

# Your Project Calculations



Project Name: HorshamPAHotelArea3-JB-RevA

S3D Model Link:  
[https://platform.skyciv.com/structural?preload\\_name=HorshamPAHotelArea3-JB-RevA&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/8\\_2023](https://platform.skyciv.com/structural?preload_name=HorshamPAHotelArea3-JB-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023)

Public Model Link:  
[https://platform.skyciv.com/structural-viewer?project\\_id=UTZULbyVdK1da7qOsyRnyQciDIqCLcOyx0kpgTqwrNHYOwo3GjvSvcCVID10boNH](https://platform.skyciv.com/structural-viewer?project_id=UTZULbyVdK1da7qOsyRnyQciDIqCLcOyx0kpgTqwrNHYOwo3GjvSvcCVID10boNH)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	4P-19.75-8TOP-XD-72-L-5Hx10W-F142
<b>Duty Classification:</b>	XD
<b>Module Width:</b>	51.30 in
<b>Module Length:</b>	93.86in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	10
<b>Total Number of Modules:</b>	50
<b>Desired Tilt Angle:</b>	19
<b>Front Edge Clearance:</b>	14.5
<b>Total Array Height at Tilt:</b>	21.49 ft
<b>Total Frame Length:</b>	78.75 ft
<b>Frame Weight:</b>	5039 lbs
<b>Array Dimensions N/S:</b>	21.58 ft
<b>Array Dimensions E/W:</b>	79.05 ft
<b>Rail Length:</b>	259.00 in
<b>Rail Spacing:</b>	3.91 ft
<b>Rail Check:</b>	Not Checked

## Support Specifications

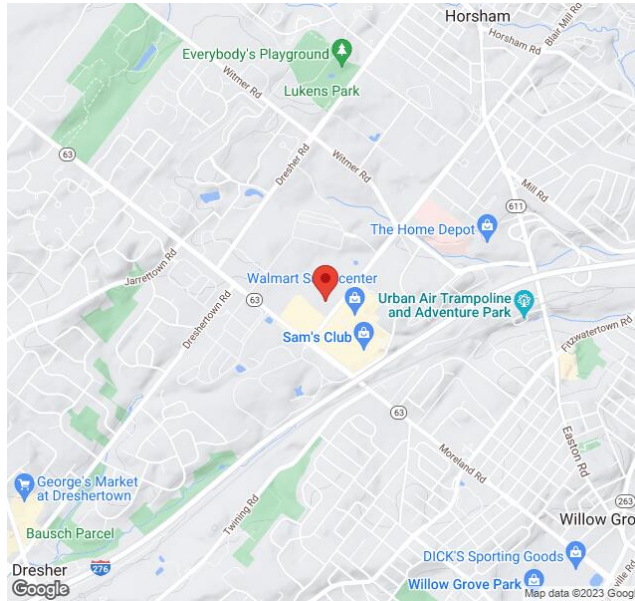
<b>Pole Size:</b>	8in Pipe Sch 40
<b>Pole Length above Grade:</b>	18.01 ft
<b>Number of Poles:</b>	4
<b>Pole Spacing:</b>	19.75 ft

## Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 7.25 ft Pile 2: 7.00 ft Pile 3: 7.00 ft Pile 4: 7.25 ft
<b>Foundation Volume:</b>	16.889 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.391475 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	720 Blair Mill Rd, Horsham, PA 19044, USA
<b>Wind Speed:</b>	105 mph
<b>Snow Load:</b>	30 psf
<b>Design Uplift Pressure:</b>	Multiple pressures
<b>Design Downforce Pressure:</b>	Multiple pressures
<b>Design Snow Pressure:</b>	0.016824 ksf



### Design Disclaimer

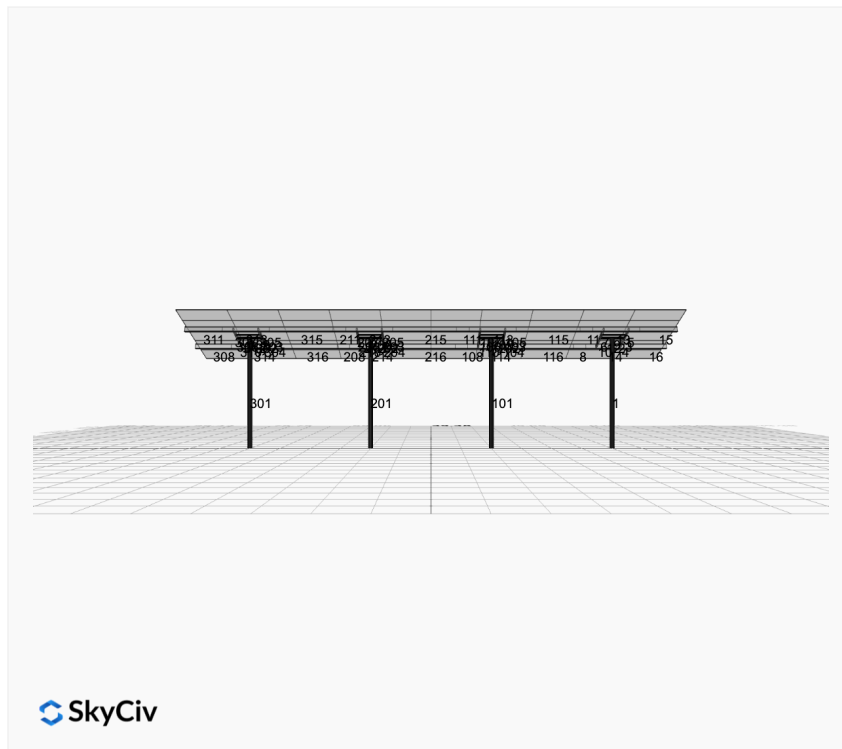
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

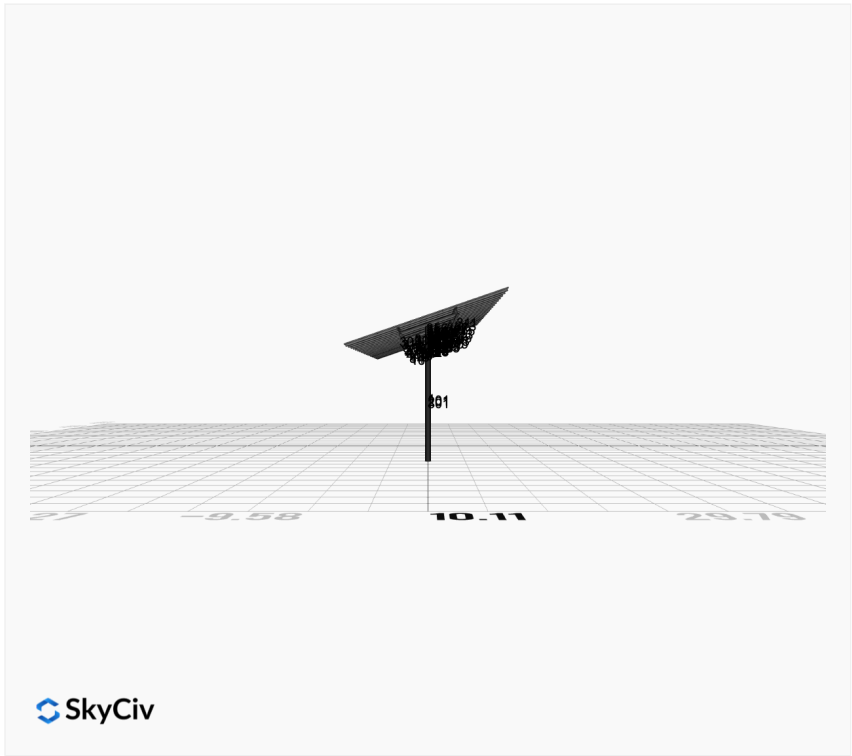
### AutoDesigner Input

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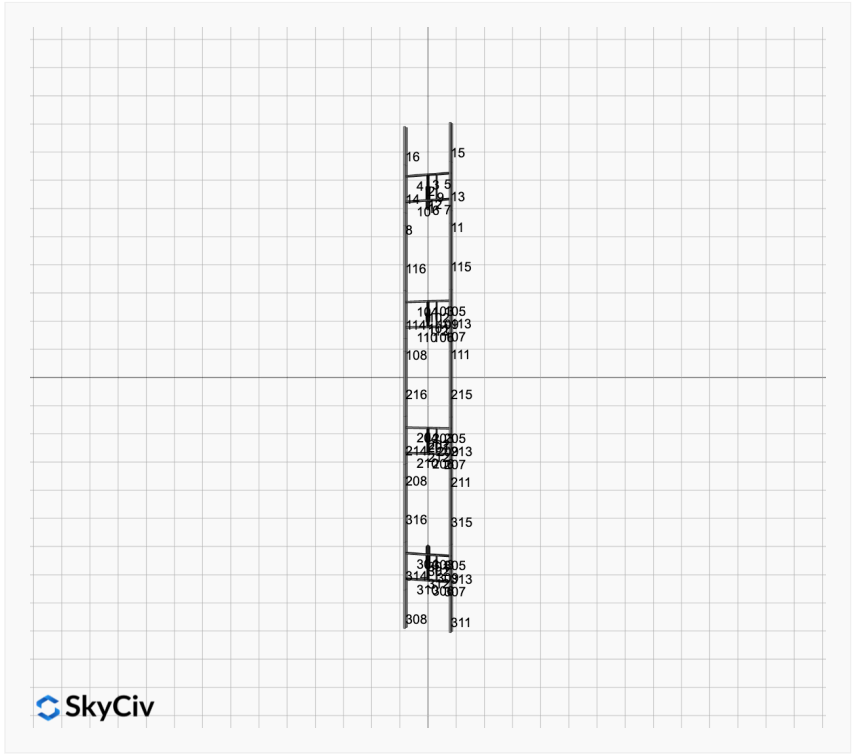
### Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent

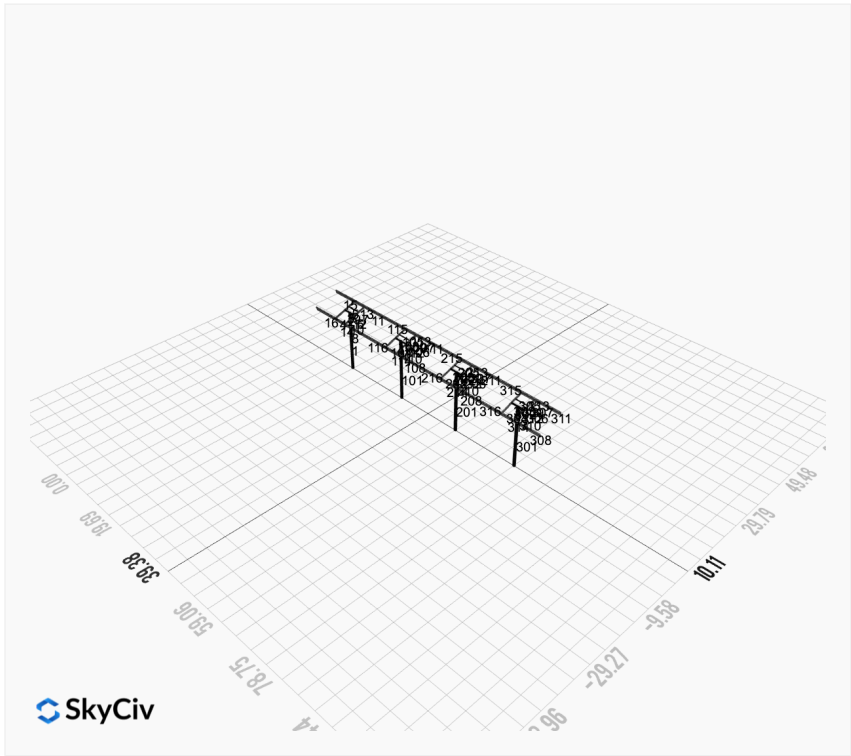




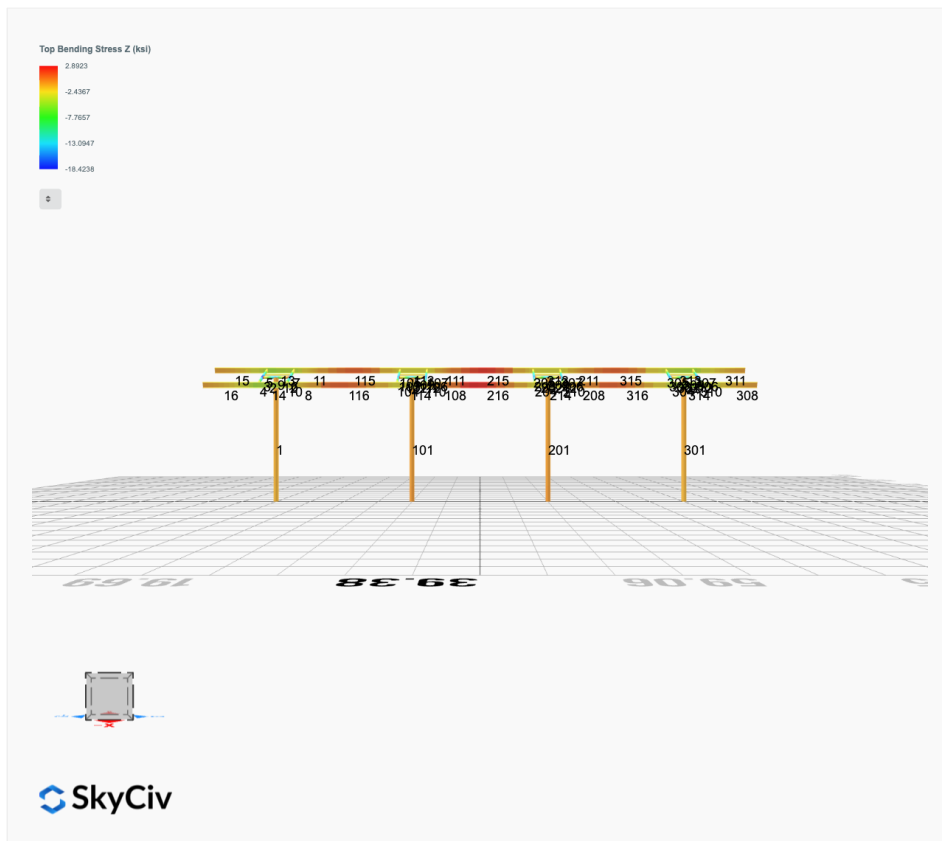
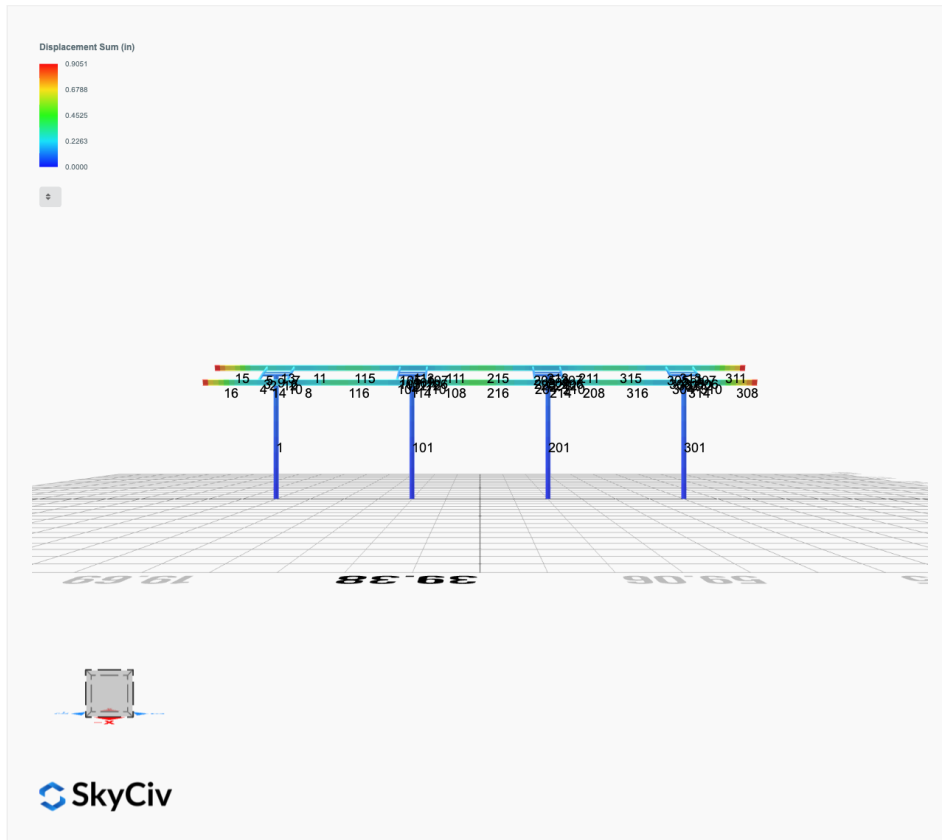
SkyCiv

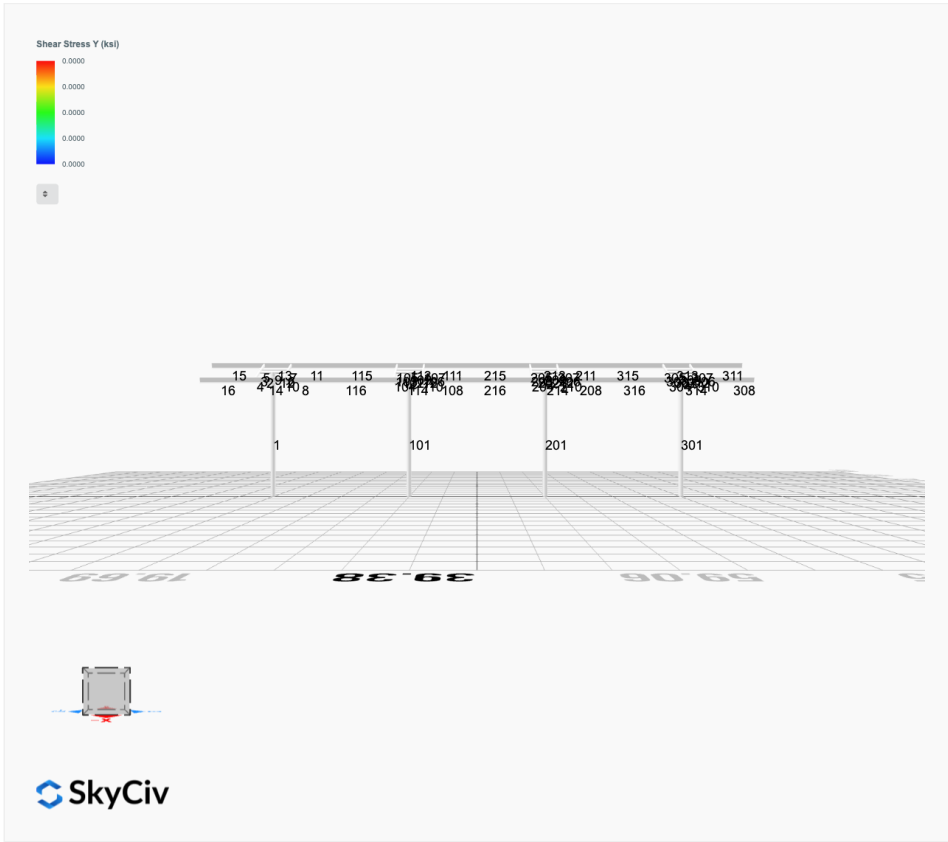
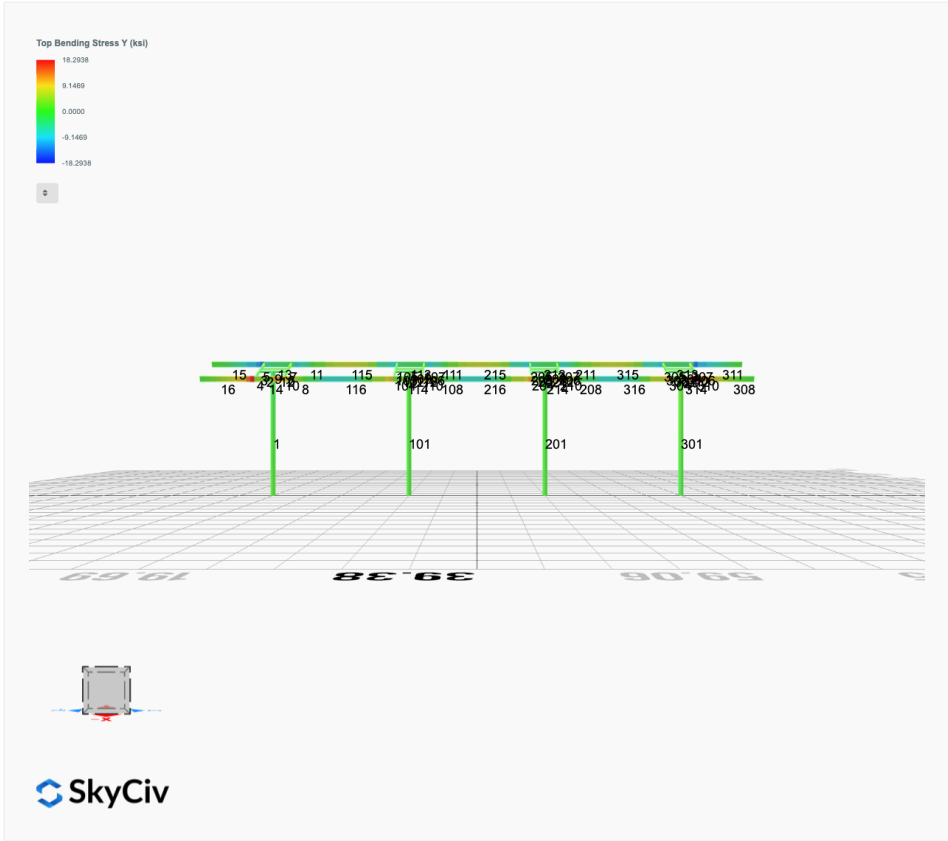


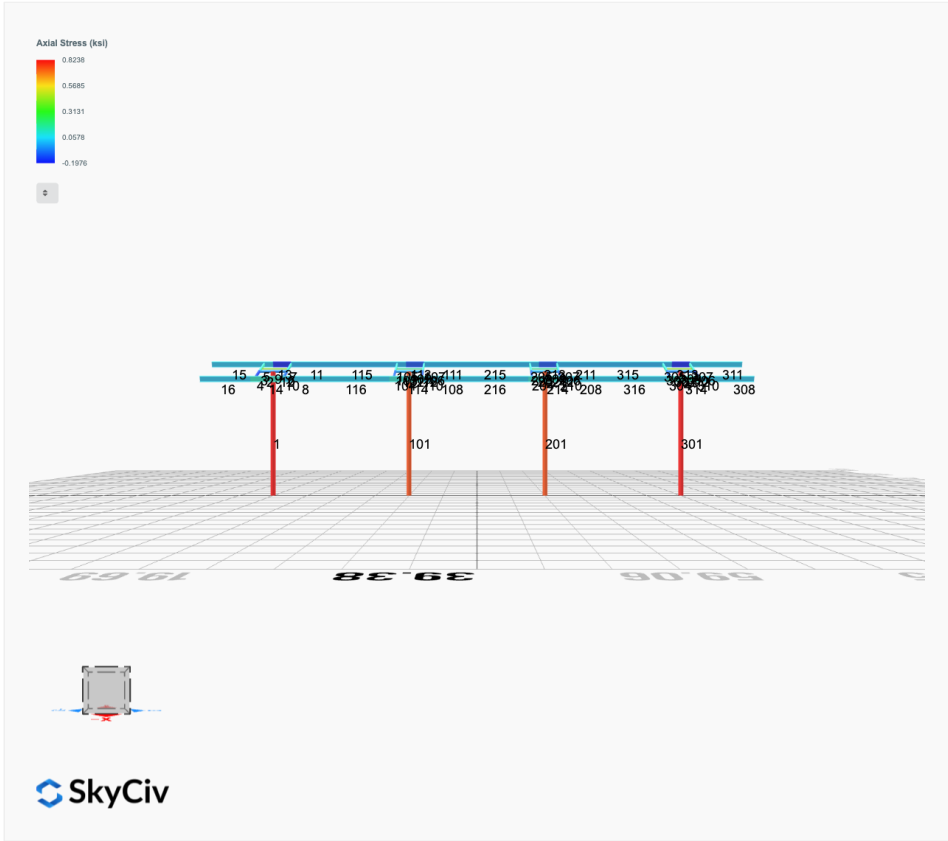
SkyCiv



# FEM Results (Envelope Worst Case for each member)







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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0141	3.1713	-0.0355	-0.2064	0.0465	0.2617
ULS: 2. D + L	-0.0141	3.1713	-0.0355	-0.2064	0.0465	0.2617
ULS: 3. D + (S or Lr or R)	-0.0542	10.0905	-0.1384	-0.8081	0.1815	0.9644
ULS: 3. D + (S or Lr or R)	-0.0141	3.1713	-0.0355	-0.2064	0.0465	0.2617
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0441	8.3607	-0.1127	-0.6576	0.1478	0.7887
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0141	3.1713	-0.0355	-0.2064	0.0465	0.2617
ULS: 5b. D + 0.7E	-0.0141	3.1713	-0.0355	-0.2064	0.0465	0.2617
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0441	8.3607	-0.1127	-0.6576	0.1478	0.7887
ULS: 8. 0.6D + 0.7E	-0.0085	1.9028	-0.0213	-0.1238	0.0279	0.1570
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6788	8.0574	-0.1150	-0.6704	0.1813	33.3974
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6788	8.0574	-0.1150	-0.6704	0.1813	33.3974
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3762	-0.8979	0.0281	0.1606	-0.0599	-22.2205
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1839	-0.3562	0.0246	0.1403	-0.0573	-31.6451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2927	12.0253	-0.1723	-1.0056	0.2489	25.6405
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2927	12.0253	-0.1723	-1.0056	0.2489	25.6405
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9986	5.3088	-0.0650	-0.3824	0.0680	-16.0729
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8544	5.7151	-0.0677	-0.3976	0.0699	-23.1414
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2626	6.8359	-0.0951	-0.5544	0.1476	25.1135
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2626	6.8359	-0.0951	-0.5544	0.1476	25.1135
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0286	0.1194	0.0122	0.0688	-0.0333	-16.5999
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8844	0.5257	0.0096	0.0536	-0.0313	-23.6684
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6732	6.7889	-0.1008	-0.5878	0.1628	33.2928
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6732	6.7889	-0.1008	-0.5878	0.1628	33.2928
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3818	-2.1664	0.0423	0.2431	-0.0785	-22.3251
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1896	-1.6247	0.0388	0.2229	-0.0759	-31.7498

### 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	18.9453
Shear X	-2.8070
Shear Z	-0.2771
Moment X	-1.6310
Moment Y (Twist)	0.3911
Moment Z	59.4411

### 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	12.0253
Shear X	-1.6788
Shear Z	-0.1723
Moment X	-1.0056
Moment Y (Twist)	0.2489
Moment Z	33.3974

## 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	0.0141	3.0606	0.0050	0.0293	-0.0051	-0.1950
ULS: 2. D + L	0.0141	3.0606	0.0050	0.0293	-0.0051	-0.1950
ULS: 3. D + (S or Lr or R)	0.0542	9.6606	0.0192	0.1135	-0.0194	-0.8339
ULS: 3. D + (S or Lr or R)	0.0141	3.0606	0.0050	0.0293	-0.0051	-0.1950
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0441	8.0106	0.0156	0.0924	-0.0158	-0.6742
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0141	3.0606	0.0050	0.0293	-0.0051	-0.1950
ULS: 5b. D + 0.7E	0.0141	3.0606	0.0050	0.0293	-0.0051	-0.1950

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0441	8.0106	0.0156	0.0924	-0.0158	-0.6742
ULS: 8. 0.6D + 0.7E	0.0085	1.8364	0.0030	0.0176	-0.0031	-0.1170
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5980	7.6911	0.0161	0.0952	-0.0138	31.9107
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5980	7.6911	0.0161	0.0952	-0.0138	31.9107
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3511	-0.7908	-0.0057	-0.0327	0.0061	-21.9517
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1838	-0.2882	-0.0019	-0.0112	-0.0025	-31.3138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1649	11.4834	0.0240	0.1418	-0.0223	23.4051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1649	11.4834	0.0240	0.1418	-0.0223	23.4051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0469	5.1220	0.0077	0.0459	-0.0074	-16.9917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9214	5.4990	0.0105	0.0620	-0.0138	-24.0133
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1950	6.5335	0.0133	0.0787	-0.0116	23.8842
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1950	6.5335	0.0133	0.0787	-0.0116	23.8842
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0168	0.1720	-0.0030	-0.0172	0.0033	-16.5125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8914	0.5490	-0.0002	-0.0011	-0.0031	-23.5341
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6036	6.4668	0.0141	0.0835	-0.0118	31.9887
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6036	6.4668	0.0141	0.0835	-0.0118	31.9887
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3455	-2.0151	-0.0077	-0.0444	0.0082	-21.8737
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1782	-1.5125	-0.0039	-0.0229	-0.0004	-31.2358

### 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	18.0941
Shear X	-2.6868
Shear Z	0.0364
Moment X	0.2177
Moment Y (Twist)	0.0313
Moment Z	56.3530

### 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	11.4834
Shear X	-1.6036
Shear Z	0.0240
Moment X	0.1418
Moment Y (Twist)	0.0223
Moment Z	31.9887

### 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	0.0141	3.0606	-0.0050	-0.0293	0.0051	-0.1950
ULS: 2. D + L	0.0141	3.0606	-0.0050	-0.0293	0.0051	-0.1950
ULS: 3. D + (S or Lr or R)	0.0542	9.6606	-0.0192	-0.1134	0.0193	-0.8339
ULS: 3. D + (S or Lr or R)	0.0141	3.0606	-0.0050	-0.0293	0.0051	-0.1950
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0441	8.0106	-0.0156	-0.0924	0.0157	-0.6742
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0141	3.0606	-0.0050	-0.0293	0.0051	-0.1950
ULS: 5b. D + 0.7E	0.0141	3.0606	-0.0050	-0.0293	0.0051	-0.1950
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0441	8.0106	-0.0156	-0.0924	0.0157	-0.6742
ULS: 8. 0.6D + 0.7E	0.0085	1.8364	-0.0030	-0.0176	0.0031	-0.1170
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5980	7.6911	-0.0161	-0.0952	0.0138	31.9107
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5980	7.6911	-0.0161	-0.0952	0.0138	31.9107
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3511	-0.7908	0.0057	0.0327	-0.0061	-21.9517
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1838	-0.2882	0.0019	0.0112	0.0025	-31.3138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1649	11.4834	-0.0240	-0.1418	0.0223	23.4051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1649	11.4834	-0.0240	-0.1418	0.0223	23.4051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0469	5.1220	-0.0077	-0.0459	0.0073	-16.9917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9214	5.4990	-0.0105	-0.0620	0.0138	-24.0133

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	-1.1950	6.5335	-0.0133	-0.0787	0.0116	23.8842
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1950	6.5335	-0.0133	-0.0787	0.0116	23.8842
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0168	0.1720	0.0030	0.0172	-0.0033	-16.5125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8914	0.5490	0.0002	0.0011	0.0031	-23.5341
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6036	6.4668	-0.0141	-0.0835	0.0118	31.9887
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6036	6.4668	-0.0141	-0.0835	0.0118	31.9887
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3455	-2.0151	0.0077	0.0444	-0.0082	-21.8737
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1782	-1.5125	0.0039	0.0229	0.0004	-31.2358

#### 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	18.0941
Shear X	-2.6868
Shear Z	-0.0364
Moment X	-0.2174
Moment Y (Twist)	0.0310
Moment Z	56.3532

#### 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	11.4834
Shear X	-1.6036
Shear Z	-0.0240
Moment X	-0.1418
Moment Y (Twist)	0.0223
Moment Z	31.9887

#### Reaction Forces for Foundation 4 (Node ID#301), (kip, kip-ft)

##### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0141	3.1713	0.0355	0.2064	-0.0465	0.2617
ULS: 2. D + L	-0.0141	3.1713	0.0355	0.2064	-0.0465	0.2617
ULS: 3. D + (S or Lr or R)	-0.0542	10.0905	0.1384	0.8081	-0.1816	0.9643
ULS: 3. D + (S or Lr or R)	-0.0141	3.1713	0.0355	0.2064	-0.0465	0.2617
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0441	8.3607	0.1127	0.6577	-0.1478	0.7886
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0141	3.1713	0.0355	0.2064	-0.0465	0.2617
ULS: 5b. D + 0.7E	-0.0141	3.1713	0.0355	0.2064	-0.0465	0.2617
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0441	8.3607	0.1127	0.6577	-0.1478	0.7886
ULS: 8. 0.6D + 0.7E	-0.0085	1.9028	0.0213	0.1238	-0.0279	0.1570
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6788	8.0574	0.1150	0.6704	-0.1814	33.3974
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6788	8.0574	0.1150	0.6704	-0.1814	33.3974
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3762	-0.8979	-0.0281	-0.1606	0.0599	-22.2205
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1839	-0.3562	-0.0246	-0.1403	0.0573	-31.6451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2927	12.0253	0.1723	1.0057	-0.2490	25.6405
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2927	12.0253	0.1723	1.0057	-0.2490	25.6405
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9986	5.3088	0.0650	0.3825	-0.0680	-16.0730
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8544	5.7151	0.0677	0.3977	-0.0700	-23.1415
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2626	6.8359	0.0951	0.5544	-0.1476	25.1135
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2626	6.8359	0.0951	0.5544	-0.1476	25.1135
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0286	0.1194	-0.0122	-0.0688	0.0333	-16.5999
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8844	0.5257	-0.0096	-0.0536	0.0313	-23.6684
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6732	6.7889	0.1008	0.5878	-0.1628	33.2928
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6732	6.7889	0.1008	0.5878	-0.1628	33.2928
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3818	-2.1664	-0.0423	-0.2431	0.0785	-22.3251
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1896	-1.6247	-0.0388	-0.2228	0.0759	-31.7498

#### Worst Case Reactions LRFD

#### Worst Case Reactions ASD

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	18.9453
Shear X	-2.8070
Shear Z	0.2771
Moment X	1.6313
Moment Y (Twist)	0.3915
Moment Z	59.4417

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	12.0253
Shear X	-1.6788
Shear Z	0.1723
Moment X	1.0057
Moment Y (Twist)	0.2490
Moment Z	33.3974

## Project Details

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

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

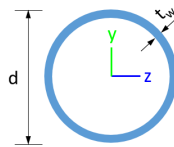


## Design Input Information

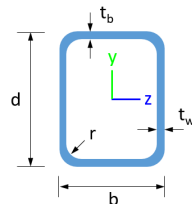
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

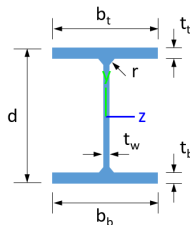
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)					
3	2in Pipe Sch 120	2.38	0.25					
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 <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
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







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	32.09	6.46	56.26	44.91
114	159.30	116.35	32.39	6.46	56.26	44.91
115	159.30	75.13	22.34	6.46	56.26	44.91
116	159.30	75.13	22.53	6.46	56.26	44.91
201	377.97	79.47	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	142.47	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	142.47	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	32.09	6.46	56.26	44.91
214	159.30	116.35	32.39	6.46	56.26	44.91
215	159.30	75.13	21.57	6.46	56.26	44.91
216	159.30	75.13	21.96	6.46	56.26	44.91
301	377.97	79.47	83.29	83.29	113.39	113.39
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	21.54	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	21.54	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	116.35	33.01	6.46	56.26	44.91
314	159.30	116.35	33.01	6.46	56.26	44.91
315	159.30	75.13	26.19	6.46	56.26	44.91
316	159.30	75.13	24.27	6.46	56.26	44.91

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	φ	Status
1	0.238	0.714	0.040	0.025	0.002	0.827	#13	0.773	Not Required	Pass
2	0.004	0.644	0.121	0.131	0.020	0.749	#21	0.036	Not Required	Pass
3	0.008	0.893	0.047	0.090	0.004	0.944	#21	0.046	Not Required	Pass
4	0.008	0.863	0.177	0.086	0.026	0.957	#21	0.033	Not Required	Pass

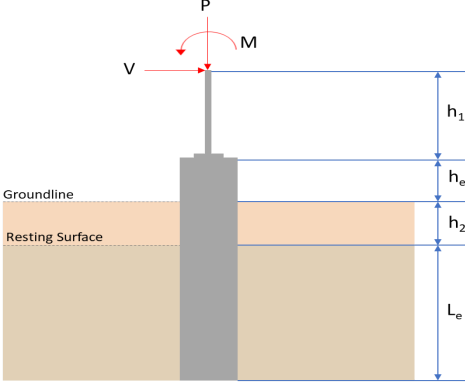
4	0.008	0.554	0.190	0.088	0.049	0.602	#21	0.076	Not Required	Warn
5	0.007	0.775	0.041	0.076	0.006	0.801	#21	0.046	Not Required	Pass
6	0.007	0.481	0.149	0.077	0.040	0.527	#21	0.076	Not Required	Pass
7	0.001	0.109	0.163	0.051	0.016	0.271	#21	0.102	Not Required	Pass
8	0.020	0.107	0.052	0.003	0.001	0.170	#21	0.206	Not Required	Pass
9	0.007	0.759	0.157	0.076	0.035	0.872	#21	0.082	Not Required	Pass
10	0.002	0.106	0.163	0.052	0.016	0.269	#21	0.068	Not Required	Pass
11	0.005	0.520	0.108	0.113	0.018	0.612	#21	0.036	Not Required	Pass
12	0.009	0.411	0.523	0.065	0.020	0.924	#21	0.204	Not Required	Pass
13	0.009	0.410	0.523	0.065	0.020	0.912	#21	0.204	Not Required	Pass
14	0.000	0.169	0.314	0.047	0.015	0.482	#21	Not Required	Not Required	Pass
15	0.000	0.164	0.314	0.046	0.015	0.478	#21	Not Required	Not Required	Pass
101	0.228	0.677	0.005	0.024	0.000	0.771	#13	0.773	Not Required	Pass
102	0.002	0.539	0.104	0.115	0.018	0.623	#21	0.036	Not Required	Pass
103	0.007	0.793	0.044	0.079	0.006	0.840	#21	0.046	Not Required	Pass
104	0.007	0.755	0.120	0.075	0.026	0.837	#21	0.082	Not Required	Pass
105	0.007	0.492	0.122	0.078	0.031	0.523	#21	0.076	Not Required	Pass
106	0.008	0.804	0.051	0.080	0.009	0.859	#21	0.046	Not Required	Pass
107	0.008	0.499	0.128	0.079	0.033	0.530	#21	0.076	Not Required	Pass
108	0.001	0.072	0.134	0.046	0.015	0.157	#21	0.102	Not Required	Pass
109	0.011	0.076	0.037	0.001	0.000	0.118	#21	0.206	Not Required	Pass
110	0.008	0.775	0.123	0.077	0.027	0.859	#21	0.082	Not Required	Pass
111	0.002	0.064	0.135	0.048	0.015	0.170	#21	0.102	Not Required	Pass
112	0.002	0.555	0.105	0.117	0.018	0.640	#21	0.036	Not Required	Pass
113	0.007	0.254	0.349	0.062	0.020	0.567	#21	0.306	Not Required	Pass
114	0.008	0.233	0.348	0.060	0.020	0.542	#21	0.306	Not Required	Pass
115	0.002	0.203	0.190	0.044	0.015	0.389	#21	0.338	Not Required	Pass
116	0.002	0.206	0.193	0.042	0.015	0.395	#21	0.507	Not Required	Pass
201	0.228	0.677	0.005	0.024	0.000	0.771	#13	0.773	Not Required	Pass
202	0.002	0.555	0.105	0.117	0.018	0.640	#21	0.036	Not Required	Pass
203	0.008	0.804	0.051	0.080	0.009	0.859	#21	0.046	Not Required	Pass
204	0.008	0.775	0.123	0.077	0.027	0.859	#21	0.082	Not Required	Pass
205	0.008	0.499	0.128	0.079	0.033	0.530	#21	0.076	Not Required	Pass
206	0.007	0.793	0.044	0.079	0.006	0.840	#21	0.046	Not Required	Pass
207	0.007	0.492	0.122	0.078	0.031	0.523	#21	0.076	Not Required	Pass
208	0.001	0.046	0.121	0.042	0.015	0.151	#24	0.102	Not Required	Pass
209	0.011	0.076	0.037	0.001	0.000	0.118	#21	0.206	Not Required	Pass
210	0.007	0.755	0.120	0.075	0.026	0.837	#21	0.082	Not Required	Pass
211	0.002	0.050	0.122	0.044	0.015	0.156	#21	0.068	Not Required	Pass
212	0.002	0.539	0.104	0.115	0.018	0.623	#21	0.036	Not Required	Pass
213	0.007	0.254	0.349	0.062	0.020	0.567	#21	0.204	Not Required	Pass
214	0.008	0.233	0.348	0.060	0.020	0.542	#21	0.306	Not Required	Pass
215	0.002	0.299	0.190	0.048	0.015	0.490	#21	0.507	Not Required	Pass
216	0.002	0.315	0.193	0.046	0.015	0.508	#21	0.507	Not Required	Pass
301	0.238	0.714	0.040	0.025	0.002	0.827	#13	0.773	Not Required	Pass
302	0.005	0.520	0.108	0.113	0.018	0.612	#21	0.036	Not Required	Pass
303	0.007	0.775	0.041	0.076	0.006	0.801	#21	0.046	Not Required	Pass
304	0.007	0.759	0.157	0.076	0.035	0.872	#21	0.082	Not Required	Pass
305	0.007	0.481	0.150	0.077	0.040	0.527	#21	0.076	Not Required	Pass
306	0.008	0.893	0.047	0.090	0.004	0.944	#21	0.046	Not Required	Pass
307	0.008	0.554	0.190	0.088	0.049	0.602	#21	0.076	Not Required	Pass
308	0.000	0.164	0.314	0.046	0.015	0.478	#21	Not Required	Not Required	Pass
309	0.020	0.107	0.052	0.003	0.001	0.170	#21	0.206	Not Required	Pass

310	0.008	0.863	0.177	0.086	0.036	0.957	#21	0.082	Not Required	Warn
311	0.000	0.169	0.314	0.047	0.015	0.482	#21	Not Required	Not Required	Pass
312	0.004	0.644	0.121	0.131	0.020	0.749	#21	0.036	Not Required	Pass
313	0.009	0.411	0.523	0.065	0.020	0.924	#21	0.204	Not Required	Pass
314	0.009	0.410	0.523	0.065	0.020	0.912	#21	0.306	Not Required	Pass
315	0.002	0.188	0.192	0.052	0.016	0.368	#21	0.338	Not Required	Pass
316	0.002	0.189	0.194	0.051	0.016	0.365	#21	0.507	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	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
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided



REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 7</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>11.483</td> <td>18.094</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.604</td> <td>-2.687</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.024</td> <td>0.036</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.142</td> <td>0.218</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>31.989</td> <td>56.353</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> 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)	11.483	18.094	$V_x$ (kip)	-1.604	-2.687	$V_z$ (kip)	0.024	0.036	$M_x$ (kipft)	0.142	0.218	$M_z$ (kipft)	31.989	56.353	
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																									
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$V_z$ (kip)	0.024	0.036																										
$M_x$ (kipft)	0.142	0.218																										
$M_z$ (kipft)	31.989	56.353																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.604 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25541 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(31.989 \text{ kipft}) + ((-1.604 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0938 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.727 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.142 \text{ kipft}) + ((0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.022611 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2811 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.727 \text{ ft}), (1.2811 \text{ ft})]$$

$$L_{e,req} = 6.727 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.727 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.961$$

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

**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

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

$$q = \frac{(11.483 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.71768 \text{ kip/ft}^2$$

$$q = 0.11109 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.71769 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35884$$

Status: **PASS**  
Ratio: **0.360**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.75$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.25541 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 5.0938 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0938 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.0938 \text{ kipft/ft})) + (4 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7773 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.0938 \text{ kipft/ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (5.0938 \text{ kipft/ft})) + (2 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.29466 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (5.0938 \text{ kipft/ft})) + ((-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 1.0285 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7773 \text{ ft})}{2}$$

$$p_a = 0.3583 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29466 \text{ kip/ft}^2)}{(0.3583 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82239$$

$p_a$  - Allowable lateral soil pressure at depth  $L_e$ ,

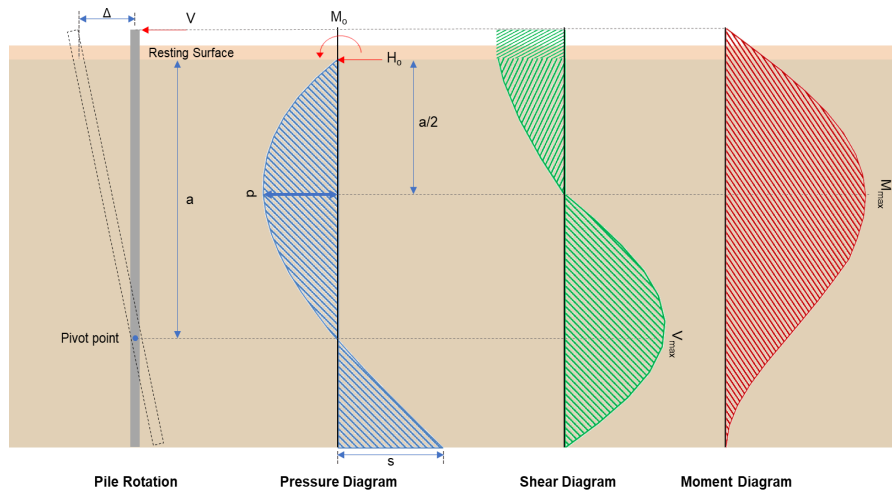
Status: **PASS**  
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$ $p_s = 1.05 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.0285 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97955$	<p>Status: <b>PASS</b> Ratio: <b>0.980</b></p>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.0038217 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.022611 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.022611 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.022611 \text{ kipft/ft})) + (4 \times (0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))}$ $a = 4.9239 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.022611 \text{ kipft/ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.022611 \text{ kipft/ft})) + (2 \times (0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))]}$ $p = 0.0036757 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.022611 \text{ kipft/ft})) + ((0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$ $s = 0.0088132 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.9239 \text{ ft})}{2}$ $p_a = 0.36929 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0036757 \text{ kip/ft}^2)}{(0.36929 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0099534$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$ $p_s = 1.05 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>

$$\text{Ratio} = \frac{(0.0088132 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0083935$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.687 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42787 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(56.353 \text{ kipft}) + ((-2.687 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.9734 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.9734 \text{ kipft/ft})}{(-0.42787 \text{ kip/ft})}$$

$$E = 20.972 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.9734 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.42787 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (8.9734 \text{ kipft/ft})) + (4 \times (-0.42787 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7728 \text{ ft}$$

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

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

$$V_{max} = ((-0.42787 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.7728 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.7728 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.211 \text{ kip}$$

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

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

$$M_{max} = ((-0.42787 \text{ kip/ft}) \times (48 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(20.972 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7728 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.7728 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.7728 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 34.643 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.036 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0057325 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.218 \text{ kipft}) + ((0.036 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.034713 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.034713 \text{ kipft/ft})}{(0.0057325 \text{ kip/ft})}$$

$$E = 6.0556 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.034713 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.0057325 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.034713 \text{ kipft/ft})) + (4 \times (0.0057325 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9206 \text{ ft}$$

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

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

$$V_{max} = ((0.0057325 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6.0556 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.9206 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (6.0556 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.9206 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.050266 \text{ kip}$$

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

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

$$M_{max} = ((0.0057325 \text{ kip/ft}) \times (48 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(6.0556 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9206 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6.0556 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.9206 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (6.0556 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.9206 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.1615 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.8$  - Alpha factor for axial strength,  
 $A_g = 2304 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.004 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.995 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.995 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1

$s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(18.094 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0067637$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

22.5.2.2

$b_w = 48 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,max}$  - Max shear strength of concrete

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.094 \text{ kip} \rightarrow 18094 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(18094 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.2

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,b}$  - Shear strength of concrete (b)

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

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

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

$V_c$  - Governing shear strength of concrete

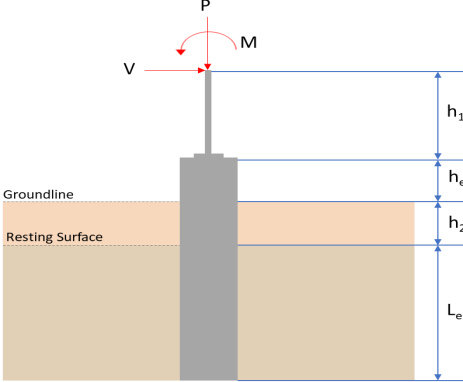
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

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

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

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - 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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.9 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.66 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 10.211 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.211 \text{ kip})}{(111.66 \text{ kip})}$ $\text{Ratio} = 0.091442$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.050266 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.050266 \text{ kip})}{(111.66 \text{ kip})}$ $\text{Ratio} = 0.00045015$	<p>Status: <b>PASS</b> Ratio: <b>0.090</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LFRD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 34.643 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(34.643 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.13879$	<p>Status: <b>PASS</b>  Ratio: <b>0.140</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.1615 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.1615 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00064704$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 7</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1192 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>11.483</td> <td>18.094</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.604</td> <td>-2.687</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.024</td> <td>-0.036</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.142</td> <td>-0.217</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>31.989</td> <td>56.353</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> 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)	11.483	18.094	$V_x$ (kip)	-1.604	-2.687	$V_z$ (kip)	-0.024	-0.036	$M_x$ (kipft)	-0.142	-0.217	$M_z$ (kipft)	31.989	56.353	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.604 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25541 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(31.989 \text{ kipft}) + ((-1.604 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0938 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.727 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.142 \text{ kipft}) + ((-0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.022611 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1558 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.727 \text{ ft}), (1.1558 \text{ ft})]$$

$$L_{e,req} = 6.727 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.727 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.961$$

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

**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

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

$$q = \frac{(11.483 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.71768 \text{ kip/ft}^2$$

$$q = 0.11109 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.71769 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35884$$

Status: **PASS**  
Ratio: **0.360**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.75$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.25541 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 5.0938 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0938 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.0938 \text{ kipft/ft})) + (4 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7773 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.0938 \text{ kipft/ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (5.0938 \text{ kipft/ft})) + (2 \times (-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.29466 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (5.0938 \text{ kipft/ft})) + ((-0.25541 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 1.0285 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7773 \text{ ft})}{2}$$

$$p_a = 0.3583 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29466 \text{ kip/ft}^2)}{(0.3583 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82239$$

$p_a$  - Allowable lateral soil pressure at depth  $L_e$ ,

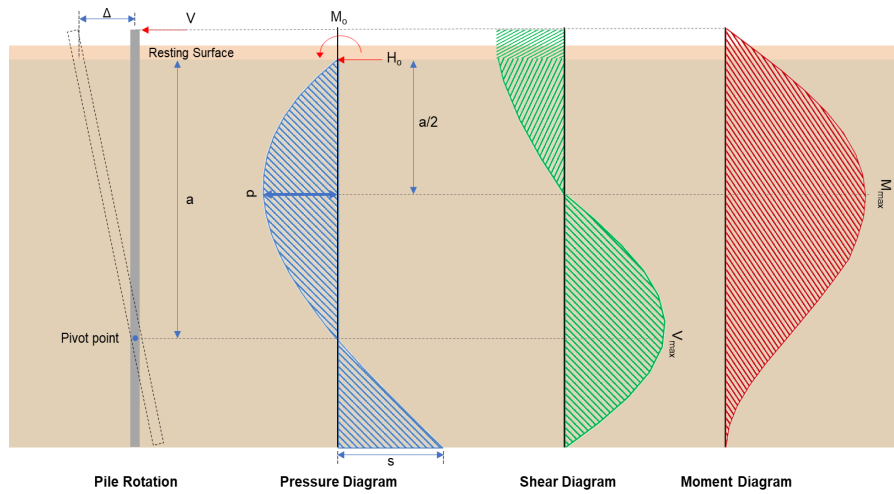
Status: **PASS**  
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$ $p_s = 1.05 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.0285 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97955$	Status: <b>PASS</b> Ratio: <b>0.980</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.0038217 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.022611 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.022611 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.022611 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))}$ $a = 4.9239 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.022611 \text{ kipft/ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.022611 \text{ kipft/ft})) + (2 \times (-0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))]}$ $p = 0.00011092 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.022611 \text{ kipft/ft})) + ((-0.0038217 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$ $s = 0.0022618 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.9239 \text{ ft})}{2}$ $p_a = 0.36929 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.00011092 \text{ kip/ft}^2)}{(0.36929 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.00030037$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$ $p_s = 1.05 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.000</b>

$$Ratio = \frac{(0.0022618 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.0021541$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.687 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42787 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(56.353 \text{ kipft}) + ((-2.687 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.9734 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.9734 \text{ kipft/ft})}{(-0.42787 \text{ kip/ft})}$$

$$E = 20.972 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.9734 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.42787 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (8.9734 \text{ kipft/ft})) + (4 \times (-0.42787 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7728 \text{ ft}$$

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

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

$$V_{max} = ((-0.42787 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.7728 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.7728 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.211 \text{ kip}$$

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

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

$$M_{max} = ((-0.42787 \text{ kip/ft}) \times (48 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(20.972 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7728 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.7728 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (20.972 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.7728 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.643 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.036 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0057325 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.217 \text{ kipft}) + ((-0.036 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.034554 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.034554 \text{ kipft/ft})}{(-0.0057325 \text{ kip/ft})}$$

$$E = 6.0278 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.034554 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.0057325 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.034554 \text{ kipft/ft})) + (4 \times (-0.0057325 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9212 \text{ ft}$$

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

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

$$V_{max} = ((-0.0057325 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6.0278 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.9212 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (6.0278 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.9212 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.050106 \text{ kip}$$

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

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

$$M_{max} = ((-0.0057325 \text{ kip/ft}) \times (48 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(6.0278 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9212 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6.0278 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.9212 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (6.0278 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.9212 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.16094 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.8$  - Alpha factor for axial strength,  
 $A_g = 2304 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.094 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.995 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.995 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(18.094 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0067637$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

22.5.2.2

$b_w = 48 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,max}$  - Max shear strength of concrete

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.094 \text{ kip} \rightarrow 18094 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(18094 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.2

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,b}$  - Shear strength of concrete (b)

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

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

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

$V_c$  - Governing shear strength of concrete

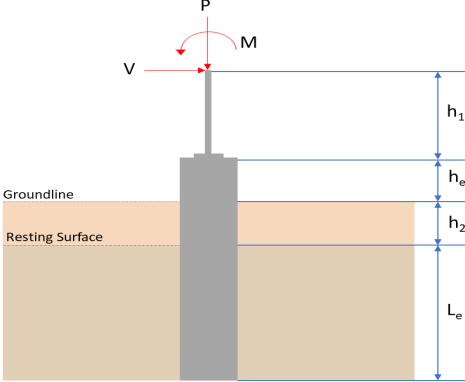
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

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

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

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - 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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.9 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.66 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 10.211 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.211 \text{ kip})}{(111.66 \text{ kip})}$ $\text{Ratio} = 0.091442$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.050106 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.050106 \text{ kip})}{(111.66 \text{ kip})}$ $\text{Ratio} = 0.00044872$	<p>Status: <b>PASS</b> Ratio: <b>0.090</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 34.643 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(34.643 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.13879$	<p>Status: <b>PASS</b>  Ratio: <b>0.140</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.16094 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.16094 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0006448$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 7.25</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_n</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>12.025</td> <td>18.945</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.679</td> <td>-2.807</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.172</td> <td>-0.277</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-1.006</td> <td>-1.631</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>33.397</td> <td>59.441</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_n$ ) (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)	12.025	18.945	$V_x$ (kip)	-1.679	-2.807	$V_z$ (kip)	-0.172	-0.277	$M_x$ (kipft)	-1.006	-1.631	$M_z$ (kipft)	33.397	59.441	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.679 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26736 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(33.397 \text{ kipft}) + ((-1.679 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.318 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.8124 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.172 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.027389 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.006 \text{ kipft}) + ((-0.172 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16019 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.1069 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.8124 \text{ ft}), (2.1069 \text{ ft})]$$

$$L_{e,req} = 6.812 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.812 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.9396$$

Status: **PASS**  
Ratio: **0.940**

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(12.025 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.75156 \text{ kip/ft}^2$$

$$q = 0.75156 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.75156 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.37578$$

Status: **PASS**  
Ratio: **0.380**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.26736 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 5.318 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.318 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.318 \text{ kipft/ft})) + (4 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9514 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (5.318 \text{ kipft/ft})) + (3 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 [(3 \times (5.318 \text{ kipft/ft})) + (2 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.28227 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 [(2 \times (5.318 \text{ kipft/ft})) + ((-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.99284 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9514 \text{ ft})}{2}$$

$$p_a = 0.37136 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28227 \text{ kip/ft}^2)}{(0.37136 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.7601$$

$p_a$  - Allowable lateral soil pressure at depth  $L_e$ ,

Status: **PASS**  
Ratio: **0.760**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.99284 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91295$$

Status: **PASS**  
Ratio: **0.910**

**Considering z-direction:**

$H_o = -0.027389 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.16019 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16019 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.027389 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.16019 \text{ kipft/ft})) + (4 \times (-0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1067 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.16019 \text{ kipft/ft})) + (3 \times (-0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 [(3 \times (0.16019 \text{ kipft/ft})) + (2 \times (-0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.00034727 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 [(2 \times (0.16019 \text{ kipft/ft})) + ((-0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.013905 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1067 \text{ ft})}{2}$$

$$p_a = 0.383 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00034727 \text{ kip/ft}^2)}{(0.383 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0009067$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

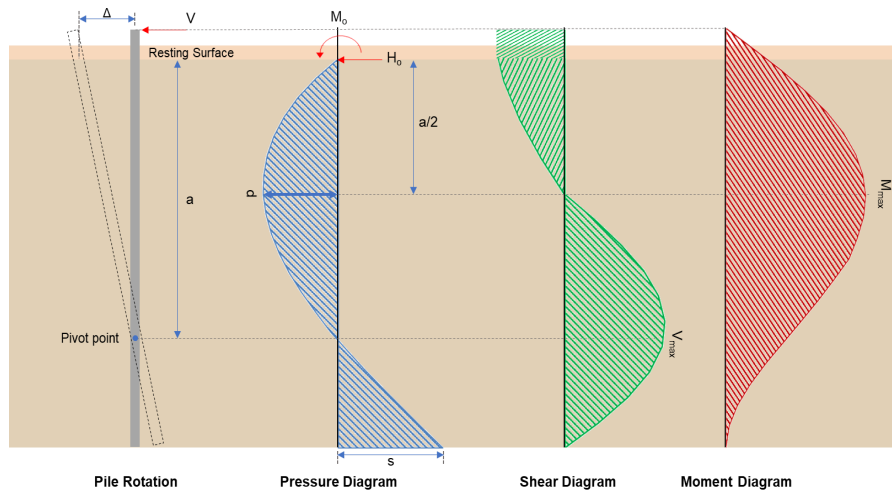
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.013905 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.012786$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.807 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44697 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(59.441 \text{ kipft}) + ((-2.807 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.4651 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.4651 \text{ kipft/ft})}{(-0.44697 \text{ kip/ft})}$$

$$E = 21.176 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.4651 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.44697 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (9.4651 \text{ kipft/ft})) + (4 \times (-0.44697 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9456 \text{ ft}$$

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

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

$$V_{max} = ((-0.44697 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.9456 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.9456 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.428 \text{ kip}$$

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

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

$$M_{max} = ((-0.44697 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(21.176 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9456 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.9456 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.9456 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 36.618 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.277 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044108 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.631 \text{ kipft}) + ((-0.277 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.25971 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.25971 \text{ kipft/ft})}{(-0.044108 \text{ kip/ft})}$$

$$E = 5.8881 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.25971 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.044108 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.25971 \text{ kipft/ft})) + (4 \times (-0.044108 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1057 \text{ ft}$$

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

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

$$V_{max} = ((-0.044108 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.1057 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.1057 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.37033 \text{ kip}$$

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

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

$$M_{max} = ((-0.044108 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(5.8881 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1057 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.1057 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.1057 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2275 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.8$  - Alpha factor for axial strength,  
 $A_g = 2304 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.945 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.967 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.967 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(18.945 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0070818$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

22.5.2.2

$b_w = 48 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,max}$  - Max shear strength of concrete

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.945 \text{ kip} \rightarrow 18945 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(18945 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121.01 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,b}$  - Shear strength of concrete (b)

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

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

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

$V_c$  - Governing shear strength of concrete

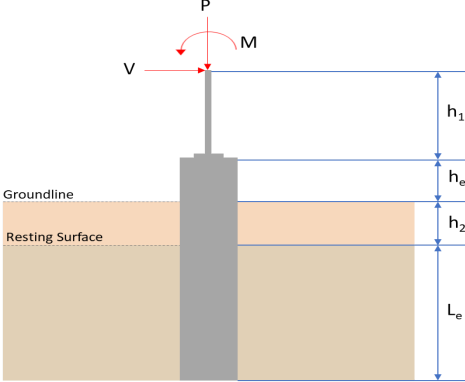
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (121.01 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121.01 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - 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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((121.01 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.74 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 10.428 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.428 \text{ kip})}{(111.74 \text{ kip})}$ $\text{Ratio} = 0.093326$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.37033 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.37033 \text{ kip})}{(111.74 \text{ kip})}$ $\text{Ratio} = 0.0033142$	<p>Status: <b>PASS</b> Ratio: <b>0.090</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 36.618 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(36.618 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.14671$	<p>Status: <b>PASS</b>  Ratio: <b>0.150</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 1.2275 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.2275 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0049181$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 7.25</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>12.025</td> <td>18.945</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.679</td> <td>-2.807</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.172</td> <td>0.277</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>1.006</td> <td>1.631</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>33.397</td> <td>59.442</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> 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)	12.025	18.945	$V_x$ (kip)	-1.679	-2.807	$V_z$ (kip)	0.172	0.277	$M_x$ (kipft)	1.006	1.631	$M_z$ (kipft)	33.397	59.442	
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)	12.025	18.945																										
$V_x$ (kip)	-1.679	-2.807																										
$V_z$ (kip)	0.172	0.277																										
$M_x$ (kipft)	1.006	1.631																										
$M_z$ (kipft)	33.397	59.442																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.679 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26736 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(33.397 \text{ kipft}) + ((-1.679 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.318 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.8124 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.172 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.027389 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.006 \text{ kipft}) + ((0.172 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16019 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.5735 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.8124 \text{ ft}), (2.5735 \text{ ft})]$$

$$L_{e,req} = 6.812 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.812 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.9396$$

Status: **PASS**  
Ratio: **0.940**

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(12.025 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.75156 \text{ kip/ft}^2$$

$$q = 0.75156 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.75156 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.37578$$

Status: **PASS**  
Ratio: **0.380**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.26736 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 5.318 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.318 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.318 \text{ kipft/ft})) + (4 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9514 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (5.318 \text{ kipft/ft})) + (3 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 [(3 \times (5.318 \text{ kipft/ft})) + (2 \times (-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.28227 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 [(2 \times (5.318 \text{ kipft/ft})) + ((-0.26736 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.99284 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9514 \text{ ft})}{2}$$

$$p_a = 0.37136 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28227 \text{ kip/ft}^2)}{(0.37136 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.7601$$

$p_a$  - Allowable lateral soil pressure at depth  $L_e$ ,

Status: **PASS**  
Ratio: **0.760**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.99284 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91295$$

Status: **PASS**  
Ratio: **0.910**

**Considering z-direction:**

$H_o = 0.027389 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.16019 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16019 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.027389 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.16019 \text{ kipft/ft})) + (4 \times (0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1067 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.16019 \text{ kipft/ft})) + (3 \times (0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 [(3 \times (0.16019 \text{ kipft/ft})) + (2 \times (0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.024854 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 [(2 \times (0.16019 \text{ kipft/ft})) + ((0.027389 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.059238 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1067 \text{ ft})}{2}$$

$$p_a = 0.383 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.024854 \text{ kip/ft}^2)}{(0.383 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.064893$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

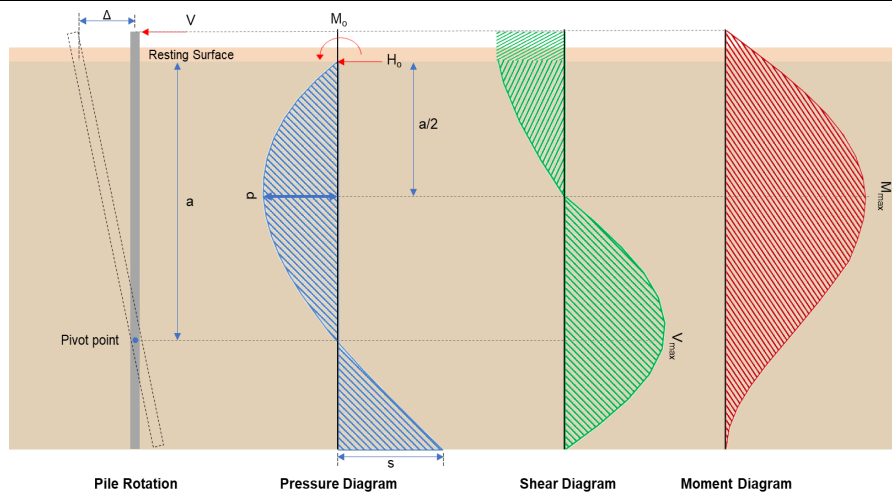
$$\text{Ratio} = \frac{s}{p_s}$$

Status: **PASS**  
Ratio: **0.060**

$$Ratio = \frac{(0.059238 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.054472$$

Status: **PASS**  
Ratio: **0.050**



#### Shear force and Bending moment (x-direction, LRF)

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.807 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44697 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(59.442 \text{ kipft}) + ((-2.807 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.4653 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.4653 \text{ kipft/ft})}{(-0.44697 \text{ kip/ft})}$$

$$E = 21.176 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.4653 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.44697 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (9.4653 \text{ kipft/ft})) + (4 \times (-0.44697 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9456 \text{ ft}$$

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

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

$$V_{max} = ((-0.44697 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.9456 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.9456 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.428 \text{ kip}$$

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

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

$$M_{max} = ((-0.44697 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(21.176 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9456 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.9456 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (21.176 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.9456 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 36.618 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.277 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.044108 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.631 \text{ kipft}) + ((0.277 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.25971 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.25971 \text{ kipft/ft})}{(0.044108 \text{ kip/ft})}$$

$$E = 5.8881 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.25971 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.044108 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.25971 \text{ kipft/ft})) + (4 \times (0.044108 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1057 \text{ ft}$$

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

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

$$V_{max} = ((0.044108 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.1057 \text{ ft})}{(7.25 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.1057 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.37033 \text{ kip}$$

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

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

$$M_{max} = ((0.044108 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(5.8881 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1057 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.1057 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (5.8881 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.1057 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 1.2275 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.8$  - Alpha factor for axial strength,  
 $A_g = 2304 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.945 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.967 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.967 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1

$s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(18.945 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0070818$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

22.5.2.2

$b_w = 48 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,max}$  - Max shear strength of concrete

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.945 \text{ kip} \rightarrow 18945 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(18945 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121.01 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  
 $V_{c,b}$  - Shear strength of concrete (b)

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (121.01 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121.01 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - 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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((121.01 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.74 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 10.428 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.428 \text{ kip})}{(111.74 \text{ kip})}$ $\text{Ratio} = 0.093328$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.37033 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.37033 \text{ kip})}{(111.74 \text{ kip})}$ $\text{Ratio} = 0.0033142$	<p>Status: <b>PASS</b> Ratio: <b>0.090</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LFRD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 36.618 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(36.618 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.14671$	<p>Status: <b>PASS</b>  Ratio: <b>0.150</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 1.2275 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.2275 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0049181$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>