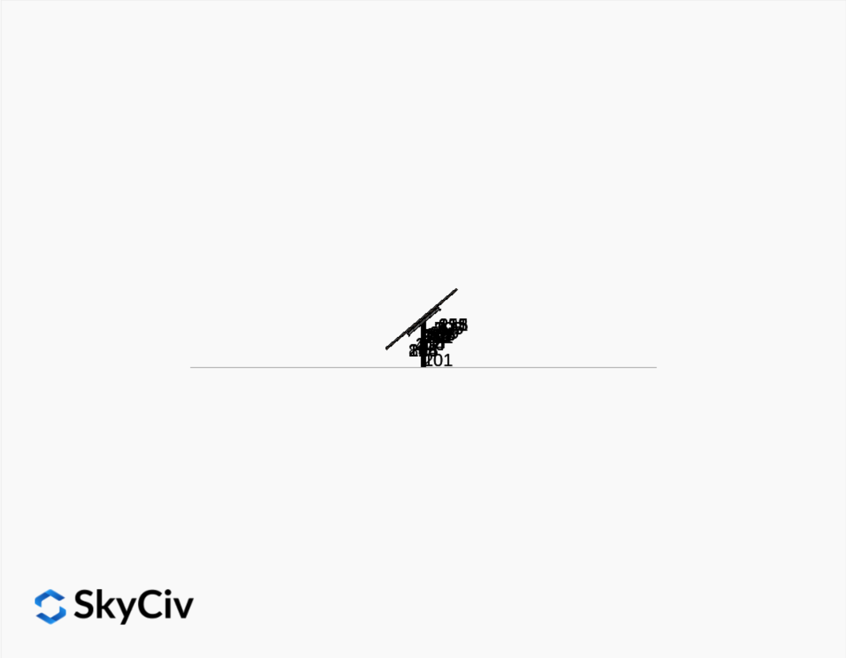
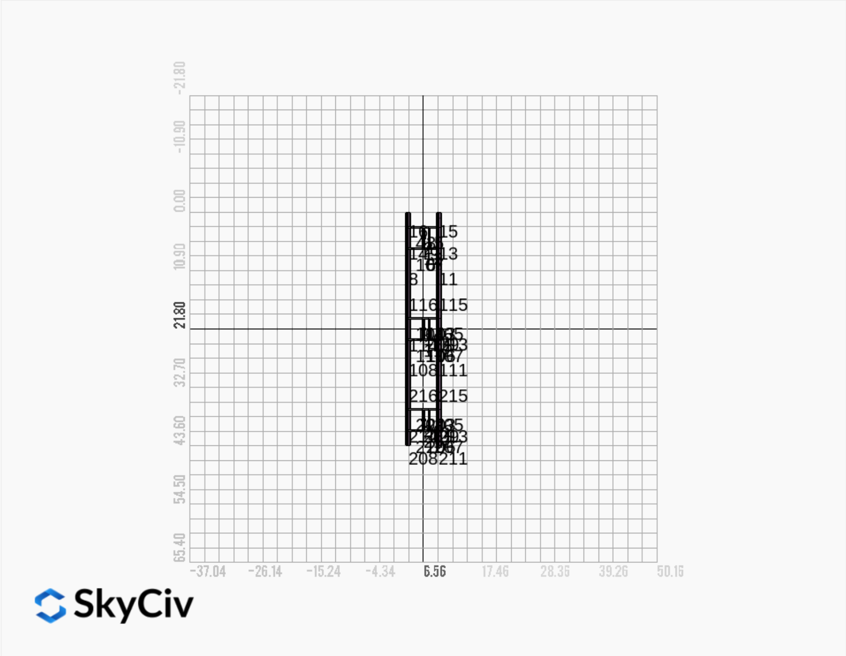
Design Notes:

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



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0.00
10.90
21.80
32.70
43.60
54.50
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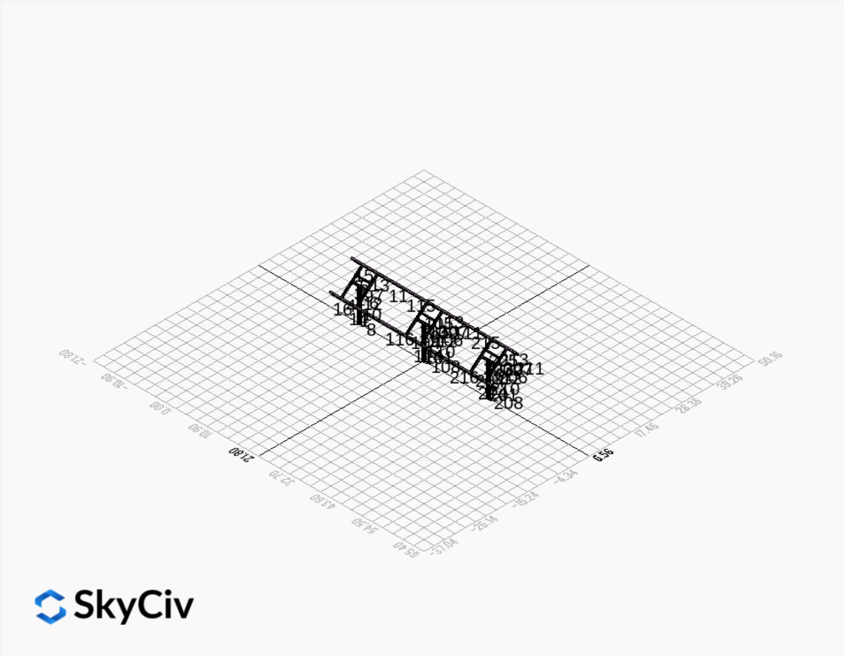
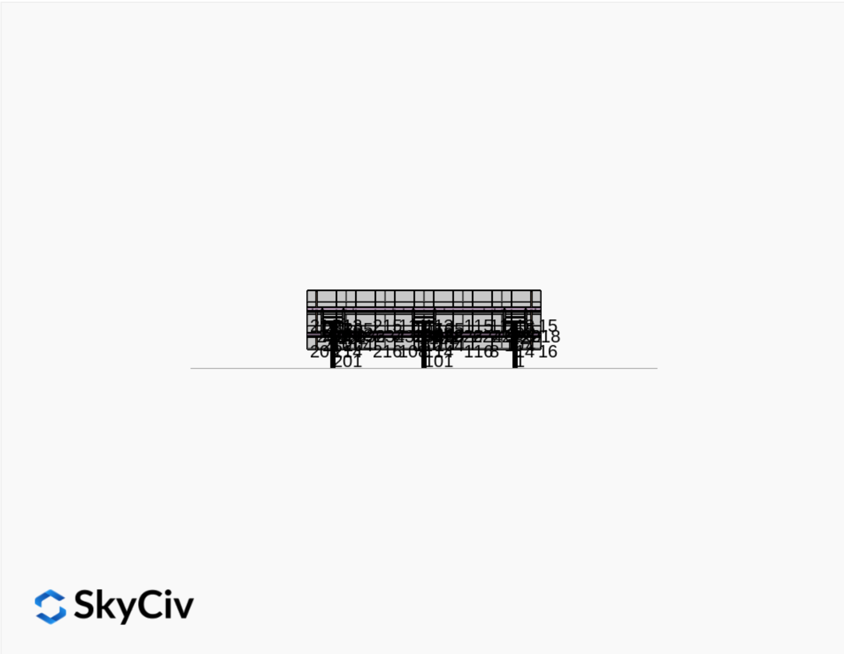
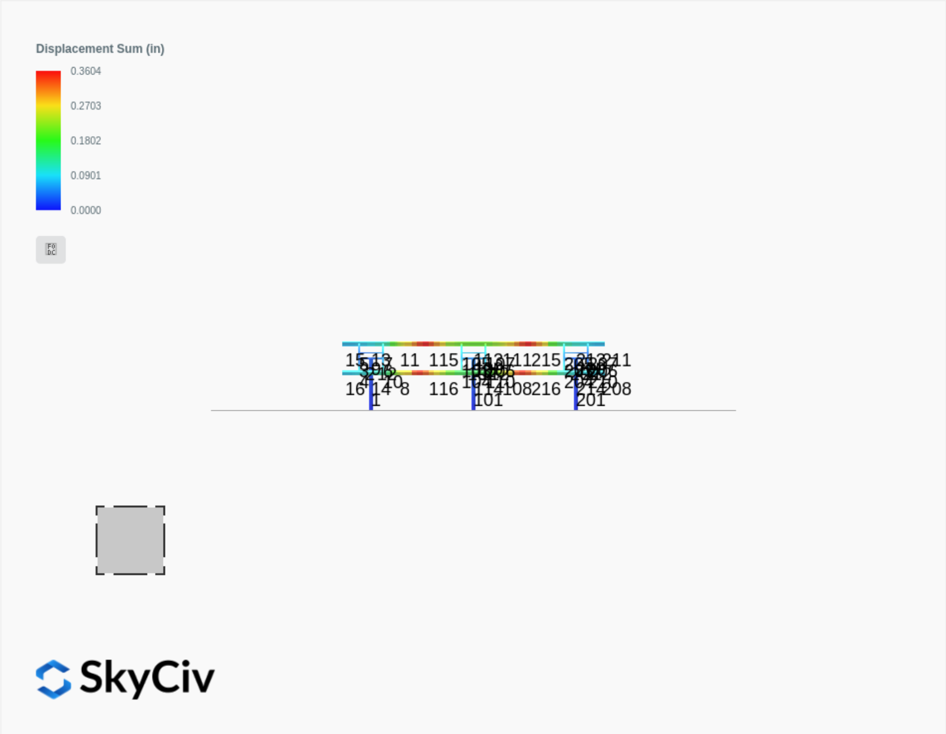
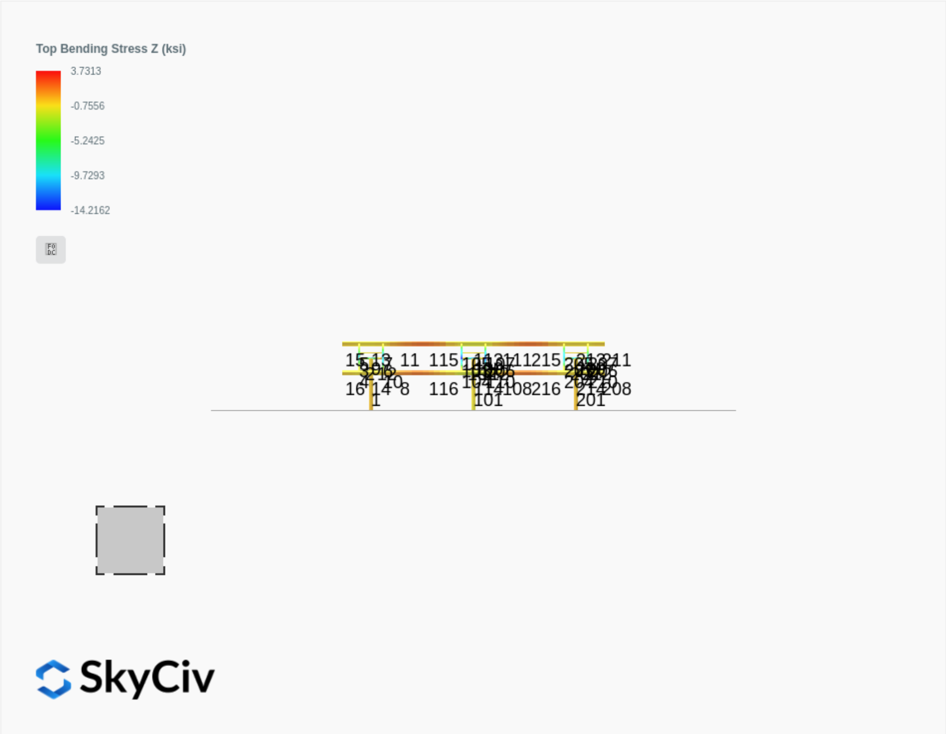
-4.34
6.56
17.46
28.36
39.26
50.16

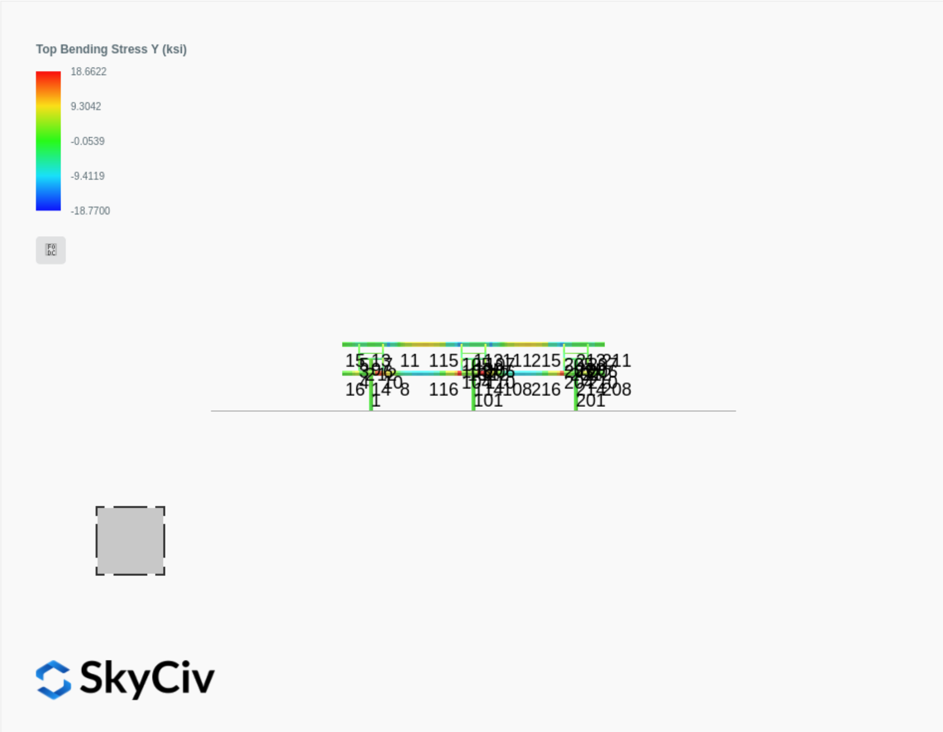




FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
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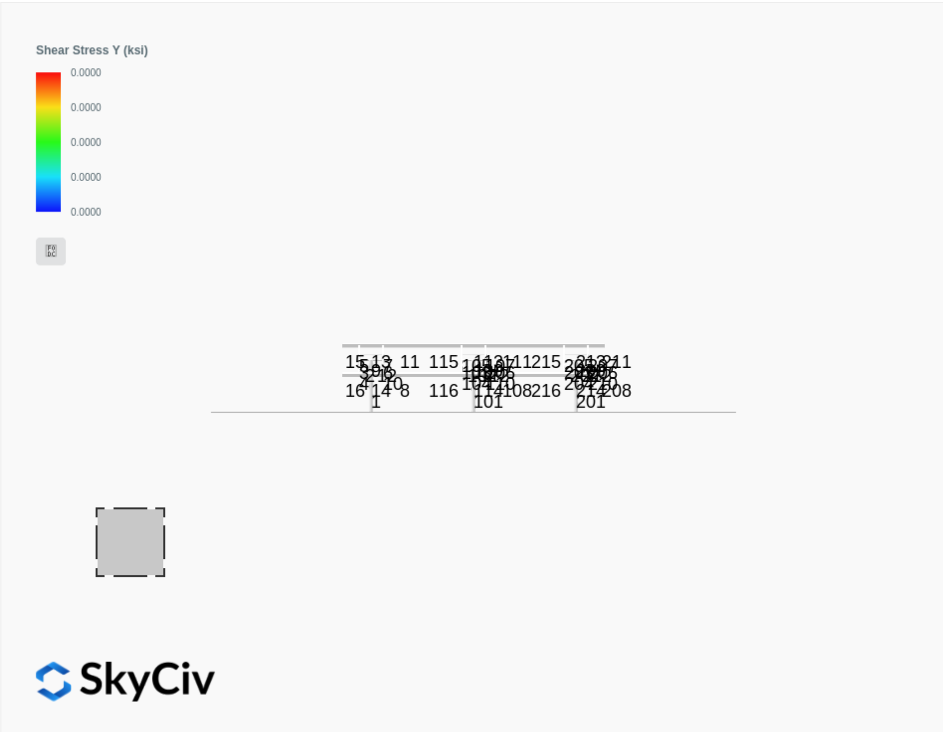
-9.4119
-18.7700


Shear Stress Y (ksi)
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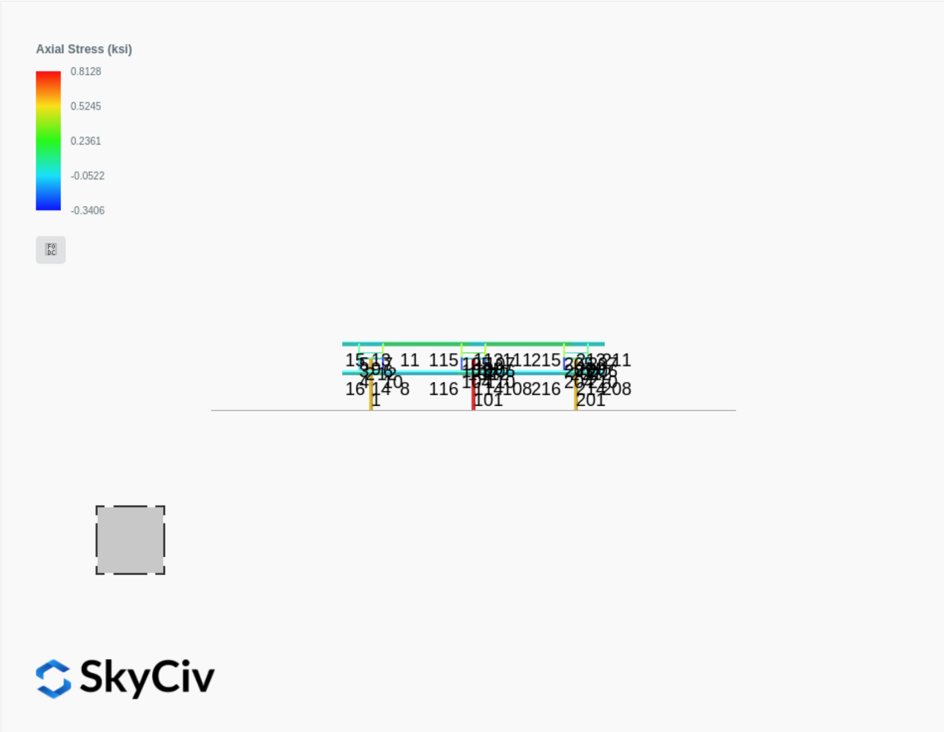
0.0000
0.0000
0.0000


Axial Stress (ksi)
0.8128
0.5245
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-0.0522
-0.3406
SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0266	1.8394	0.0722	0.1600	-0.0399	-0.1721
ULS: 2. D + L	0.0266	1.8394	0.0722	0.1600	-0.0399	-0.1721
ULS: 3. D + (S or Lr or R)	0.1265	6.8098	0.3452	0.7668	-0.1925	-0.8922
ULS: 3. D + (S or Lr or R)	0.0266	1.8394	0.0722	0.1600	-0.0399	-0.1721
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.1015	5.5672	0.2770	0.6151	-0.1544	-0.7122
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0266	1.8394	0.0722	0.1600	-0.0399	-0.1721
ULS: 5b. D + 0.7E	0.0266	1.8394	0.0722	0.1600	-0.0399	-0.1721
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.1015	5.5672	0.2770	0.6151	-0.1544	-0.7122
ULS: 8. 0.6D + 0.7E	0.0159	1.1037	0.0433	0.0960	-0.0239	-0.1033
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1710	5.5883	0.3648	0.7479	-0.9470	28.6814
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.1710	5.5883	0.3648	0.7479	-0.9470	28.6814
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5679	-1.1428	-0.1546	-0.2956	0.6638	-21.8025
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.2183	-0.7169	-0.1441	-0.2740	0.6389	-26.5068
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2967	8.3789	0.4964	1.0561	-0.8347	20.9280
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.2967	8.3789	0.4964	1.0561	-0.8347	20.9280
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0075	3.3306	0.1069	0.2734	0.3734	-16.9349
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7453	3.6500	0.1147	0.2897	0.3547	-20.4632
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3716	4.6511	0.2916	0.6009	-0.7202	21.4680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.3716	4.6511	0.2916	0.6009	-0.7202	21.4680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9325	-0.3972	-0.0979	-0.1817	0.4879	-16.3949
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.6704	-0.0778	-0.0901	-0.1655	0.4692	-19.9232
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1817	4.8526	0.3359	0.6839	-0.9310	28.7503
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.1817	4.8526	0.3359	0.6839	-0.9310	28.7503
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5572	-1.8785	-0.1835	-0.3596	0.6798	-21.7336
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.2077	-1.4527	-0.1730	-0.3380	0.6548	-26.4380

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.2867
Shear X	-5.3293
Shear Z	0.7721
Moment X	1.6671
Moment Y (Twist)	1.6431
Moment Z	48.1062

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	8.3789
Shear X	-3.1817
Shear Z	0.4964
Moment X	1.0561
Moment Y (Twist)	0.9470
Moment Z	28.7503

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	-0.0531	2.3336	0.0000	0.0000	0.0000	0.4233
ULS: 2. D + L	-0.0531	2.3336	0.0000	0.0000	0.0000	0.4233
ULS: 3. D + (S or Lr or R)	-0.2529	9.1607	0.0000	-0.0000	0.0001	1.9677
ULS: 3. D + (S or Lr or R)	-0.0531	2.3336	0.0000	0.0000	0.0000	0.4233
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.2030	7.4539	0.0000	-0.0000	0.0001	1.5816
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0531	2.3336	0.0000	0.0000	0.0000	0.4233
ULS: 5b. D + 0.7E	-0.0531	2.3336	0.0000	0.0000	0.0000	0.4233

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.2030	7.4539	0.0000	-0.0000	0.0001	1.5816
ULS: 8. 0.6D + 0.7E	-0.0319	1.4001	0.0000	0.0000	0.0000	0.2540
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.3338	7.5588	0.0000	0.0000	0.0000	37.7241
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.3338	7.5588	0.0000	0.0000	0.0000	37.7241
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3729	-1.8422	0.0000	0.0000	0.0000	-27.6240
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.7877	-1.1634	0.0000	0.0000	0.0000	-32.3832
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4135	11.3729	0.0000	-0.0000	0.0001	29.5572
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.4135	11.3729	0.0000	-0.0000	0.0001	29.5572
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3665	4.3221	0.0000	-0.0000	0.0001	-19.4539
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9276	4.8312	0.0000	-0.0000	0.0001	-23.0233
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2636	6.2525	0.0000	0.0000	0.0000	28.3989
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.2636	6.2525	0.0000	0.0000	0.0000	28.3989
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5164	-0.7983	0.0000	0.0000	0.0000	-20.6122
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0775	-0.2892	0.0000	0.0000	0.0000	-24.1816
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.3126	6.6254	0.0000	0.0000	0.0000	37.5548
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.3126	6.6254	0.0000	0.0000	0.0000	37.5548
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3942	-2.7756	0.0000	0.0000	0.0000	-27.7933
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.8089	-2.0968	0.0000	0.0000	0.0000	-32.5525

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	18.0727
Shear X	-7.2915
Shear Z	0.0000
Moment X	0.0002
Moment Y (Twist)	0.0005
Moment Z	64.1390

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	11.3729
Shear X	-4.3338
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0001
Moment Z	37.7241

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	0.0266	1.8394	-0.0722	-0.1600	0.0399	-0.1721
ULS: 2. D + L	0.0266	1.8394	-0.0722	-0.1600	0.0399	-0.1721
ULS: 3. D + (S or Lr or R)	0.1265	6.8098	-0.3452	-0.7670	0.1927	-0.8921
ULS: 3. D + (S or Lr or R)	0.0266	1.8394	-0.0722	-0.1600	0.0399	-0.1721
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.1015	5.5672	-0.2770	-0.6153	0.1545	-0.7121
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0266	1.8394	-0.0722	-0.1600	0.0399	-0.1721
ULS: 5b. D + 0.7E	0.0266	1.8394	-0.0722	-0.1600	0.0399	-0.1721
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.1015	5.5672	-0.2770	-0.6153	0.1545	-0.7121
ULS: 8. 0.6D + 0.7E	0.0159	1.1037	-0.0433	-0.0960	0.0239	-0.1033
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1710	5.5883	-0.3648	-0.7479	0.9470	28.6814
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.1710	5.5883	-0.3648	-0.7479	0.9470	28.6814
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5679	-1.1428	0.1546	0.2956	-0.6638	-21.8025
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.2183	-0.7169	0.1441	0.2740	-0.6389	-26.5068
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2967	8.3789	-0.4964	-1.0562	0.8348	20.9281
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.2967	8.3789	-0.4964	-1.0562	0.8348	20.9281
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0075	3.3306	-0.1069	-0.2735	-0.3733	-16.9348
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7453	3.6500	-0.1147	-0.2898	-0.3546	-20.4631

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	-2.3716	4.6511	-0.2916	-0.6009	0.7202	21.4680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.3716	4.6511	-0.2916	-0.6009	0.7202	21.4680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9325	-0.3972	0.0979	0.1817	-0.4879	-16.3949
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.6704	-0.0778	0.0901	0.1655	-0.4692	-19.9231
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1817	4.8526	-0.3359	-0.6839	0.9310	28.7503
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.1817	4.8526	-0.3359	-0.6839	0.9310	28.7503
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5572	-1.8785	0.1835	0.3596	-0.6798	-21.7336
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.2077	-1.4527	0.1730	0.3380	-0.6548	-26.4380

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.2867
Shear X	-5.3293
Shear Z	-0.7721
Moment X	-1.6676
Moment Y (Twist)	1.6434
Moment Z	48.1071

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	8.3789
Shear X	-3.1817
Shear Z	-0.4964
Moment X	-1.0562
Moment Y (Twist)	0.9470
Moment Z	28.7503

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
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Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
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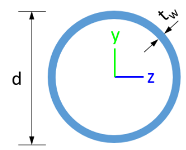
Section Dimensions



ID	Name	d (in)	t_w (in)
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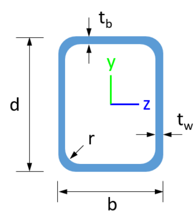
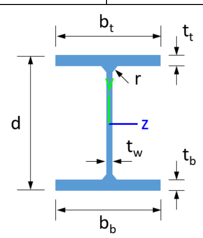
6	4in Pipe Sch 120	4.50	0.44
9	8in Pipe Sch 40	8.63	0.32



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	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)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

17	HSS5x3x1/4	3.37	11.00	4.81	10.70	62.42	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	9	18.00	18.00	8.57	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.17,1.16,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.18,1.18,1.18,1.16,1.16,1.17,1.17,1.17,1.17	300	200	1
4	17	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.69,1.70,1.67,1.67,1.65,1.70,1.67,1.67,1.66,1.60,1.67,1.67,1.68,1.67,1.68,1.68,1.65,1.81,1.67,1.67,1.66,1.64	300	200	1
5	17	1.52	1.52	2.33	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
8	20	1.33	1.33	2.05	1.20,1.20,1.20,1.20,1.20,1.20,1.19,1.19,1.19,1.60,1.19,1.19,1.19,1.07,1.19,1.19,1.20,1.21,1.19,1.19,1.19,1.50,1.19,1.19,1.19,1.09	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.64,1.67,1.67,1.68,1.67,1.67,1.67,1.65,2.46,1.67,1.67,1.66,1.65	300	200	1
11	20	1.33	1.33	2.05	1.21,1.21,1.21,1.21,1.21,1.21,1.23,1.23,1.24,1.26,1.23,1.23,1.23,1.25,1.22,1.22,1.20,1.16,1.23,1.23,1.24,1.26,1.23,1.23,1.23,1.25	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.43,1.46,1.43,1.47,1.45,1.43,1.44,1.44,1.42,1.52,1.43,1.43,1.44,1.50,1.46,1.46,1.50,1.52,1.43,1.43,1.44,1.51,1.43,1.43,1.44,1.49	300	200	1
14	20	4.88	4.00	7.50	1.49,1.53,1.49,1.54,1.52,1.49,1.64,1.64,1.82,2.16,1.66,1.66,1.72,1.79,1.59,1.59,1.43,1.50,1.63,1.63,1.80,2.09,1.66,1.66,1.71,1.74	300	200	1
15	20	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	20	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	9	18.00	18.00	8.57	-	300	200	1
102	6	1.30	1.30	2.00	-	300	200	1
103	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
104	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.13,1.67,1.67,1.66,1.65	300	200	1
105	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
106	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
107	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1

108	20	1.33	1.33	2.05	1.53,1.53,1.53,1.54,1.53,1.53,1.50,1.50,1.43,1.10,1.49,1.49,1.46,2.25,1.52,1.52,1.61,1.47,1.50,1.50,1.44,1.11,1.49,1.49,1.47,2.34	300	200	1
109	3	2.60	2.60	4.00	-	300	200	1
110	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.13,1.67,1.67,1.66,1.65	300	200	1
111	20	1.33	1.33	2.05	1.41,1.41,1.41,1.40,1.41,1.41,1.27,1.27,1.18,1.16,1.26,1.26,1.23,1.20,1.33,1.33,1.69,2.37,1.27,1.27,1.20,1.18,1.26,1.26,1.24,1.21	300	200	1
112	6	1.30	1.30	2.00	-	300	200	1
113	20	4.88	4.00	7.50	1.06,1.06,1.06,1.07,1.06,1.06,1.10,1.10,3.21,3.72,1.11,1.11,1.34,2.55,1.08,1.08,1.05,1.03,1.10,1.10,2.12,3.67,1.12,1.12,1.23,2.31	300	200	1
114	20	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.06,1.06,1.06,1.02,1.05,1.05,1.05,1.06,1.05,1.05,1.06,1.00,1.06,1.06,1.06,1.03	300	200	1
115	20	4.84	4.84	7.45	1.12,1.12,1.12,1.12,1.12,1.12,1.09,1.09,1.06,1.05,1.08,1.08,1.08,1.06,1.10,1.10,1.18,1.31,1.09,1.09,1.06,1.06,1.08,1.08,1.08,1.07	300	200	1
116	20	4.84	4.84	7.45	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.13,1.05,1.15,1.15,1.14,1.46,1.15,1.15,1.16,1.14,1.15,1.15,1.14,1.65,1.15,1.15,1.14,1.37	300	200	1
201	9	18.00	18.00	8.57	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
204	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.64,1.67,1.67,1.68,1.67,1.67,1.67,1.65,2.46,1.67,1.67,1.66,1.65	300	200	1
205	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.17,1.16,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.18,1.18,1.18,1.16,1.16,1.17,1.17,1.17,1.17	300	200	1
207	17	1.52	1.52	2.33	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
208	20	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
209	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.69,1.70,1.67,1.67,1.65,1.70,1.67,1.67,1.66,1.60,1.67,1.67,1.68,1.67,1.68,1.68,1.65,1.81,1.67,1.67,1.66,1.64	300	200	1
211	20	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
212	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.43,1.46,1.43,1.47,1.45,1.43,1.44,1.44,1.42,1.52,1.43,1.43,1.44,1.50,1.46,1.46,1.50,1.53,1.43,1.43,1.44,1.51,1.43,1.43,1.44,1.49	300	200	1
214	20	4.88	4.00	7.50	1.49,1.53,1.49,1.54,1.51,1.49,1.64,1.64,1.82,2.17,1.66,1.66,1.72,1.80,1.60,1.60,1.43,1.50,1.64,1.64,1.80,2.11,1.66,1.66,1.71,1.75	300	200	1
215	20	4.84	4.84	7.45	1.06,1.06,1.06,1.06,1.06,1.06,1.07,1.07,1.08,1.09,1.07,1.07,1.07,1.08,1.07,1.07,1.06,1.07,1.07,1.07,1.08,1.09,1.07,1.07,1.07,1.08	300	200	1
216	20	4.84	4.84	7.45	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.05,1.20,1.06,1.06,1.06,1.09,1.06,1.06,1.06,1.06,1.06,1.06,1.06,2.08,1.06,1.06,1.06,1.08	300	200	1

Member Design Capacity

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	254.57	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	43.40	6.46	56.26	44.91
14	159.30	116.35	43.70	6.46	56.26	44.91
15	159.30	137.23	46.90	6.46	56.26	44.91
16	159.30	137.23	46.90	6.46	56.26	44.91
101	377.97	254.57	83.29	83.29	113.39	113.39
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.48	6.46	56.26	44.91
114	159.30	116.35	30.56	6.46	56.26	44.91
115	159.30	104.63	32.27	6.46	56.26	44.91
116	159.30	104.63	32.27	6.46	56.26	44.91
201	377.97	254.57	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	137.23	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	137.23	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	43.40	6.46	56.26	44.91
214	159.30	116.35	43.70	6.46	56.26	44.91
215	159.30	104.63	32.58	6.46	56.26	44.91
216	159.30	104.63	32.27	6.46	56.26	44.91

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	0.052	0.578	0.059	0.047	0.007	0.616	#13	0.368	Not Required	Pass
2	0.001	0.282	0.154	0.068	0.032	0.389	#13	0.054	Not Required	Pass
3	0.008	0.470	0.035	0.046	0.006	0.498	#21	0.046	Not Required	Pass
4	0.007	0.443	0.079	0.044	0.018	0.526	#21	0.082	Not Required	Pass
5	0.007	0.291	0.034	0.046	0.008	0.300	#13	0.076	Not Required	Pass
6	0.013	0.644	0.147	0.065	0.044	0.784	#21	0.046	Not Required	Pass
7	0.014	0.399	0.182	0.064	0.045	0.434	#21	0.076	Not Required	Pass
8	0.005	0.122	0.160	0.032	0.025	0.283	#21	0.102	Not Required	Pass
9	0.001	0.048	0.097	0.003	0.006	0.120	#21	0.206	Not Required	Pass
10	0.015	0.585	0.167	0.059	0.038	0.684	#21	0.082	Not Required	Pass
11	0.008	0.134	0.155	0.037	0.025	0.289	#21	0.102	Not Required	Pass
12	0.002	0.464	0.215	0.104	0.040	0.620	#13	0.054	Not Required	Pass
13	0.009	0.068	0.509	0.051	0.035	0.518	#23	0.306	Not Required	Pass
14	0.005	0.067	0.499	0.045	0.035	0.515	#24	0.204	Not Required	Pass
15	0.000	0.005	0.018	0.008	0.005	0.023	#21	Not Required	Not Required	Pass
16	0.000	0.005	0.018	0.008	0.005	0.023	#21	Not Required	Not Required	Pass
101	0.071	0.770	0.000	0.064	0.000	0.799	#13	0.368	Not Required	Pass
102	0.002	0.514	0.260	0.118	0.048	0.702	#21	0.054	Not Required	Pass
103	0.014	0.745	0.099	0.074	0.023	0.850	#21	0.046	Not Required	Pass
104	0.014	0.749	0.185	0.075	0.042	0.890	#21	0.082	Not Required	Pass
105	0.014	0.463	0.190	0.073	0.048	0.510	#21	0.076	Not Required	Pass
106	0.014	0.745	0.099	0.074	0.023	0.850	#21	0.046	Not Required	Pass
107	0.014	0.463	0.190	0.073	0.048	0.510	#21	0.076	Not Required	Pass
108	0.005	0.089	0.157	0.042	0.026	0.223	#21	0.102	Not Required	Pass
109	0.012	0.049	0.055	0.001	0.000	0.106	#21	0.206	Not Required	Pass
110	0.014	0.749	0.185	0.075	0.042	0.890	#21	0.082	Not Required	Pass
111	0.008	0.126	0.159	0.040	0.026	0.252	#21	0.102	Not Required	Pass
112	0.002	0.514	0.260	0.118	0.048	0.702	#21	0.054	Not Required	Pass
113	0.009	0.121	0.531	0.054	0.035	0.626	#21	0.306	Not Required	Pass
114	0.008	0.173	0.528	0.056	0.035	0.672	#21	0.306	Not Required	Pass
115	0.011	0.225	0.282	0.040	0.026	0.503	#21	0.370	Not Required	Pass
116	0.005	0.188	0.279	0.042	0.026	0.465	#21	0.370	Not Required	Pass
201	0.052	0.578	0.059	0.047	0.007	0.616	#13	0.368	Not Required	Pass
202	0.002	0.464	0.215	0.104	0.040	0.620	#13	0.054	Not Required	Pass
203	0.013	0.644	0.147	0.065	0.044	0.784	#21	0.046	Not Required	Pass
204	0.015	0.585	0.167	0.059	0.038	0.684	#21	0.082	Not Required	Pass
205	0.014	0.399	0.181	0.064	0.045	0.434	#21	0.076	Not Required	Pass
206	0.008	0.470	0.035	0.046	0.006	0.498	#21	0.046	Not Required	Pass
207	0.007	0.291	0.034	0.046	0.008	0.301	#13	0.076	Not Required	Pass
208	0.000	0.005	0.018	0.008	0.005	0.023	#21	Not Required	Not Required	Pass
209	0.001	0.048	0.097	0.003	0.006	0.120	#21	0.206	Not Required	Pass
210	0.007	0.443	0.079	0.044	0.018	0.526	#21	0.122	Not Required	Pass
211	0.000	0.005	0.018	0.008	0.005	0.023	#21	Not Required	Not Required	Pass
212	0.001	0.282	0.154	0.068	0.032	0.389	#13	0.054	Not Required	Pass
213	0.009	0.068	0.508	0.051	0.035	0.518	#23	0.204	Not Required	Pass
214	0.005	0.067	0.499	0.045	0.035	0.515	#24	0.306	Not Required	Pass
215	0.011	0.238	0.282	0.037	0.025	0.510	#21	0.370	Not Required	Pass
216	0.005	0.203	0.280	0.032	0.025	0.483	#21	0.370	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/comression)
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

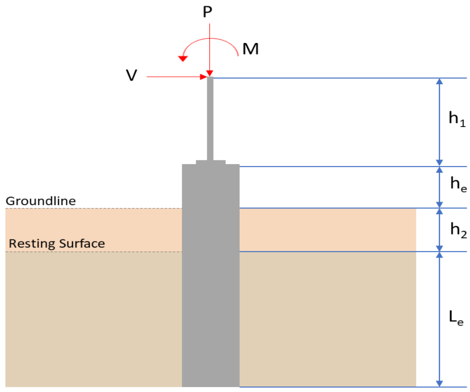
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.379	13.287
V_x (kip)	-3.182	-5.329
V_z (kip)	0.496	0.772
M_x (kipft)	1.056	1.667
M_z (kipft)	28.750	48.106

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.182\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.50669\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.75\ \mathrm{kipft}) + ((-3.182\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 4.578\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:
$L_{e,x} = 5.7602\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.496\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.078981\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.056\ \mathrm{kipft}) + ((0.496\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.16815\ \mathrm{kipft/ft}$$

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:
$L_{e,z} = 3.0297\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.7602\ \mathrm{ft}), (3.0297\ \mathrm{ft})]$$

$$L_{e,req} = 5.76\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.76\ \mathrm{ft})}{(6.25\ \mathrm{ft})}$$

$$Ratio = 0.9216$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

$$A = (48\ \mathrm{in}) \times (48\ \mathrm{in})$$

$$A = 16\ \mathrm{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.379\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.52369\ \mathrm{kip/ft}^2$$

$q = 0.52369\ \text{kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.52369\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.26184$$

Status: **PASS**
Ratio: **0.260**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.50669\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.578\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.578\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (-0.50669\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (4.578\ \text{kipft/ft})) + (4 \times (-0.50669\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.331\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.578\ \text{kipft/ft})) + (3 \times (-0.50669\ \text{kip/ft}) \times (6.25\ \text{ft}))]^{\ 2}}{(6.25\ \text{ft})^2 \times [(3 \times (4.578\ \text{kipft/ft})) + (2 \times (-0.50669\ \text{kip/ft}) \times (6.25\ \text{ft}))]}$$

$$p = 0.20145\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.578\ \text{kipft/ft})) + ((-0.50669\ \text{kip/ft}) \times (6.25\ \text{ft}))]}{(6.25\ \text{ft})^2}$$

$$s = 0.91995\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.331\ \text{ft})}{2}$$

$$p_a = 0.32483\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.20145\ \text{kip/ft}^2)}{(0.32483\ \text{kip/ft}^2)}$$

$$Ratio = 0.62016$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.91995\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.98128$$

Status: **PASS**
Ratio: **0.980**

Considering z-direction:

$H_o = 0.078981\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.16815\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16815\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (0.078981\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.16815\ \mathrm{kipft/ft})) + (4 \times (0.078981\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.5114\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.16815\ \mathrm{kipft/ft})) + (3 \times (0.078981\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^2}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.16815\ \mathrm{kipft/ft})) + (2 \times (0.078981\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.05969\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.16815\ \mathrm{kipft/ft})) + ((0.078981\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.12748\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.5114\ \mathrm{ft})}{2}$$

$$p_a = 0.33835\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.05969\ \mathrm{kip/ft^2}\right)}{\left(0.33835\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.17641$$

Status: **PASS**
Ratio: **0.180**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

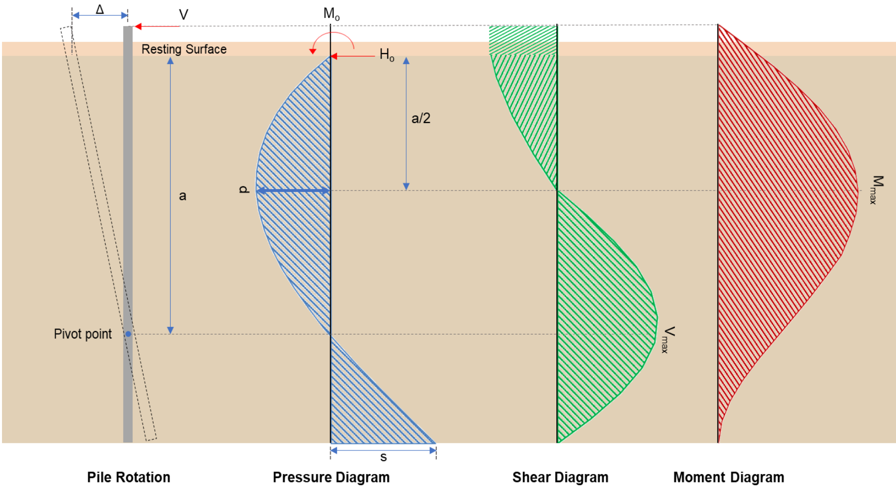
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.12748\ \text{kip/ft}^2)}{(0.9375\ \text{kip/ft}^2)}$$

$$Ratio = 0.13598$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.329\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.84857\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(48.106\ \text{kipft}) + ((-5.329\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 7.6602\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.6602\ \text{kipft/ft})}{(-0.84857\ \text{kip/ft})}$$

$$E = 9.0272\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.6602\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (-0.84857\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (7.6602\ \text{kipft/ft})) + (4 \times (-0.84857\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.3311\ \text{ft}$$

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

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

$$V_{max} = ((-0.84857\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.0272\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3311\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.0272\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3311\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.913\ \text{kip}$$

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

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

$$M_{max} = ((-0.84857\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.25\ \mathrm{ft})) \times \left[\left(\frac{(9.0272\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + \frac{(4.3311\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (9.0272\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3311\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (9.0272\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3311\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 32.182\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.772\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.12293\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.667\ \mathrm{kipft}) + ((0.772\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.26545\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.26545\ \mathrm{kipft/ft})}{(0.12293\ \mathrm{kip/ft})}$$

$$E = 2.1593\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.26545\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (0.12293\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.26545\ \mathrm{kipft/ft})) + (4 \times (0.12293\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.5097\ \mathrm{ft}$$

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

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

$$V_{max} = ((0.12293\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.1593\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.5097\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (2.1593\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.5097\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.63011\ \mathrm{kip}$$

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

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

$$M_{max} = ((0.12293\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.25\ \mathrm{ft})) \times \left[\left(\frac{(2.1593\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + \frac{(4.5097\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (2.1593\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.5097\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (2.1593\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.5097\ \mathrm{ft})}{2 \times (6.25\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 1.6962 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(13.287 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.155 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.155 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

$$Ratio = \frac{\left(4.1472 \text{ in}^2\right)}{\left(4.2951 \text{ in}^2\right)}$$

$$Ratio = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**	

	Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\text{ kip}$$	
	$Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(13.287\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0049668$$	Status: **PASS** Ratio: **0.000**

	Shear Strength (ACI 318-19, LRFD) **Parameters:** $b_w = 48\text{ in}$ - Effective width,	
22.5.2.2	d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\text{ in})$$ $$d = 38.4\text{ in}$$	
22.5.5.1.3	λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(38.4\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\text{ psi})} \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,max} = 296.21\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 13.287\text{ kip} \rightarrow 13287\text{ lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(13287\text{ lbf})}{6 \times (2304\text{ in}^2)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 120.26\text{ kip}$$ 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) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 348.89\text{ kip}$$	
	V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (120.26\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 120.26\text{ kip}$$	

	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$,	
22.5.1.2	$V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$ $$V_{s,a} = 8 \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{s,a} = 737.28\ \mathrm{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\, d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$ $$A_v = 0.11045\ \mathrm{in}^2$$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (38.4\ \mathrm{in})}{(10\ \mathrm{in})}$$ $$V_{s,b} = 50.894\ \mathrm{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$ $$V_s = 50.894\ \mathrm{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.26\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$ $$\phi V_n = 111.25\ \mathrm{kip}$$ **Considering x-direction:** $V_{max} = 10.913\ \mathrm{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(10.913\ \mathrm{kip})}{(111.25\ \mathrm{kip})}$$ $$Ratio = 0.098097$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $V_{max} = 0.63011\ \mathrm{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.63011\ \mathrm{kip})}{(111.25\ \mathrm{kip})}$$ $$Ratio = 0.005664$$	Status: **PASS** Ratio: **0.010**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$ $$S_m = 18432\ \mathrm{in}^3$$	

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\text{ ksi}) \times \left(18432\text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6\text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6\text{ kipft}), (2121.6\text{ kipft})]$$ $$\phi M_n = 249.6\text{ kipft}$$ **Considering x-direction:** $M_{max} = 32.182\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(32.182\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.12893$$	Status: **PASS** Ratio: **0.130**
	Considering z-direction: $M_{max} = 1.6962\text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(1.6962\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.0067959$$	Status: **PASS** Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS

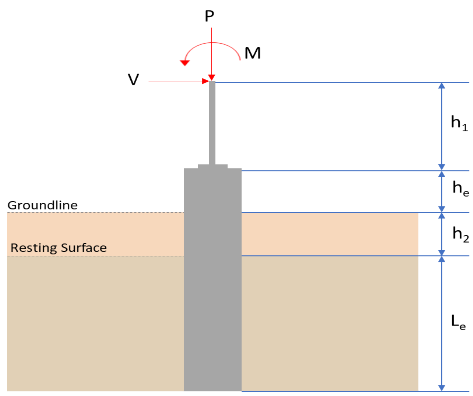
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.379	13.287
V_x (kip)	-3.182	-5.329
V_z (kip)	-0.496	-0.772
M_x (kipft)	-1.056	-1.668
M_z (kipft)	28.750	48.107

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.182\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.50669\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.75\ \text{kipft}) + ((-3.182\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.578\ \text{kipft/ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:
$L_{e,x} = 5.7602\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.496\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.078981\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.056\ \text{kipft}) + ((-0.496\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.16815\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.7355\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.7602\ \text{ft}), (1.7355\ \text{ft})]$$

$$L_{e,req} = 5.76\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.76\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.9216$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

$$A = (48\ \text{in}) \times (48\ \text{in})$$

$$A = 16\ \text{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.379\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.52369\ \text{kip/ft}^2$$

$q = 0.52369 \text{ kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.52369 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.26184$$

Status: **PASS**
Ratio: **0.260**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.50669 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.578 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.578 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.50669 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (4.578 \text{ kipft/ft})) + (4 \times (-0.50669 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.331 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.578 \text{ kipft/ft})) + (3 \times (-0.50669 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.578 \text{ kipft/ft})) + (2 \times (-0.50669 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.20145 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.578 \text{ kipft/ft})) + ((-0.50669 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.91995 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.331 \text{ ft})}{2}$$

$$p_a = 0.32483 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.20145 \text{ kip/ft}^2)}{(0.32483 \text{ kip/ft}^2)}$$

$$Ratio = 0.62016$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.91995 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.98128$$

Status: **PASS**
Ratio: **0.980**

Considering z-direction:

$H_o = -0.078981 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 0.16815 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16815 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.078981 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.16815 \text{ kipft/ft})) + (4 \times (-0.078981 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5114 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.16815 \text{ kipft/ft})) + (3 \times (-0.078981 \text{ kip/ft}) \times (6.25 \text{ ft}))]^{\,2}}{(6.25 \text{ ft})^2 \times [(3 \times (0.16815 \text{ kipft/ft})) + (2 \times (-0.078981 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.025981 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.16815 \text{ kipft/ft})) + ((-0.078981 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.024165 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5114 \text{ ft})}{2}$$

$$p_a = 0.33835 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(-0.025981 \text{ kip/ft}^2)}{(0.33835 \text{ kip/ft}^2)}$$

$$Ratio = -0.076787$$

Status: **PASS**
Ratio: **-0.080**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

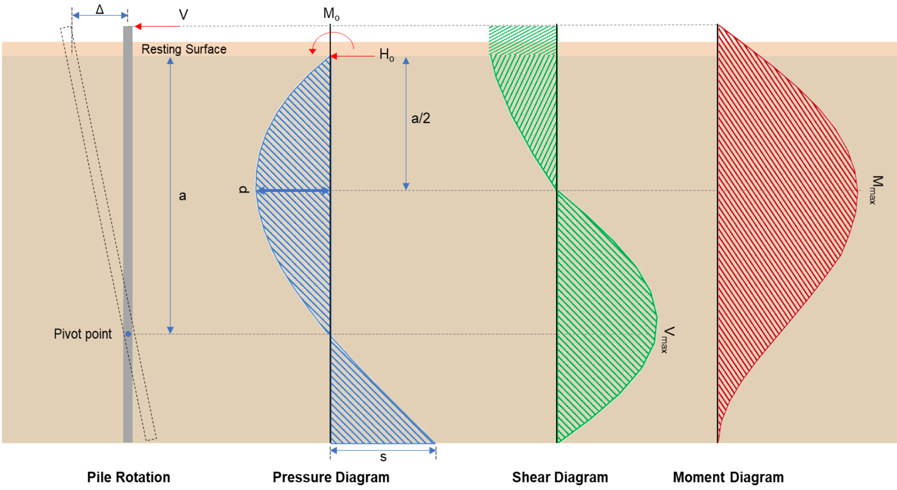
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.024165\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.025776$$

Status: **PASS**
Ratio: **-0.030**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.329\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.84857\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(48.107\ \mathrm{kipft}) + ((-5.329\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 7.6604\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.6604\ \mathrm{kipft/ft})}{(-0.84857\ \mathrm{kip/ft})}$$

$$E = 9.0274\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.6604\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.84857\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (7.6604\ \mathrm{kipft/ft})) + (4 \times (-0.84857\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3311\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.84857\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.0274\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3311\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.0274\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3311\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.913\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.84857\ \text{kip/ft})\times(48\ \text{in})\times(6.25\ \text{ft}))\times\left[\left(\frac{(9.0274\ \text{ft})}{(6.25\ \text{ft})}+\frac{(4.3311\ \text{ft})}{2\times(6.25\ \text{ft})}\right) - \left[\left(\frac{4\times(9.0274\ \text{ft})}{(6.25\ \text{ft})}+3\right)\times\left(\frac{(4.3311\ \text{ft})}{2\times(6.25\ \text{ft})}\right)^3\right]+\left[\left(\frac{3\times(9.0274\ \text{ft})}{(6.25\ \text{ft})}+2\right)\times\left(\frac{(4.3311\ \text{ft})}{2\times(6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 32.182\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.772\ \text{kip})}{1.57\times(48\ \text{in})}$$

$$H_o = -0.12293\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.668\ \text{kipft})+((-0.772\ \text{kip})\times(0\ \text{ft}))}{1.57\times(48\ \text{in})}$$

$$M_o = 0.26561\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.26561\ \text{kipft/ft})}{(-0.12293\ \text{kip/ft})}$$

$$E = 2.1606\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4\times(0.26561\ \text{kipft/ft})\times(6.25\ \text{ft}))+\left(3\times(-0.12293\ \text{kip/ft})\times(6.25\ \text{ft})^2\right)}{(6\times(0.26561\ \text{kipft/ft}))+(4\times(-0.12293\ \text{kip/ft})\times(6.25\ \text{ft}))}$$

$$a = 4.5096\ \text{ft}$$

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

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

$$V_{max} = ((-0.12293\ \text{kip/ft})\times(48\ \text{in}))\times\left[1-\left[3\times\left(\frac{4\times(2.1606\ \text{ft})}{(6.25\ \text{ft})}+3\right)\times\left(\frac{(4.5096\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right]+\left[4\times\left(\frac{3\times(2.1606\ \text{ft})}{(6.25\ \text{ft})}+2\right)\times\left(\frac{(4.5096\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.63028\ \text{kip}$$

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

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

$$M_{max} = ((-0.12293\ \text{kip/ft})\times(48\ \text{in})\times(6.25\ \text{ft}))\times\left[\left(\frac{(2.1606\ \text{ft})}{(6.25\ \text{ft})}+\frac{(4.5096\ \text{ft})}{2\times(6.25\ \text{ft})}\right) - \left[\left(\frac{4\times(2.1606\ \text{ft})}{(6.25\ \text{ft})}+3\right)\times\left(\frac{(4.5096\ \text{ft})}{2\times(6.25\ \text{ft})}\right)^3\right]+\left[\left(\frac{3\times(2.1606\ \text{ft})}{(6.25\ \text{ft})}+2\right)\times\left(\frac{(4.5096\ \text{ft})}{2\times(6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 1.6968 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(13.287\text{ kip})}{(0.65)\times(0.8)} - \left(0.85\times(2.5\text{ ksi})\times\left(2304\text{ in}^2\right)\right)}{(60\text{ ksi}) - (0.85\times(2.5\text{ ksi}))}, \left(0.08\times\left(2304\text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.155 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.155\text{ in}^2\right), \left(0.0018\times\left(2304\text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(4.1472\text{ in}^2\right)}{\left(0.3068\text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

$$Ratio = \frac{\left(4.1472\text{ in}^2\right)}{\left(4.2951\text{ in}^2\right)}$$

$$Ratio = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**		
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(13.287\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0049668$$	Status: **PASS** Ratio: **0.000**	
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a) 22.5.5.1.2	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48\text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\text{ in})$$ $$d = 38.4\text{ in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ 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 $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\text{ psi})} \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,max} = 296.21\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 13.287\text{ kip} \rightarrow 13287\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(13287\text{ lbf})}{6 \times \left(2304\text{ in}^2\right)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 120.26\text{ kip}$$ 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) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 348.89\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (120.26\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 120.26\text{ kip}$$		

22.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$ $$V_{s,a} = 8 \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{s,a} = 737.28\ \text{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\, d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \text{in})^2}{4}$$ $$A_v = 0.11045\ \text{in}^2$$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \text{in}^2\right) \times (60\ \text{ksi}) \times (38.4\ \text{in})}{(10\ \text{in})}$$ $$V_{s,b} = 50.894\ \text{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28\ \text{kip}), (50.894\ \text{kip})\right]$$ $$V_s = 50.894\ \text{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.26\ \text{kip}) + (50.894\ \text{kip}))$$ $$\phi V_n = 111.25\ \text{kip}$$ **Considering x-direction:** $V_{max} = 10.913\ \text{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(10.913\ \text{kip})}{(111.25\ \text{kip})}$$ $$Ratio = 0.098098$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $V_{max} = 0.63028\ \text{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.63028\ \text{kip})}{(111.25\ \text{kip})}$$ $$Ratio = 0.0056656$$	Status: **PASS** Ratio: **0.010**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48\ \text{in}) \times (48\ \text{in})^2}{6}$$ $$S_m = 18432\ \text{in}^3$$	

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 32.182 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(32.182 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.12894$$	Status: **PASS** Ratio: **0.130**
	Considering z-direction: $M_{max} = 1.6968 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(1.6968 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0067981$$	Status: **PASS** Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS

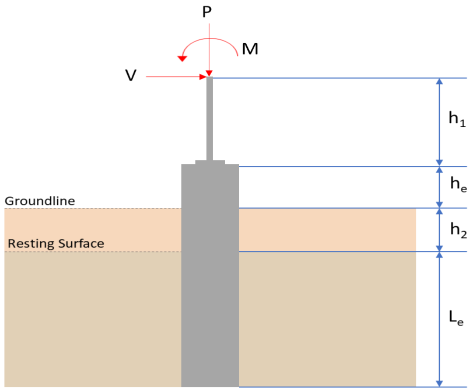
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.75\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	11.373	18.073
V_x (kip)	-4.334	-7.292
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	37.724	64.139

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.334\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.69013\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(37.724\ \mathrm{kipft}) + ((-4.334\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.007\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 6.1065\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.1065\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 6.106\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6.75\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.106\ \mathrm{ft})}{(6.75\ \mathrm{ft})}$$

$$Ratio = 0.90459$$

Status: PASS
Ratio: 0.900

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

$$A = (48\ \mathrm{in}) \times (48\ \mathrm{in})$$

$$A = 16\ \mathrm{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.373\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.71081\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.71081\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.35541$$

Status: PASS
Ratio: 0.360

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.69013\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 6.007\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.007\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.69013\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (6.007\ \mathrm{kipft/ft})) + (4 \times (-0.69013\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6917\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.007\ \mathrm{kipft/ft})) + (3 \times (-0.69013\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^{\ 2}}{(6.75\ \mathrm{ft})^2 \times [(3 \times (6.007\ \mathrm{kipft/ft})) + (2 \times (-0.69013\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = 0.19112\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (6.007\ \mathrm{kipft/ft})) + ((-0.69013\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.96865\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.6917\ \mathrm{ft})}{2}$$

$$p_a = 0.35188\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.19112\ \mathrm{kip/ft^2})}{(0.35188\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.54314$$

Status: **PASS**
Ratio: **0.540**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

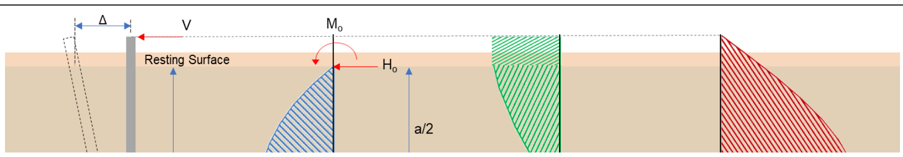
$Ratio$ - Lateral soil capacity

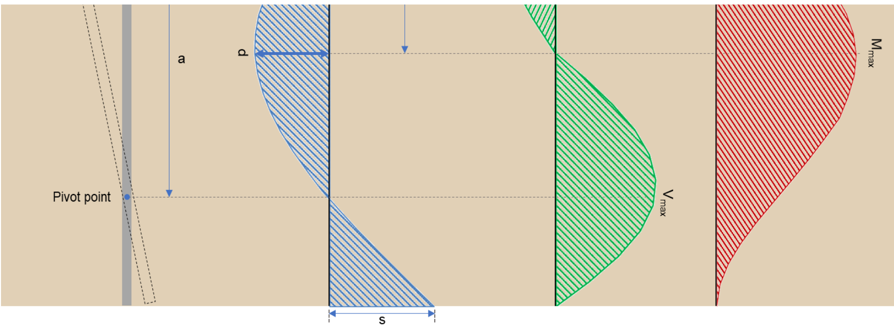
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.96865\ \mathrm{kip/ft^2})}{(1.0125\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.95669$$

Status: **PASS**
Ratio: **0.960**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.292\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -1.1611\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(64.139\text{ kipft}) + ((-7.292\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 10.213\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.213\text{ kipft/ft})}{(-1.1611\text{ kip/ft})}$$

$$E = 8.7958\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.213\text{ kipft/ft}) \times (6.75\text{ ft})) + \left(3 \times (-1.1611\text{ kip/ft}) \times (6.75\text{ ft})^2\right)}{(6 \times (10.213\text{ kipft/ft})) + (4 \times (-1.1611\text{ kip/ft}) \times (6.75\text{ ft}))}$$

$$a = 4.6904\text{ ft}$$

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

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

$$V_{max} = ((-1.1611\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.7958\text{ ft})}{(6.75\text{ ft})} + 3\right) \times \left(\frac{(4.6904\text{ ft})}{(6.75\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (8.7958\text{ ft})}{(6.75\text{ ft})} + 2\right) \times \left(\frac{(4.6904\text{ ft})}{(6.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 13.773\text{ kip}$$

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

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

$$M_{max} = ((-1.1611\text{ kip/ft}) \times (48\text{ in}) \times (6.75\text{ ft})) \times \left[\left(\frac{(8.7958\text{ ft})}{(6.75\text{ ft})} + \frac{(4.6904\text{ ft})}{2 \times (6.75\text{ ft})}\right) - \left[\left(\frac{4 \times (8.7958\text{ ft})}{(6.75\text{ ft})} + 3\right) \times \left(\frac{(4.6904\text{ ft})}{2 \times (6.75\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (8.7958\text{ ft})}{(6.75\text{ ft})} + 2\right) \times \left(\frac{(4.6904\text{ ft})}{2 \times (6.75\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 43.647 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(18.073 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -83.996 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-83.996 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

$$Ratio = \frac{\left(4.1472 \text{ in}^2\right)}{\left(4.2951 \text{ in}^2\right)}$$

$$Ratio = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**	

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

$$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$

$$\phi P_N = 2675.2\ \mathrm{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.073\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.0067558$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \mathrm{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,max} = 296.21\ \mathrm{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 18.073\ \mathrm{kip} \rightarrow 18073\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(18073\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 120.9\ \mathrm{kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (120.9\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 120.9\ \mathrm{kip}$$

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

22.5.1.2 $V_{s,a}$ - 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,

$$A_v = \frac{\pi\, d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375\ \text{in})^2}{4}$$

$$A_v = 0.11045\ \text{in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2\ A_v\ f_{ywk}\ d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times \left(0.11045\ \text{in}^2\right) \times (60\ \text{ksi}) \times (38.4\ \text{in})}{(10\ \text{in})}$$

$$V_{s,b} = 50.894\ \text{kip}$$

V_s - Governing shear strength of steel

$$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$

$$V_s = MIN\left[(737.28\ \text{kip}), (50.894\ \text{kip})\right]$$

$$V_s = 50.894\ \text{kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\ (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((120.9\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 111.66\ \text{kip}$$

Considering x-direction:

$V_{max} = 13.773\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(13.773\ \text{kip})}{(111.66\ \text{kip})}$$

$$Ratio = 0.12334$$

Status: **PASS**
Ratio: **0.120**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b\ D^2}{6}$$

$$S_m = \frac{(48\ \text{in}) \times (48\ \text{in})^2}{6}$$

$$S_m = 18432\ \text{in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$

$$\phi M_{n,1} = 249.600kipft$$

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi\, 0.85\, f'_c\, S_m$$

$$\phi M_{n,2} = \phi\ 0.85\ f_{ck}\ S_m$$

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

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

Therefore,
ϕM_n - Allowable flexural strength,

$$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = MIN[(249.6\ \mathrm{kipft}), (2121.6\ \mathrm{kipft})]$$

$$\phi M_n = 249.6\ \mathrm{kipft}$$

Considering x-direction:

$M_{max} = 43.647\ \mathrm{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(43.647\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$

$$Ratio = 0.17487$$

Status: **PASS**
Ratio: **0.170**