Spring 2017

CIVE 7302 Advanced Foundation Engineering

Assignment #6

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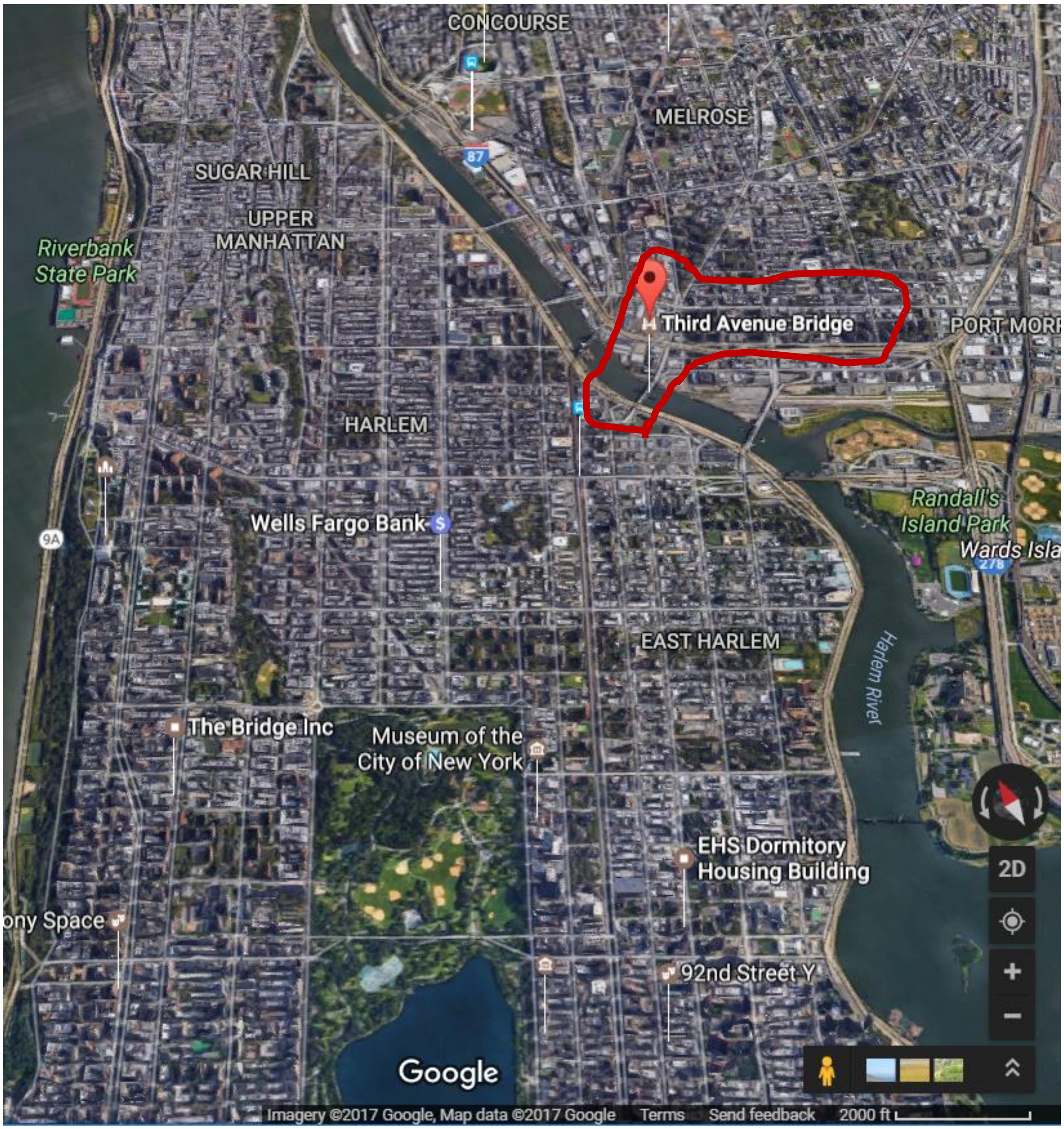
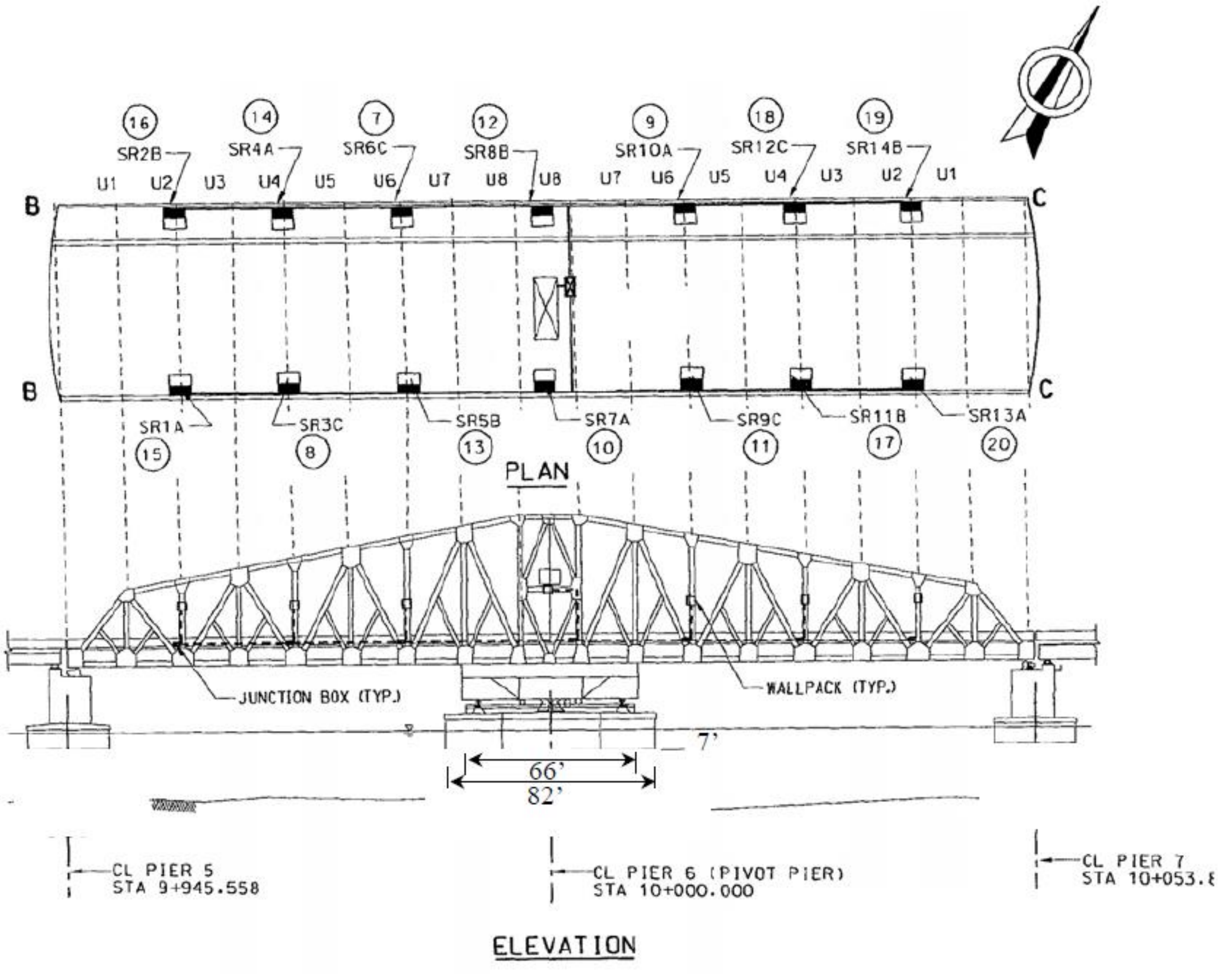
This report explains pivot pier foundation design for the construction of Third Avenue Bridge in New York City, New York. The bridge will connect the northeast Manhattan and Bronx in New York City. This bridge will carry most of the traffic in between north from the Bowery and Fourth Street in Manhattan to Webster Avenue in the Bronx as shown the figure below. The average daily traffic is expected to be 60000 (New York City DOT, 2014)



The planned bridge is going to have moveable span. The truss swing span superstructure is going to be 350 ft long and 90 ft wide and it will be operated from the control base located at the center of the span as well. The design plan of the truss swing span superstructure is shown as below:



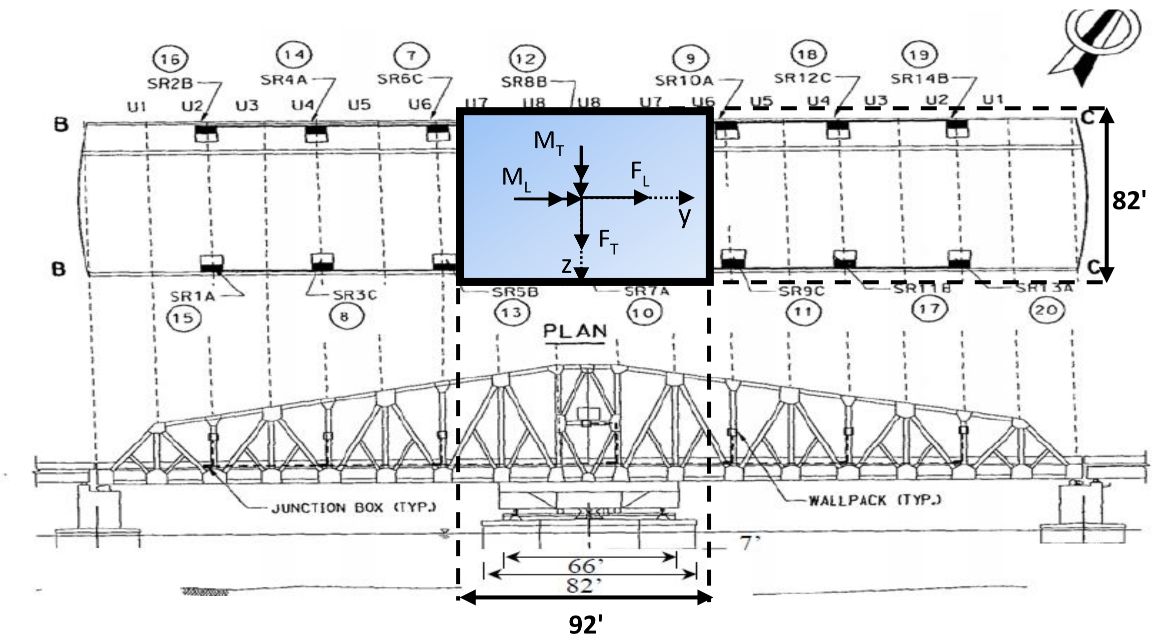
In order to come up with a pivot pier foundation design for this swing bridge, loading conditions due to the structure, traffic and seismic, geotechnical investigation in terms of soil profile and constructability have to be taken into consideration all together.

Loading Conditions

Loadings and moments at the pile cap under the pivot pier are given as below:

	Pile Cap Base Forces, kip			Pile Cap Base Moments, kip-ft		
	F_V	F_T	F_L	M_V	M_T	M_L
Dead Load	16519	0	0	0	0	0
L + T + V	4206	4537	7872	9929	116366	106276

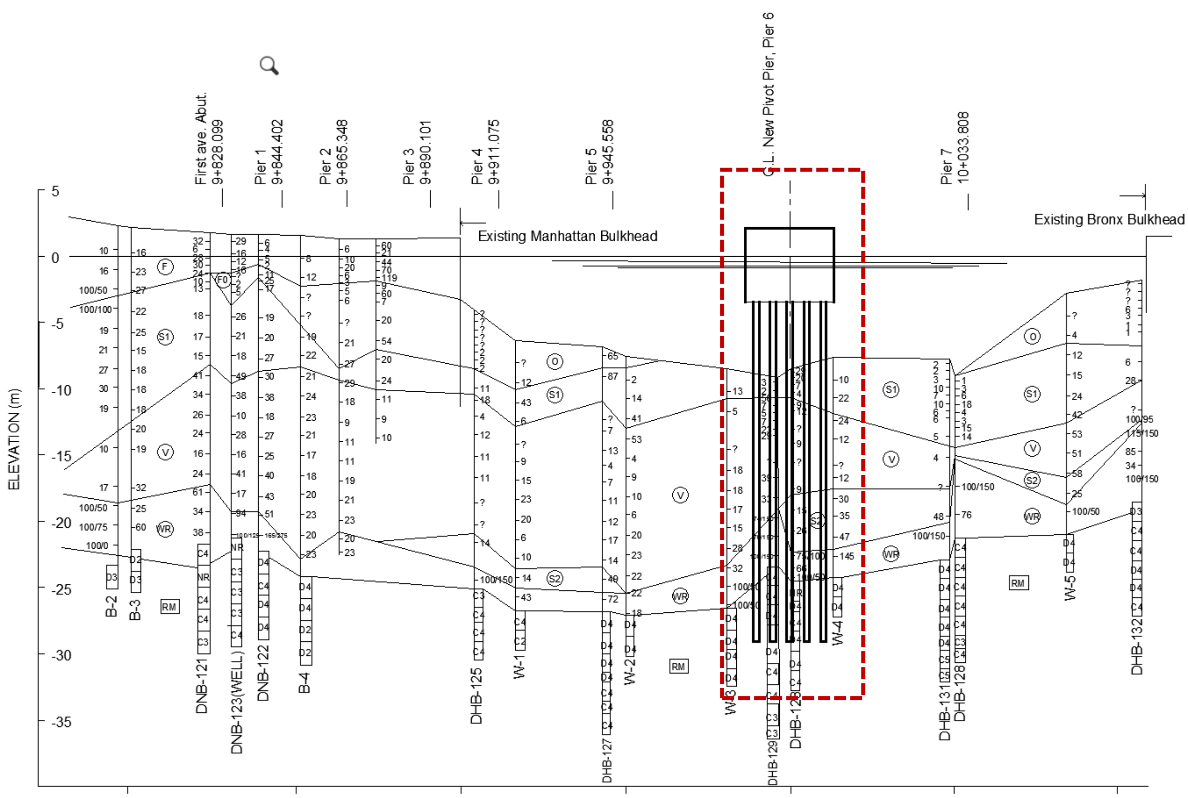
There are 3 forces and moments acting on the pile cap which are longitudinal, transverse and vertical forces and moments. Dead load comes from the weight of the superstructure and L+T+V (longitudinal, transverse and vertical) comes from the live load, wind and seismic load. The figure shown below indicates the directions of the loadings and moments on the design plan.



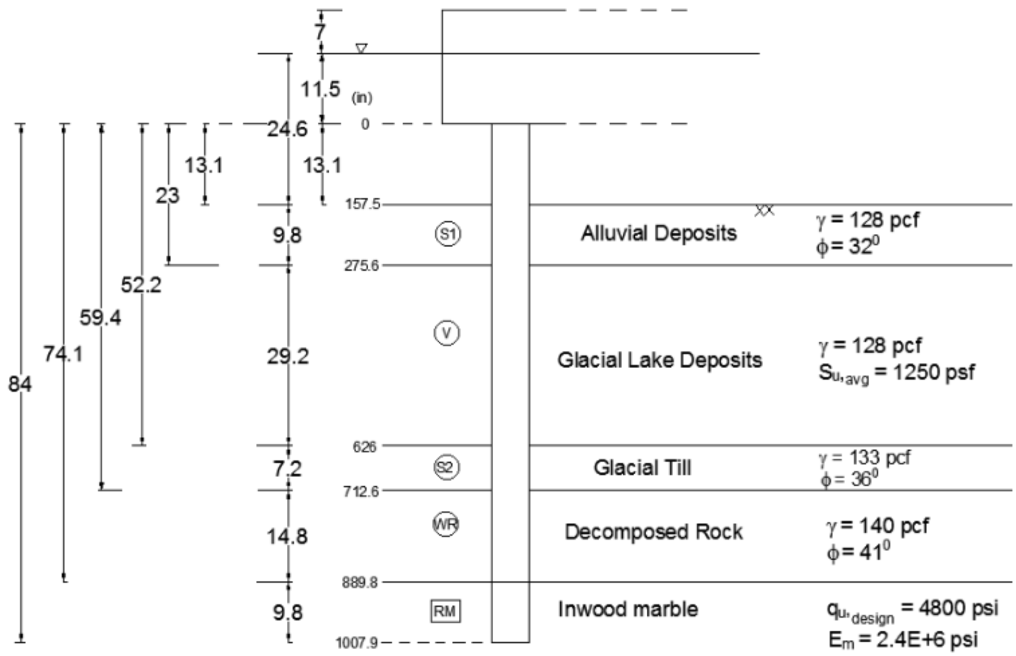
Since this bridge is a swing bridge, all the forces and moments will be carried by the center pivot pier itself when the bridge rotates; therefore, the design of the pivot pier foundation is very significant.

Soil Profile

The soil profile at where center pivot pier exits will be investigated. The surface ground starts at an elevation of 25 ft (7.5 m). Soil profile starts with 10 ft (3 m) of alluvial deposits and continues with 29 ft (9 m) glacial lake deposits. There is a 7 ft (2.2 m) glacial till soil layer underlying the glacial lake deposits. Before the bedrock, there is about 15 ft (4.5 m) of decomposed rock layer exists under the glacial till. The bedrock layer starts at a depth of 85.6 ft (26.1 m) and it is classified as Inwood marble. The general soil profile under the center pivot pier is shown below:



Only the soil profile under the center pier pivot is considered during calculations and modeling the foundation as well. Therefore, information from the borehole 128 and 129 (DHB-128 and DHB-129) are used. Generalized 1-D soil profile with corresponding unit weights and other soil information are shown as below:



*All the dimensions in the figures are in feets.

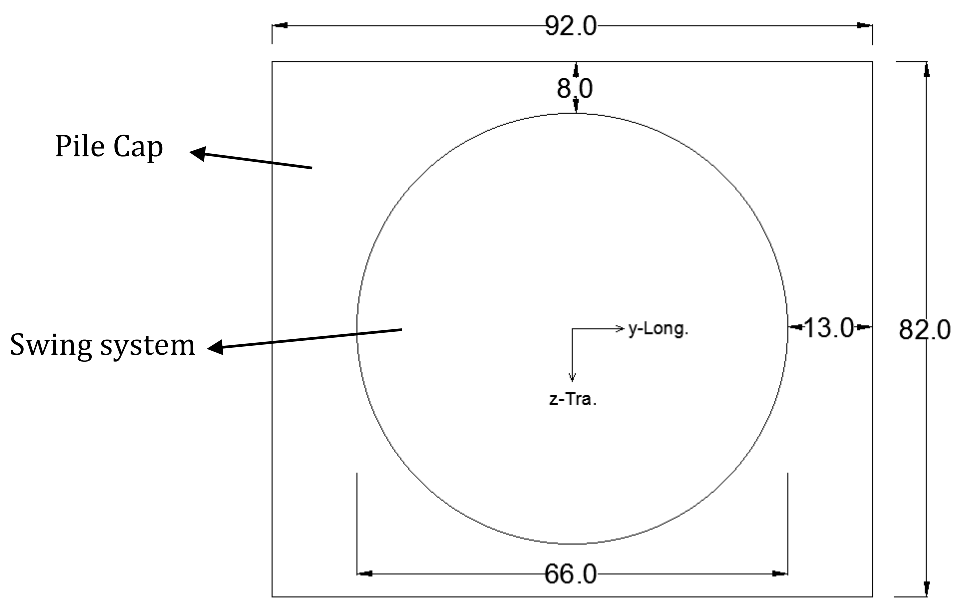
From the SPT numbers from DBH-128 and DBH-129 for alluvial deposits and glacial tills, friction angles are estimated by using Peck, Schmertmann and Hatanaka method. The friction angles obtained from these methods (shown the table below) are close the friction angles given from the problem. Therefore, the friction angles given the problem are used during the calculations.

Layer(s)	Elevation (m)	Depth (ft)	DHB-129	σ'$_v$ (psf)	ϕ		
					Peck	Schmertmann	Hatanaka
Alluvial Deposits	9.5	1.2	3	77	28	31	37
	10.1	3.2	2	211	28	27	31

Layer(s)	Elevation (m)	Depth	DHB-128	σ'$_v$ (psf)	ϕ		
					Peck	Schmertmann	Hatanaka
Alluvial Deposits	9.7	4.0	7	260	29	38	40
	10.3	6.0	4	392	28	32	33
Glacial Till	19.3	35.4	15	2352	31	37	37
	20.7	40.2	26	2690	35	41	41

Constructability and Design Alternatives

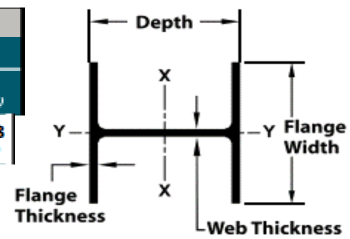
Considering loading condition and soil profile, loadings and moments due to dead load and seismic condition are very huge so that special attention should be put for selecting foundation type. Although constructability of the piles is more difficult than other foundation types, pile foundation should be used to carry all loadings and moments. Pile foundation is also important for swing bridge because when the bridge rotates, all the loadings and moments are carried by the center pier so these forces should be transferred from the structure to the stiff layer which is bedrock. There are two different pile foundation alternatives considered which are 6 ft diameter pipe piles filled with concrete and H piles. Both have their own advantages and disadvantages in terms of design and constructability. These pros and cons will be mentioned later in this report. Pile cap for both alternatives are designed so that there will be 7 ft (2.1 m) above the water level and 11.5 ft (3.5 m) below the water level. The pile cap does not sit on the ground surface; it will be on the water surface to prevent usage of excess amount of concrete for the pile cap. However, pile cap should have enough thickness to carry bending moments due to the structure; therefore, 18.5 ft (5.6 m) thickness is selected. The pile cap dimensions are 92 ft (28 m) in length and 82 (25 m) ft in width. These dimensions are selected so that there is enough space (at least 8 ft) at sides for future maintenance of the rail system.



1.) **<u>Steel H Piles</u>**

First design alternative is steel H pile. One of the advantage of using H piles is that H piles is one of the best way to transfer loads from the structure to the bedrock. Since loadings and moments are very huge, a big H pile is selected to use which is HP 18" x 181. The table below indicates the geometrical information about the HP 18" x 181.

	WEIGHT	AREA	DEPTH	FLANGE WIDTH	FLANGE	WEB	COATING AREA	AXIS X-X			AXIS Y-Y		
								I	S	r	I	S	r
	lb/ft (kg/m)	in² (cm²)	in (mm)	in (mm)	in (mm)	in (mm)	ft²/ft (m²/m)	in⁴ (cm⁴)	in³ (cm³)	in (cm)	in⁴ (cm⁴)	in³ (cm³)	in (cm)
HP 18" HP 460 mm	181 269	53.2 343	18.00 457	18.000 457	1.000 25.4	1.000 25.4	8.66 2.64	3017 125579	335 5493	7.53 19.1	974 40545	108.1 1774	4.28 10.9



Before analyzing the piles in terms of horizontal deflection by using GROUP, axial capacity calculations are done by hand and how many piles are needed to carry axial loading.

Pile Properties:

$\text{Area of Steel, } A_s\ (\text{in}^2) = 53.2\ \text{in}^2$

$$\text{Equivalent Diameter (in)} = \sqrt{\frac{(18 \text{ x } 18)\text{x}4}{\pi}} = 20.3 \text{ in}$$

$\text{Perimeter (not plugged)} = 4\text{x}18 + 2\text{x}18 - 2\text{x}1 = 106 \text{ in}$

$\text{Perimeter (plugged)} = 2\text{x}18 + 2\text{x}18 = 72 \text{ in}$

$\text{Moment of Inertia, strong axis, } I_x = 3017\ \text{in}^4$

$\text{Moment of Inertia, weak axis, } I_y = 974\ \text{in}^4$

$\text{Modulus of Elasticity, } E_s = 29000000\ \text{psi}$ from http://www.amesweb.info/Materials/Modulus-of-Elasticity-Metals.aspx

Ultimate Unit Tip & Side Resistance

- **Tip Resistance**

$q_p = s_u \; x \; (1 + N_\phi) \; if \; the \; tip \; of \; the \; pile \; is \; at \; the \; bedrock \; (Inwood \; marble).$

$\phi = 25^0 \; from \; the \; table \; shown \; below \; for \; marble$

Table 11.12 Typical Values of Angle of Friction ϕ' of Rocks

Type of rock	Angle of friction, ϕ' (deg)
Sandstone	27–45
Limestone	30–40
Shale	10–20
Granite	40–50
Marble	25–30

$s_u = \frac{q_u}{2}$ where q_u: unconfined compressive strength $\rightarrow s_u = \frac{9600}{2} = 4800$ psi

$N_\phi = \tan\left(45 + \frac{\phi}{2}\right) = \tan\left(45 + \frac{25}{2}\right) = 1.57$

$Q_p = q_p \; x \; A_s = 4800 \; x \; (1 + 1.57) \; x \; 53 = 656195 \text{ lb} \rightarrow \mathbf{Q_p = 656 \; k}$

- **Side Resistance**

$\rightarrow L' = 15D = 15 \; x \frac{20.3}{12} = 25 \text{ ft} \rightarrow$ critical depth

$\rightarrow$ Driven pile, 1.2K is used $\rightarrow K = K_0 = 1 - \sin\phi$

$\rightarrow$ Both plugged $(\delta = \phi)$and not plugged $(\delta = 0.4\phi)$ are considered.

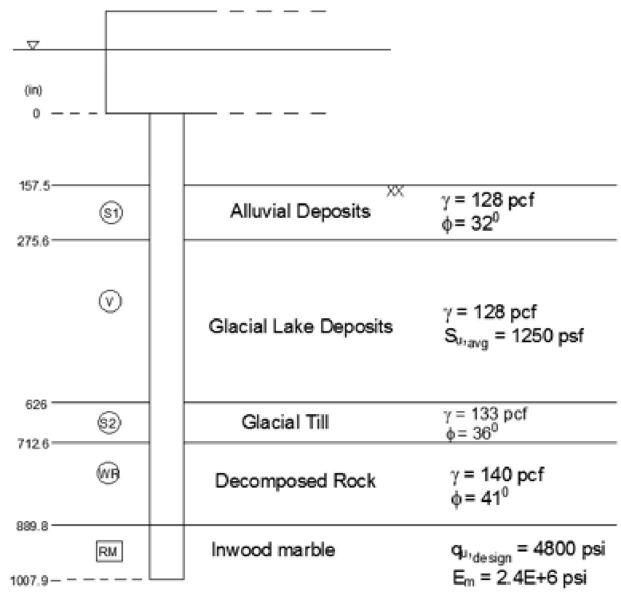
$\rightarrow$ perimeter (unplugged) = 106 in, & periemter(plugged) = 72 in

$\rightarrow$ There are 2 s_u given in the problem for glacial lake deposits which are 1250 and 2500 psf. To calculate f_{av} for this layer, α method (Table 11.10 in the lecture notes) is used. The table below indicates f values for 2 s_u values.

S_u (psf)	1250	2500
S_u / p_a	0.6	1.3
α	0.62	0.41
f = Su x α	775.0	1025.0

Note that f = 800 psf = 5.56 psi is used for the rest of side friction calculations.

The figure and table below indicates the soil profile with depth in inches and side friction calculation, respectively.



K = 1.2K_0

Layer(s)		Adjusted Depth (m)	Adjusted Depth (ft)	γ_b (pci)	σ'_{v0} (psi)	ϕ	σ'_{h0} (psi)
Footing Bottom Elevation		0	0.0	0.0000	0.00	0	
Alluvial Deposits	**start**	4	13.1	0.0380	0.00	32	0.00
	end	7	23.0	0.0380	4.48	32	2.53
Glacial Lake Deposits	**start**	7	23.0	0.0380	4.48	0	0.00
	critical depth	7.62	25.0	0.0380	5.41	0	0.00
	end	15.9	52.2	0.0380	17.79	0	0.00
Glacial Till	**start**	15.9	52.2	0.0409	17.79	36	8.80
	end	18.1	59.4	0.0409	21.32	36	10.55
Decomposed Rock	**start**	18.1	59.4	0.0449	21.32	41	8.80
	end	22.6	74.1	0.0449	29.28	41	12.08

					tan(ϕ)	tan(0.4ϕ)		plugged	not plugged
Layer(s)		**Adjusted Depth (ft)**	**σ'_{h0} (psi)**	**ϕ**	**f_s**	**f_s**	**Length (in)**	**Q_s (lb)**	**Q_s (lb)**
Alluvial Deposits	**start**	13.1	0.00	32	0.79	0.29	118	6720	3597
	end	23.0	2.53						
Glacial Lake Deposits	**start**	23.0	0.00	0	5.56	5.56	350	103561	103561
	critical depth	25.0	0.00						
	critical depth	25.0	0.00						
	end	52.2	0.00						
Glacial Till	**start**	52.2	8.80	36	1.84	0.65	87	11460	5962
	end	59.4	10.55						
Decomposed Rock	**start**	59.4	8.80	41	2.20	0.74	177	28046	13980
	end	74.1	12.08						
							Sum (lb)	149787	127100

Sum (k)	150	127

Note that 135k of skin friction is considered to calculate ultimate resistance:

$$\mathrm{Q_{ult}} = 656 + 135 = 791 \text{ k}$$

$$\mathrm{F_v}\ (\text{Dead} + \text{V}) = 20725 \text{ k}$$

$$\mathrm{Q_{all}} = \frac{\mathrm{Q_{ult}}}{\mathrm{F.S}} = \frac{791}{2.5} = 316 \text{ k}$$

$$\#\ \text{of piles} = \frac{20725}{316} \rightarrow \#\ \textbf{of piles} = \mathbf{65}$$

The alignment of the H piles are selected by considering transverse and longitudinal force. The loadings at the pile cap plays significant role in determining the alignment of the H-piles.

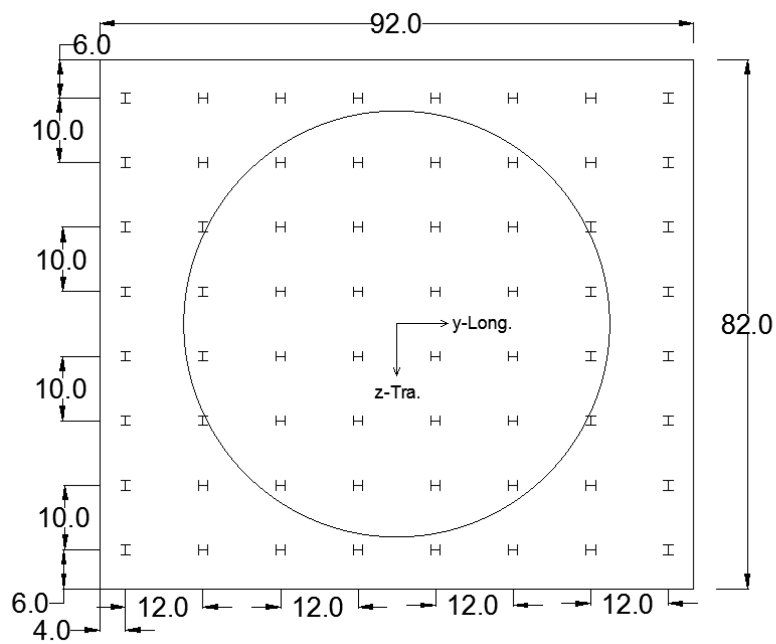
$$F_T = 4537\ k, F_L = 7872\ k$$

$By\ linear\ relationship$:

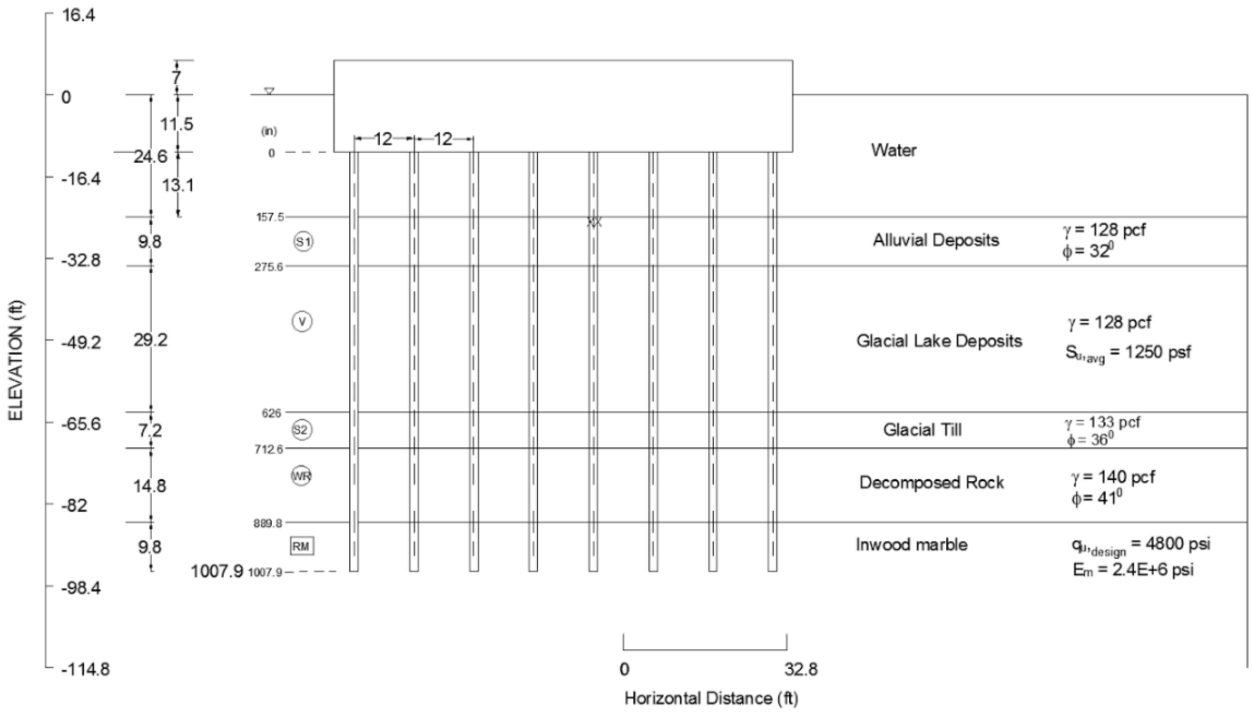
$$\#\ \text{of piles for strong axis aligned through transverse direction} = 65 \text{ x} \frac{4537}{4537 + 7872} = 24$$

$$\#\ \text{of piles for strong axis aligned through longitudinal direction} = 65 \text{ x} \frac{7872}{4537 + 7872} = 41$$

Pile configuration and is determined as below:



*All dimensions are in feet



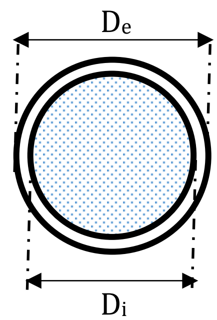
For this project, H piles is not a good option because of the followings:

- Bedrock layer is at a depth of 74.1 ft (22.6 m) below the pile cap; therefore, 84 ft (25.6 m) piles are planned to construct for being about 10 ft below the bedrock. However, it is very difficult to drive 84 ft H piles at a one time.
- Preliminary axial load calculation indicates that 65 piles needed to carry axial loading even though the H piles are HP 18" x 181 which is very big H pile. Again it is not feasible to drive 65 H-pile (HP 18" x 181).
- Since 13.1 ft of the piles (4 m) are surrounded by the water below the pile cap until the ground surface, all transverse and longitudinal loadings and moments are carried by the H piles. There will be no soil resistance to carry these loadings and moments. Since H piles have one weak and one strong axis; there is going to be a problem taking these loadings and moments.

2.) Steel Pipe Piles filled with Concrete

Second design alternative is steel pipe pile filled with concrete. In order to take huge loadings and moments, 6 ft diameter pipe piles are selected. Like H piles, sheet pile walls are going to use to make the field suitable for constructability. Although it will take more time to construct than the H-piles, pipe piles can be driven part by part into the ground. It is not necessary to drive whole pipe pile at a one time. One of the another advantage of using pipe pile with concrete is that since moment of inertia of the composite pile is much more than H-pile, loadings and moments can be carried even though 4 meter of pile section just contacts with water (pile cap sits on the water).

Pile Properties:



$\text{Outside diameter}, D_e = 6 \text{ ft} = 72 \text{ in}$

$\text{Due to corrosion, reduce by } 1/16\text{"}, \qquad D_e = 5.99 \text{ ft}$

$\text{Thickness}, t = 0.063 \text{ ft} = 0.75 \text{ in}$

$\text{Inner diameter}, D_i = 5.875 \text{ ft} = 70.5 \text{ in}$

$\text{Outside radius}, r_e = 35.94 \text{ in},$

$\text{Inner radius}, r_i = 35.25 \text{ in}$

$\text{Outside perimeter}, p = 225.8 \text{ in}$

$\text{Area of steel}, A_s = \pi\,(35.94^2 - 35.25^2) = 154 \text{ in}^2$

$\text{Area of concrete}, A_c = \pi\,(35.25^2) = 4057 \text{ in}^2$

$$\text{Moment of Inertia of Steel}, I_s = \frac{\pi}{4} \text{ x } (35.94^4 - 35.25^4) = 97406 \text{ in}^4$$

$$\text{Moment of Inertia of Concrete}, I_c = \frac{\pi}{4} \text{ x } (35.94^4) = 1212625 \text{ in}^4$$

Equivalent Section:

$\text{Assume } E_e = E_c = 3500000 \text{ psi from } E = 57000 \text{ x } \sqrt{f_c} \text{ where } f_c = 4000 \text{ psi},$

$E_s = 29000000 \text{ psi}$

$$\text{Equivalent Area, } A_e = A_c + \frac{E_s}{E_c} x A_s = 4057 + \frac{29000000}{3500000} x154 = 5331 \text{ in}^2$$

$$\text{Equivalent Moment of Inertia, } I_e = I_c + \frac{E_s}{E_c} x I_s = 1212625 + \frac{29000000}{3500000} x97406 = 2019700 \text{ in}^4$$

Ultimate Unit Tip & Side Resistance

- **Tip Resistance**

$q_p = s_u\ x\ (1 + N_\phi)$ *if the tip of the pile is at the bedrock (Inwood marble).*

$\phi = 25^0$ *from the table* 11.12 (*lecture notes*) *for marble*

$$s_u = \frac{q_u}{2} \text{ where } q_u\text{: unconfined compressive strength} \rightarrow s_u = \frac{9600}{2} = 4800 \text{ psi}$$

$$N_\phi = \tan\left(45 + \frac{\phi}{2}\right) = \tan\left(45 + \frac{25}{2}\right) = 1.57$$

$$Q_p = q_p \text{ x } A_s = 4800 \text{ x } (1 + 1.57) \text{ x } 154 = 1896477 \text{ lb} \rightarrow \mathbf{Q_p = 1896\ k}$$

- **Side Resistance**

$\rightarrow L' = 15D = 15 \text{ x } 6 = 90$ ft $\rightarrow$ critical depth (below the pile tip)

$\rightarrow$ K is used $\rightarrow K = K_0 = 1 - \sin\phi$

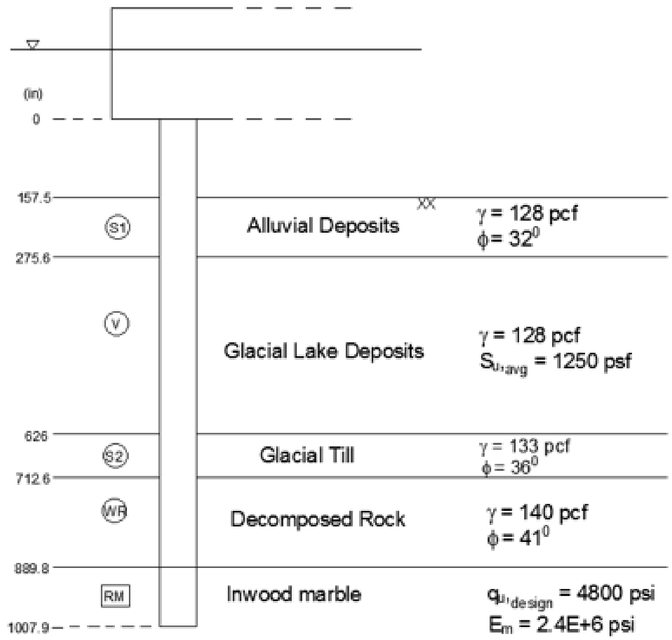
$\rightarrow \left(\delta = \frac{2}{3}\phi\right)$ is considered, $\rightarrow$ perimeter $= 225.8$ in

$\rightarrow$ There are 2 s_u given in the problem for glacial lake deposits which are 1250 and 2500 psf. To calculate f_{av} for this layer, α method (Table 11.10 in the lecture notes) is used. The table below indicates f values for 2 s_u values.

S_u (psf)	1250	2500
S_u / p_a	0.6	1.3
α	0.62	0.41
f = Su x α	775.0	1025.0

Note that f = 800 psf = 5.56 psi is used for the rest of side friction calculations.

The figure and table below indicates the soil profile with depth in inches and side friction calculation, respectively.



Layer(s)		Adjusted Depth (m)	Adjusted Depth (ft)	γ_b (pci)	σ'_{v0} (psi)	ϕ	σ'_{h0} (psi)
Footing Bottom Elevation		0	0.0	0.0000	0.00	0	
Alluvial Deposits	**start**	4	13.1	0.0380	0.00	32	0.00
	end	7	23.0	0.0380	4.48	32	2.11
Glacial Lake Deposits	**start**	7	23.0	0.0380	4.48	0	0.00
	end	15.9	52.2	0.0380	17.79	0	0.00
Glacial Till	**start**	15.9	52.2	0.0409	17.79	36	7.33
	end	18.1	59.4	0.0409	21.32	36	8.79
Decomposed Rock	**start**	18.1	59.4	0.0449	21.32	41	7.33
	end	22.6	74.1	0.0449	29.28	41	10.07

Layer(s)		Adjusted Depth (ft)	σ'_{h0} (psi)	ϕ	f_s	Length (in)	Q_s (lb)
Alluvial Deposits	**start**	13.1	0.00	32	0.41	118	10977
	end	23.0	2.11				
Glacial Lake Deposits	**start**	23.0	0.00	0	5.56	350	439553
	end	52.2	0.00				
Glacial Till	**start**	52.2	7.33	36	3.59	87	70192
	end	59.4	8.79				
Decomposed Rock	**start**	59.4	7.33	41	4.50	177	179945
	end	74.1	10.07				
						Sum (lb)	700667

Sum (k)	701

Note that 135k of skin friction is considered to calculate ultimate resistance:

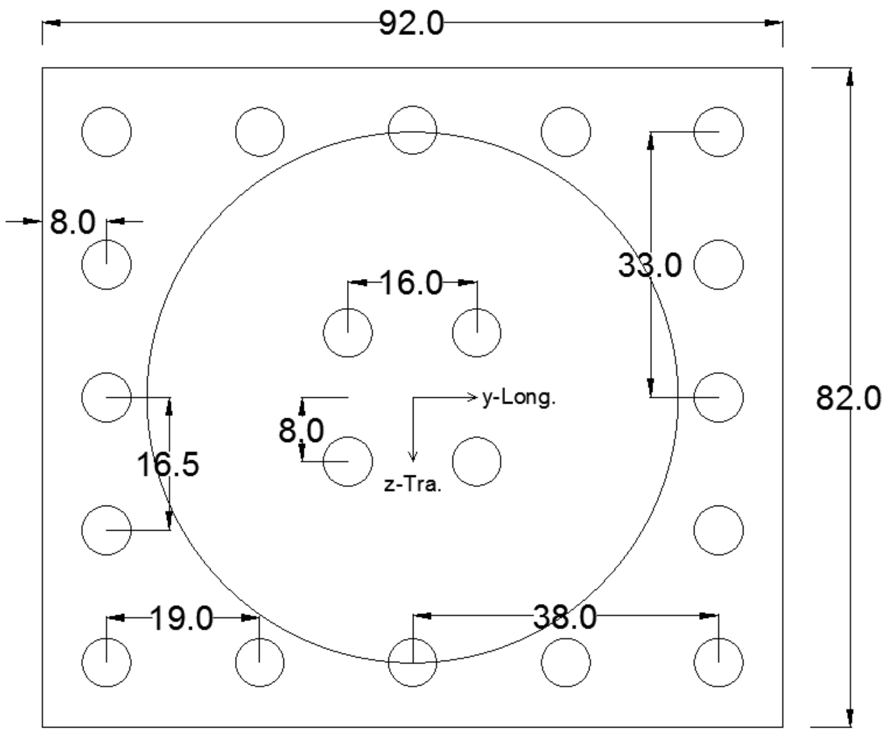
$$Q_{ult} = 1896 + 701 = 2597\ \mathrm{k}$$

$$F_v\ (\mathrm{Dead} + V) = 20725\ \mathrm{k}$$

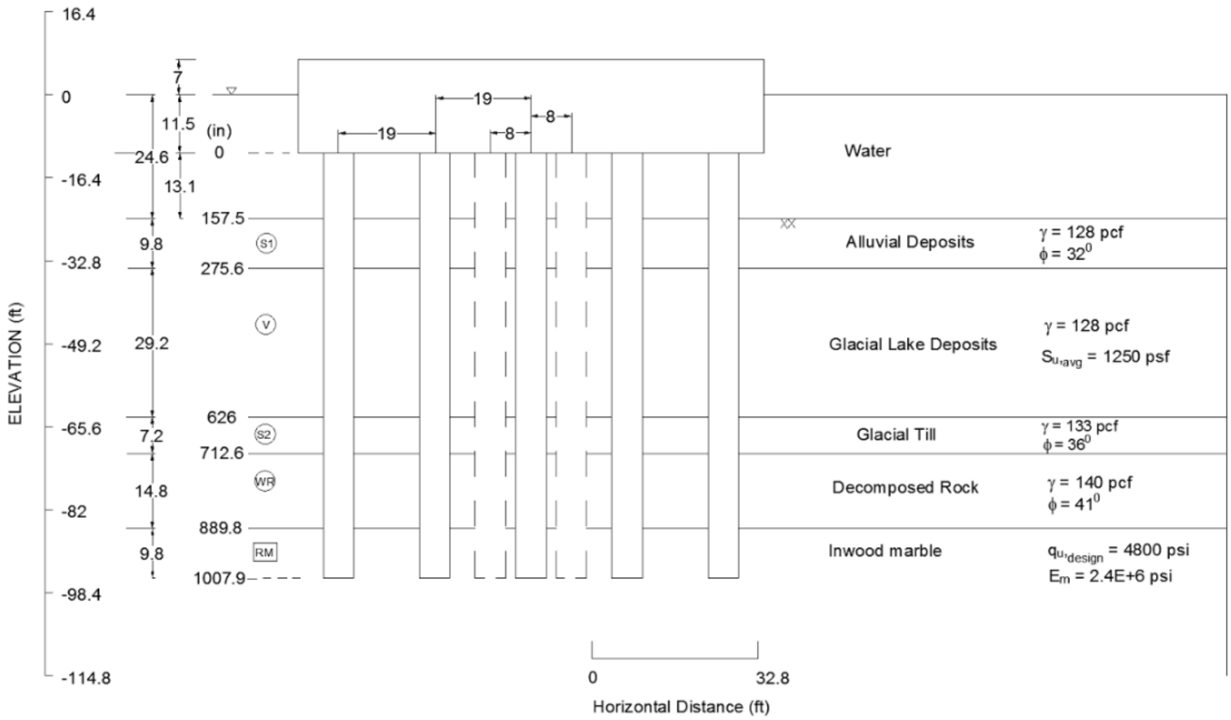
$$Q_{all} = \frac{Q_{ult}}{F.S} = \frac{2597}{2.5} = 1039\ \mathrm{k}$$

$$\#\ \mathrm{of\ piles} = \frac{20725}{1039} \rightarrow \#\ \mathbf{of\ piles = 20}$$

Initial pile configuration is selected as below. This configuration is first used to model the foundation and will be analyzed by GROUP software for horizontal deflections and rotations.



*All dimensions are in feet



From the preliminary design, 6' diameter pipe piles filled with concrete is selected as a foundation type of the center pivot pier for the Third Avenue Bridge. Preliminary design only indicated the axial load calculations; however, lateral deflections were not considered from the preliminary design. Since the foundation is going to be a group pile, it is very difficult to estimate horizontal deflections by hand due nonlinear behavior of the soil. Therefore, **"GROUP"** software is used as a tool to estimate vertical, transverse and longitudinal deflections and loadings and the moments at both the pile cap and each of the piles as well.

GROUP MODELLING

- **Pile Cross Section:**

From the preliminary design, 6' diameter pipe piles filled with concrete was selected as a pile. Since the group does not have that kind of pile, general pipe is selected and equivalent section properties which were estimated from the preliminary design are used.

$\text{Equivalent diameter} = 71.875 \text{ in (due to } 1/16'' \text{corrosion)}$
$\text{Equivalent Area} = 5331 \text{ in}^2$
$\text{Inertia zz} = \text{Inertia yy} = 2019700 \text{ in}^4$

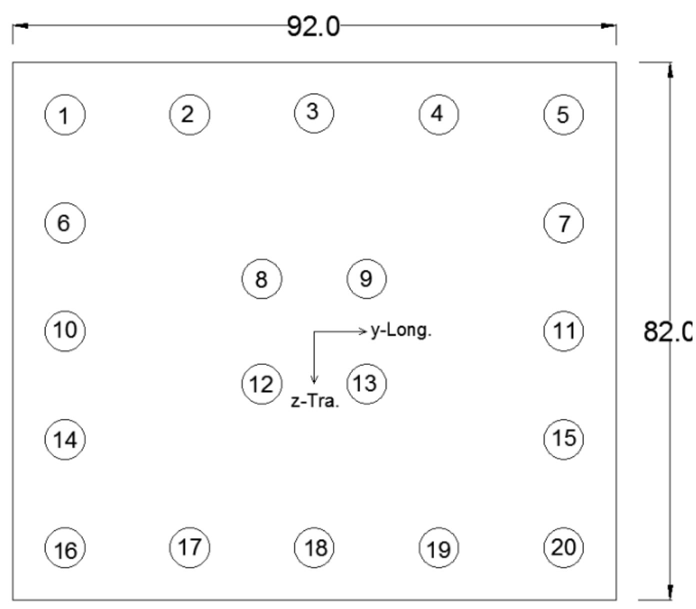
- **Pile and Pile Group Properties:**

Piles are designed so that about 10 ft of the tip of the piles will be in the bedrock layer.

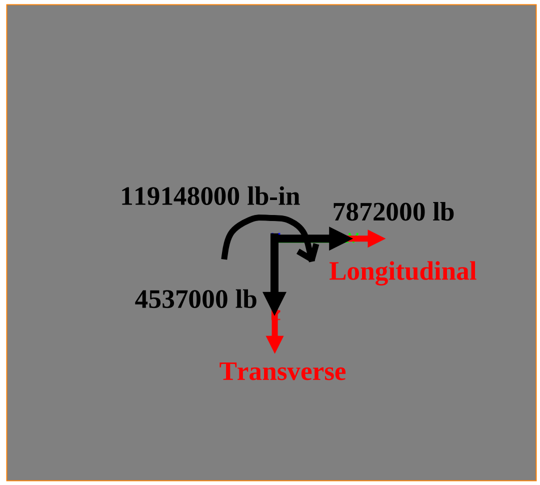
$\text{Length of the pile} = 1007.9 \cong 1008 \text{ in}$
$\text{Equivalent Young Modulus} = 3500000 \text{ lbs/in}^2$

All the pile head connections are selected as fixed in both directions (transverse and longitudinal) for taking moments. Since the spacing between the piles are larger than 2.5D = 25.x6 = 15ft, p and y-multipliers are chosen as 1.

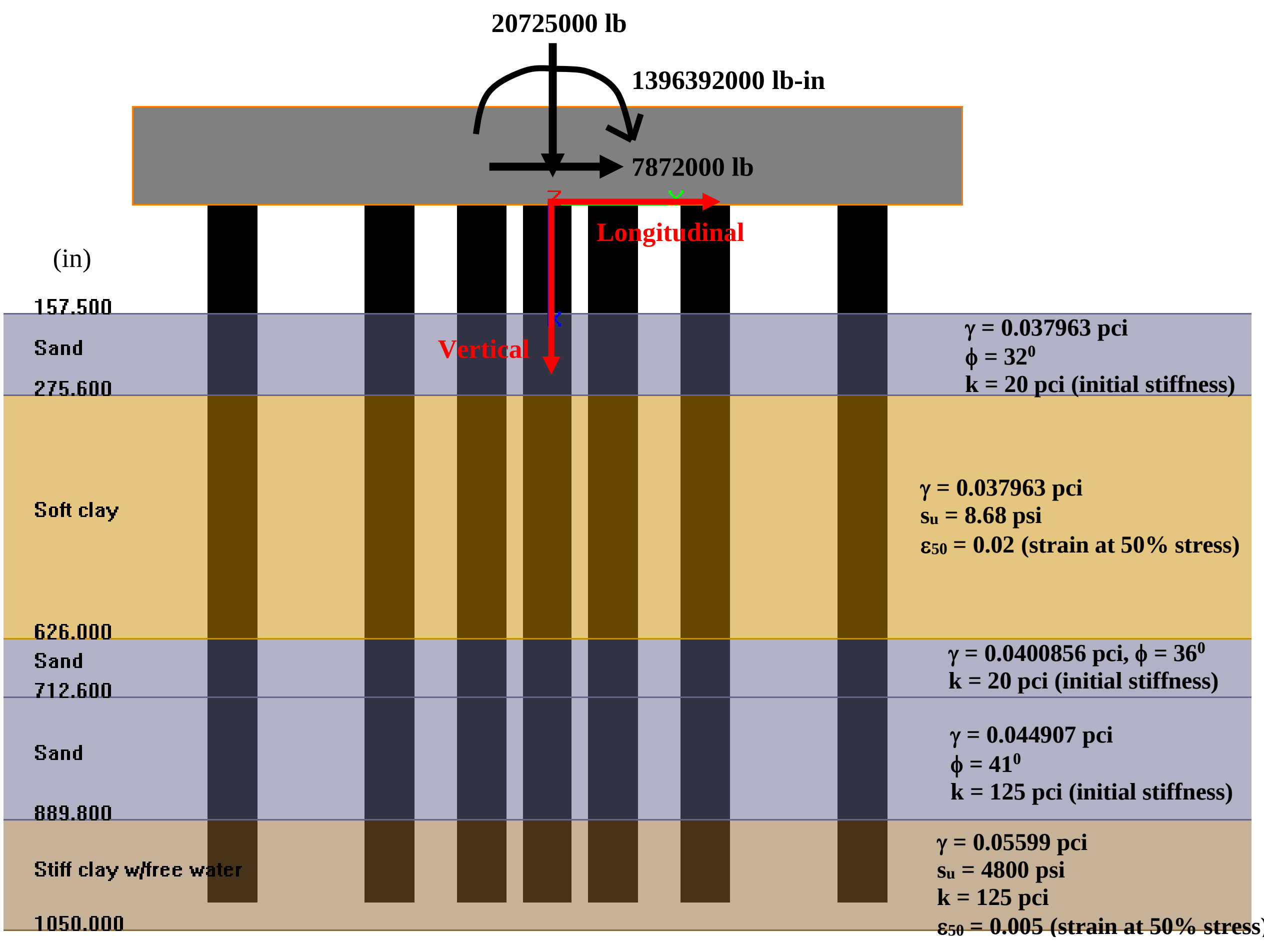
- **Pile Configuration**



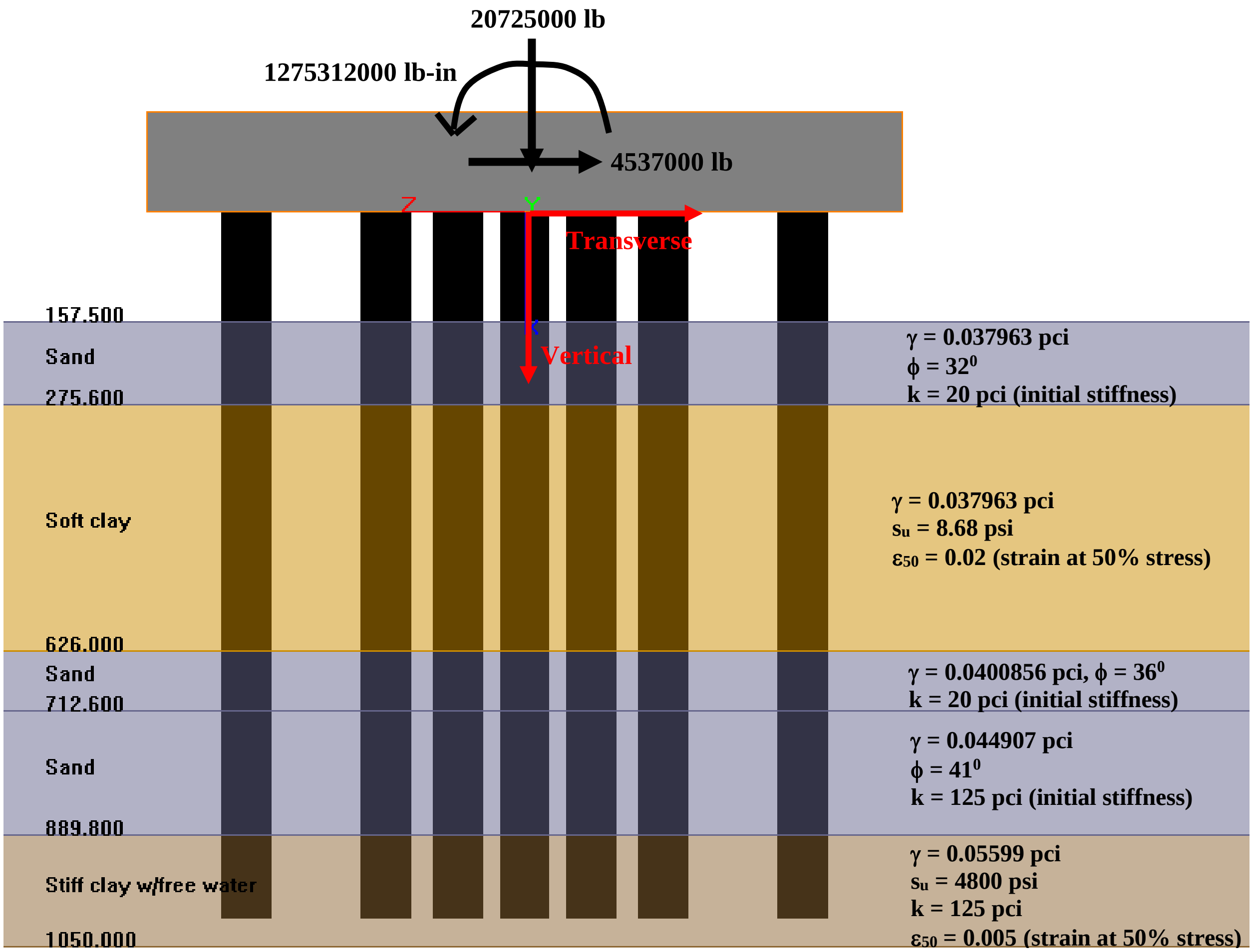
- **Loading and Soil Layer Properties**
  - **y – z axis (longitudinal – transverse axis)**



  - **x – y axis (vertical – longitudinal axis)**



- **x – z axis (vertical – transverse axis)**



- **Results**
  - **Pile Cap Displacements, in**

Vertical displacement, U_v = 0.058505 in = **0.06 in**

Transverse displacement, U_T = 0.89224 in = **0.9 in**

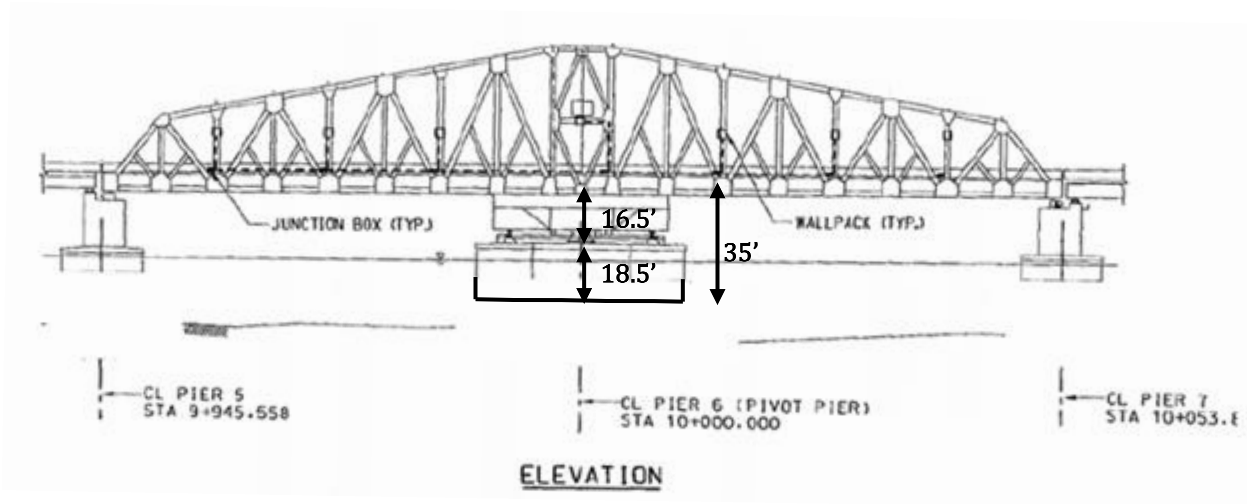
Longitudinal displacement, U_L = 1.52868 in = **1.53 in**

- **Pile Cap Rotations, rad**

Vertical rotation, θ_v = 1.29673E-4 rad = **1.30E-4 rad**

Transverse rotation, 9.31751E-5 rad = **9.32E-5 rad**

Longitudinal rotation, θ_L = -1.01046E-4 rad = **-1.01E-4 rad**



Longitudinal displacement at the superstructure level:

$\mathbf{U_{lon\text{-}total}} = U_L + \theta_L$ x (elevation from the pile cap base)

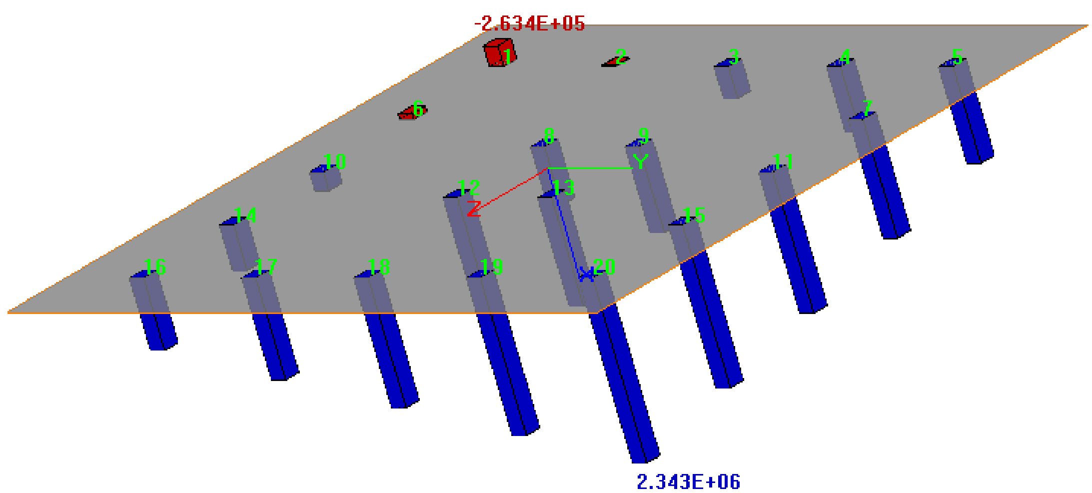
$\mathbf{U_{lon\text{-}total}} = 1.53 + 1.01E\text{-}4 \times 35 \times 12 =$ **1.57 in**

Limitation is 2 in at the superstructure level. The design is acceptable for displacements.

- **Pile Forces, kip**

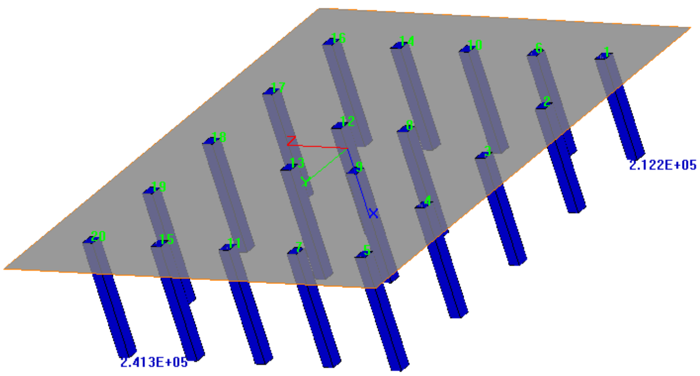
Max pile axial force, f_{Vmax} = 2343 k (compression)

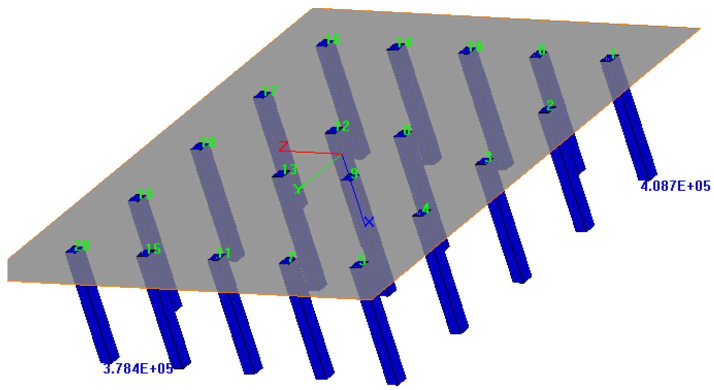
Min pile axial force, f_{Vmin} = -263 k (tension)



Max pile transverse shear force, f_{Tmax} = 241 k

Max pile longitudinal shear force, f_{Lmax} = 408 k





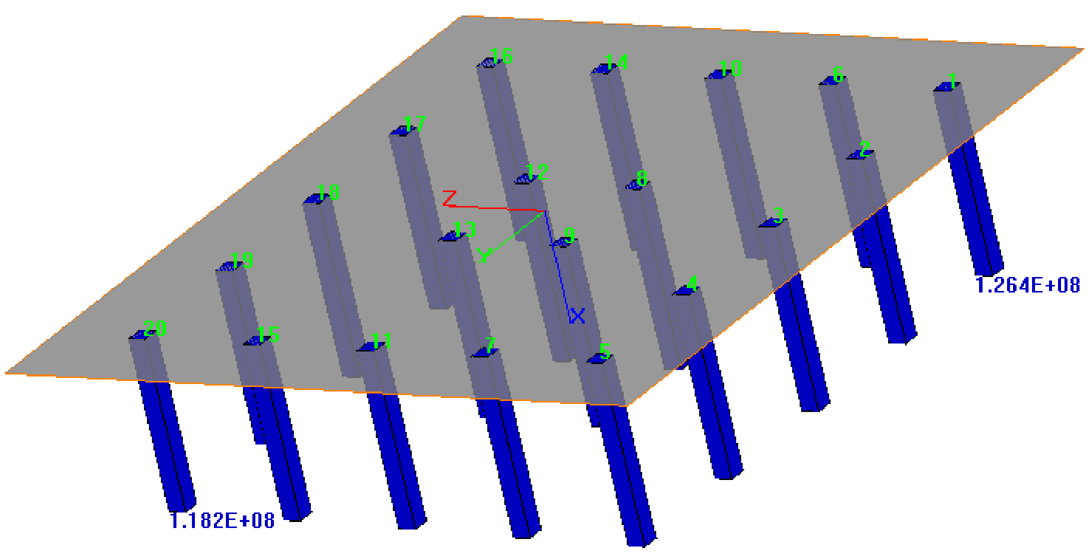
- **Pile Moments, kip-ft**

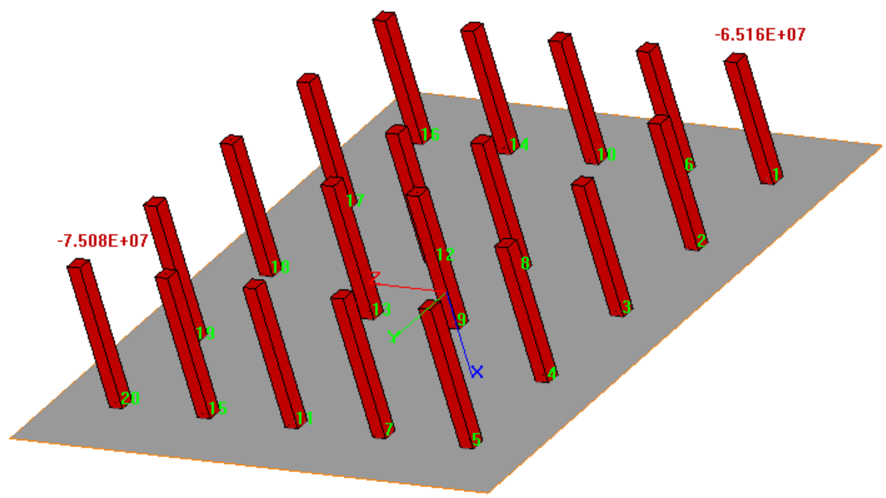
Moments are selected from the critical pile which has the maximum axial force. Because structural capacity of the pile is generally governed with the axial load (P/A±Mc/I).

Max vertical pile moment, m_{Vmax} = 0.08 k-ft

Max transverse pile moment, m_{Tmax} = 9852 k-ft (1.18E+08 lb-in)

Max longitudinal pile moment, m_{Lmax} = 6256 k-ft (7.51E+08 lb-in)





Summary:

	Pile Cap Displacements, in			Pile Cap Rotations, rad		
	U_V	U_T	U_L	θ_V	θ_T	θ_L
L + T + V	0.06	0.9	1.53	1.3E-04	9.32E-05	-1.01E-04

	Pile Forces, kip				Pile Moments, kip-ft		
	f_{Vmax}	f_{Vmin}	f_{Tmax}	f_{Lmax}	m_{Vmax}	m_{Tmax}	m_{Lmax}
L + T + V	2343	-263	241	408	0.08	9852	6256

*Moments are taken from the critical pile which is pile 20.

Construction of the pipe piles filled with concrete:

First, 6 ft diameters pipe piles are going to be driven into the ground (20 of them). The top of the piles are located above the water level. At the pile cap bottom level, divers will weld bulges around the pipe piles so that precast concrete pile cap will sit on these bulges. Precast concrete pile cap is designed in a way that there are holes for the piles and pile cap will be placed on the piles and sit on the bulges. Pile holes should be a little bit bigger so that pile holes will be able to get in the pipe piles. After placing the pile cap, the concrete will be poured until the pile cap bottom elevation and pipe pile will be cut from top to the pile cap bottom elevation.

Reinforcement bars in the concrete in the pipe pile and reinforcement bars in the precast concrete pile cap will be connected so that they can work together. At the end, concrete will be poured for the piles holes and the center pivot pier will be ready for the swing system to put.