Fall 14

CE 818 Design Project

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Raft versus Pile Foundation Design Problem

Michigan State University

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ABSTRACT

In this report, a foundation design and analysis for fifty-one-story building is going to be conducted. There are two foundation types being considered and these foundation types are going to be checked whether they are safe or not in terms of several aspects in geotechnical perspective.

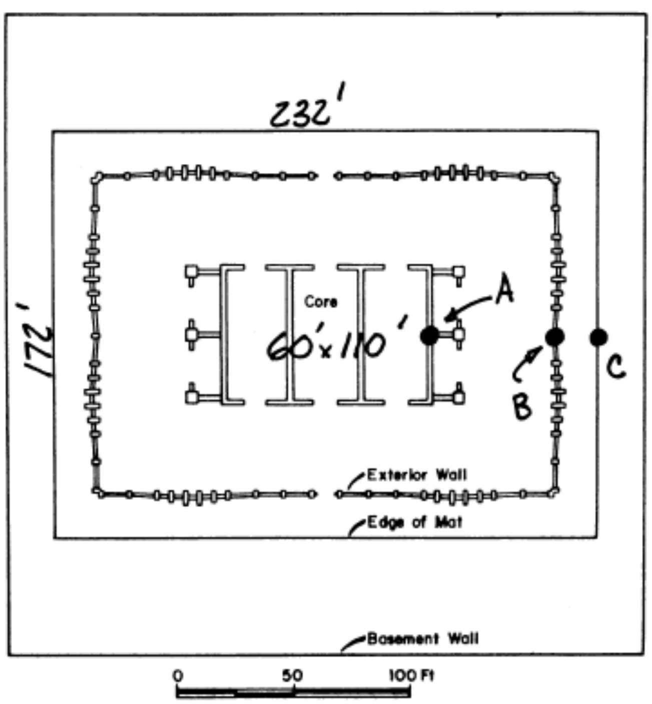
This design paper includes a brief introduction part, soil configuration and properties on which building stands on, design and analysis part for different foundation types, summary and conclusion.

During the report, results are explained where they come from, which method is used etc. All detailed calculations are in the appendices part.

INTRODUCTION

In the scope of this paper, foundation of a fifty-one-story office building will be designed and analyzed. Since the building has so huge pressure on the soil that the foundation can be designed with considering different types of approaches during the analyzing. There are 2 foundation design options being considered: (a) 60 meter deep excavation and the building is going to be constructed on mat foundation or the other option is that (b) pile foundation system is designed for appropriate pile lengths, sizes and configuration so that the foundation is able to carry the load due to the building itself.

The building plan is as the figure below. There are three parts that form the building which are main core, exterior wall and garage column. The main pressure is due to the main core on which the building rises. The effect of the walls comparing to the main core is so small; therefore, the design should be done with consideration of this fact.



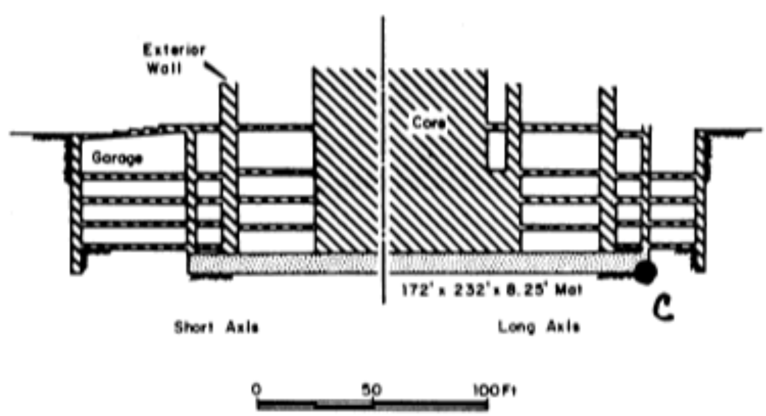


Figure 2: Building Plan Cross-Section

Figure 1: Basement Cross Section

SOIL CONFIGURATION AND PROPERTIES

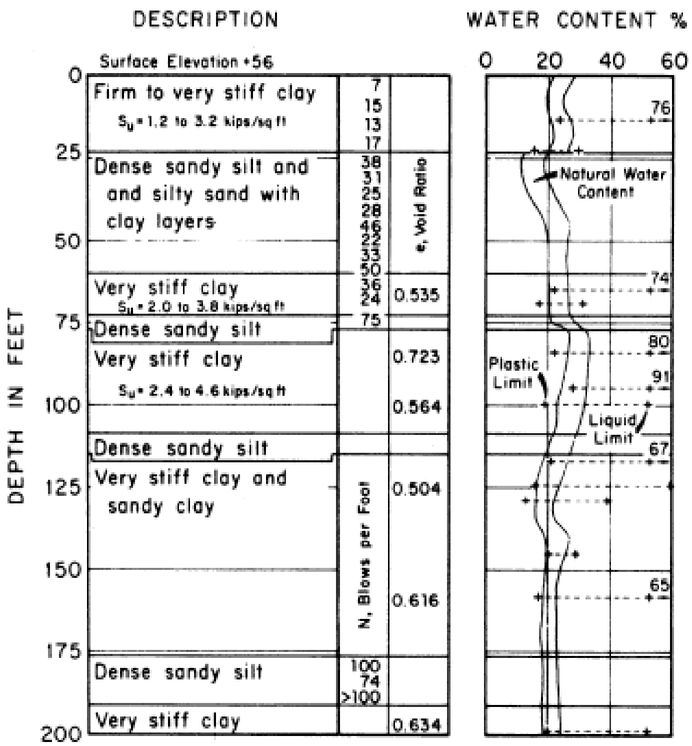


Figure 3: Soil Profile

The soil under the fifty-one-story office building generally consists of stiff and dense soils. Under the surface elevation, the soil profile begins with a firm to very stiff clay and continues with dense sandy silt very stiff clay in sequence at 4 times up to reaching at a depth of 200 feet. The soil profile and some test results such as SPT-N, void ratio and Atterberg Limit Test for each layer is as shown the Figure 3. The water table does not appear during the field tests or do not indicated in the field test results; therefore, the water table depth is assumed to be deep enough in order not to be included the water effect in the calculations.

There are some missing parameters such as unit weight and specific gravity of the soil layers at the end of the field tests, and these parameters are either assumed or calculated by some correlations. The soil layers also are assumed to be fully saturated.

There are some G_s values available, and unit weight of the soil layers are calculated by using these G_s values and water content of corresponding soil layer (Das, 2011). Soil profile and configuration with soil properties are tabulated in the Appendices A.

The consolidation test data for 3 different depths is available in the Appendices B and preconsolidation pressure is obtained by using Casagrande Method. According to this method (Holtz, Kovacs, & Thomas, Compressibility of Soil and Rock, 2011);

The straight line portions of the e – log(σ'_v) curve at the break in the curve is extended (line 1 and line 2) and a line 3 is placed at the break point parallel to the log σ'_v axis.

The line 4 is placed in a direction so that the angle between line 1 and 3 should be equal. The pressure value at the intersection of line 2 and 4 is preconsolidation pressure.

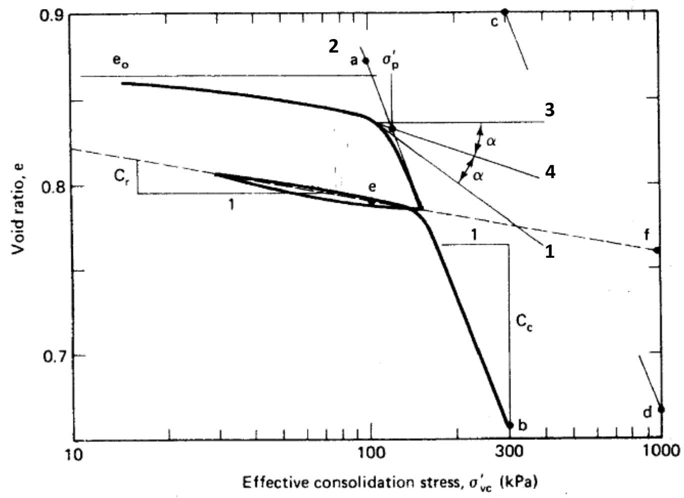


Figure 4: Obtaining Pre consolidation Pressure

The consolidation test data was obtained by the laboratory tests and during the transporting process from the field and laboratory, the sample can be damaged and so the sample is accepted as disturbed. Therefore, the lab data is adjusted for the field by the Schmertmann procedure (Holtz, Kovacs, & Thomas, Compressibility of Soil and Rock, 2011).

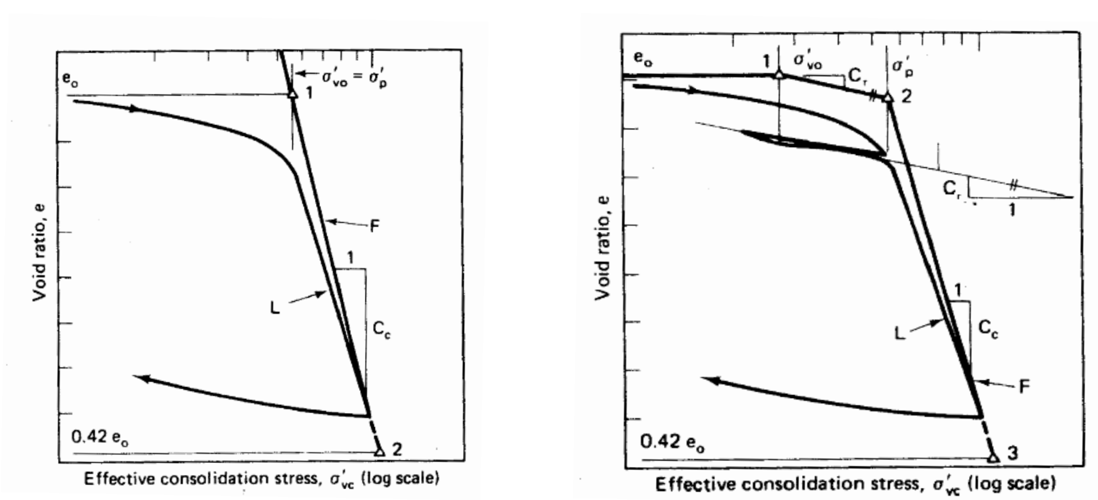


Figure 5: Obtaining Compression Index (Schmertmann)

For Schmertmann adjustment, preconsolidation pressure should be obtained by Casagrande method, and initial void ratio should be known at the end of the field tests. The method assumes that, the field and laboratory e - $\log(\sigma'_v)$ line intersects at the void ratio is equal to $0.42e_0$ and the slope of the extension line to the laboratory virgin compression line is equal to C_c. For normally consolidated clays, the intersection between a horizontal line from e_0 and the extension line from $0.42e_0$ with a slope of C_c is preconsolidation pressure.

Although the problem states that the clay layers are over consolidated, the OCR from the calculation of preconsolidation stress that obtains from the graph over initial stress is less than one (1) which means that the clays are under consolidated. However; after taking the averages of OCR which is calculated from the preconsolidation pressures from some correlations (which is showed up in appendices), the OCR is bigger than 1.

FOUNDATION DESIGN AND ANALYSIS

There are 2 different foundation types considered for fifty-one-story office building. The first option is deep excavation and constructing mat foundation. The other option is that instead of deep excavation, 15 meter excavation and then pile foundation is going to be constructed. In this part, each option is studied individually and at the end, the results for each foundation types are considered.

Both foundation types are generally examined in terms of bearing capacity and settlement aspect in geotechnical vision.

DEEP EXCAVATION AND MAT FOUNDATION

The first option is to excavate excessive soil on which the building constructs; therefore, huge stresses releases before the load coming from the building itself.

The foundation dimensions and properties can be presented as the table below:

Foundation Dimensions & Properties		
B	**ft**	172
L	**ft**	232
D_f	**ft**	60
Gross Pressure	**kip / ft^2**	8.45

Table 1: Foundation Dimensions

- **Bearing Capacity**

At 60 ft depth, the building is on a very stiff clay layer. The initial vertical stress at this depth is 7.786 kip / ft^2. After removing the soil by excavation, the soil relieves as initial amount of vertical stress. After the load of the building is applied on the soil at the depth of 60 ft, net pressure on the load will be the difference between pressure from the building and the soil pressure before the excavation.

$$q_{net} = q_{net} - \sigma'_v$$

Since the mat foundation sits down the very stiff clay layer and condition of the soil is fully saturated, friction angle (Ø) is equal to zero. For bearing capacity calculation, the general bearing capacity equation is used after taking fully saturated condition, shape and depth factors into consideration.

$$q_{bL} = (s_c d_c i_c b_c g_c)cN_c + (s_q d_q i_q b_q g_q)q_0 N_q + \frac{1}{2}(s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma)\gamma B N_\gamma$$

$$q_{net(u)} = 5.14c_u(1 + \frac{0.195B}{L})(1 + 0.4\frac{D_f}{B})(F_{ci})$$

The allowable bearing capacity when the factor of safety is taken as 3 is found as **6.22 ksf** which is bigger than applied pressure after subtracting from excavation pressure which is **0.664 ksf**. Therefore, in the aspect of bearing capacity, the mat foundation with deep excavation is okay.

- **Settlement**

Settlement of the foundation consists of 2 type of settlements which are immediate and consolidation settlement.

- ***Immediate Settlement***

Immediate settlement occurs just after the construction of the building is done. There is no general rule regarding constraints of the immediate settlement. However, any immediate settlement bigger than 1 inch settlement is considered as not safe. Therefore, the limiting value for immediate settlement is 1 inch.

Immediate settlement is calculated from Janbu's Method for saturated clay.

$$S_e = A_1 x A_2 \frac{q_0 x B}{E_s}$$

From the formula A_1 and A_2 is a factor related with foundation depth, width, length and distance between foundation depth and hard soil strata. q_0, B and E_s is net pressure on the soil, foundation width and modulus of elasticity respectively.

At the end of the calculation (Appendices C) , the immediate settlement is calculated as 0.12 inch which is less than 1 inch. Therefore, in aspect of the immediate settlement, the mat foundation is suitable.

- ***Consolidation Settlement***

Consolidation settlement occurs at the saturated clay layers under loading. The soil profile under the fifty-one-story office building has 4 different clay layers and there are 3 consolidation test data for these clay layers. However, the depth of stiff clay layer for which the consolidation test data is not available begins at 200 ft which means the soil layer is deep enough in order not to be affected by building load.

First clay layer just under the foundation is examined as a one part, second clay layer (z = 86 to 112 ft) is examined as 2 equal layers and last clay layer (z =125 to 181 ft) is divided into 4 equal layers during calculation of consolidation settlement.

Since all clay layers are over consolidated, below consolidation settlements formula is used:

$$S_c = \frac{C_s H_c}{1+e_0} log \frac{\sigma'_c}{\sigma'_0} + \frac{C_c H_c}{1+e_0} log \frac{\sigma'_0 + \Delta'\sigma_{avg}}{\sigma'_0}$$

where

σ'_0: average effective pressure on the clay layer before foundation construction

$\Delta'\sigma_{avg}$: average increase in effective pressure on the clay layer

σ'_c: pre consolidation pressure

e_0: initial void ratio

C_c: compression index

C_s: swelling index

H_c: thickness of the clay layer

Since the layers are divided into sub layers, instead of taking average of effective pressure, pressures at the mid depth of this sub layer is used during the calculation. In addition, C_c and C_s values are calculated from consolidation test data graph and some correlations, which are indicated in Appendices C.

At the end of the consolidation settlement calculation, 1.294-inch consolidation settlement is found which means that the settlement is less that 10 inch and there will be no problem according to settlement for mat foundation.

PILE FOUNDATION

Instead of mat foundation with deep excavation, other option is to construct pile foundation. Pile foundation is constructed under the fifty-one-story office building with 15 ft excavation. In other words, at this time, excavation for pile foundation is less than excavation for mat foundation due to the fact that bearing capacity condition is provided by pile foundation.

For pile foundation, 15 ft excavation is conducted and 3 ft slab thickness, 2.5 ft diameter and 40 meter length is assumed above piles at the first. If these foundation dimensions provide enough bearing capacity to carry load due to the building and satisfy settlement condition for reasonable number of piles, calculations are going to be continued with these dimensions.

In this part, bearing capacity of the single pile is going to be calculated and then number of piles being used for pile foundation is determined. After determining number of piles, calculation for bearing capacity is again going to be done with taking group effect of piles into consideration. Settlement is also going to be calculated with group effect. At the end of the calculations, all results are going to be checked with design criteria's.

All detailed calculations are represented in Appendices D. In this part, only results are shown and discussed.

Bearing Capacity of Single Pile

Bearing capacity of pile foundations consist of two parts: end bearing capacity and skin resistance.

- ***End Bearing Capacity***

Piles reach at depth of 58 meter at which dense sandy silt and silty sand with clay layer exists. During end bearing capacity calculations, soil parameters which belong to that layer are used.

There is couple of ways for calculating end bearing capacity. In this section, end bearing capacity is examined by using Meyerhof, Vesic and Coyle Costello Methods. Field Correlations by Meyerhof is also taken into consideration and end bearing capacity results for all these methods are going to be taken an average and average result is accepted as end bearing capacity.

The table is represented below showing end bearing results for different approaches.

Method	Q_p (kip)
Meyerhof	1145.38
Vesic	2659.83
Coyle and Castello	3693.51
Field Corr. (Meyerhof)	1476.30

Table 2: Bearing Capacities for Different Methods

At the end, the average of these values is calculated as 2150.58 kip is used for end bearing capacity.

- ***Skin Resistance***

The other thing affected bearing capacity is skin resistance of the pile. 40 meter pile at the beginning depth of 18 ft lies through two different soil layers which are firm to very stiff clay (z=18 to 24 ft) and dense sandy silt (z=24 to 58 ft).

For clay layer, α and λ methods are used in order to calculate skin resistance. Since drained friction angle of remolded clay is not provided, β method is not able to be used for calculating skin resistance. After calculations for clay layer with 2 methods, average of these values are taken.

For sand layer, skin resistance is calculated by Coyle and Castello method and also field correlations by Meyer and Briaud. Again, after calculations of skin resistance with these methods, averages are taken and used for further calculations.

At the end of the calculations, side resistances of a single pile for clay and sand layer are found as the results indicated as below table:

<table>
<tr><th colspan="2">Side Friction</th><th>Q_s (kip)</th><th>Average (kips)</th></tr>
<tr><td rowspan="2">clay</td><td>α method</td><td>51.71</td><td rowspan="2">99.54</td></tr>
<tr><td>λ method</td><td>147.37</td></tr>
<tr><td rowspan="3">sand</td><td>Coyle and Castello</td><td>1507.85</td><td rowspan="3">753.34</td></tr>
<tr><td>Field Corr. (Meyer)</td><td>399.57</td></tr>
<tr><td>Field Corr. (Briaud)</td><td>352.60</td></tr>
</table>

Table 3: Skin friction of a Single Pile

To summary,

<table>
<tr><th colspan="2">End Bearing Capacity</th><th>Q_p (kip)</th><th>Average (kip)</th><th>Total (kip)</th><th>Q_{ult} (kip)</th><th>F.S</th><th>Q_{all} (kip)</th></tr>
<tr><td colspan="2">Meyerhof</td><td>1145.38</td><td rowspan="4">2150.58</td><td rowspan="4">2150.58</td><td rowspan="11">3003.46</td><td rowspan="11">2.50</td><td rowspan="11">1201.38</td></tr>
<tr><td colspan="2">Vesic</td><td>2287.13</td></tr>
<tr><td colspan="2">Coyle and Castello</td><td>3693.51</td></tr>
<tr><td colspan="2">Field Corr. (Meyer)</td><td>1476.30</td></tr>
<tr><td colspan="5"></td></tr>
<tr><td colspan="2">Side Friction</td><td>Q_s (kip)</td><td>average (kip)</td><td>Total (kip)</td></tr>
<tr><td rowspan="2">clay</td><td>α method</td><td>51.71</td><td rowspan="2">99.54</td><td rowspan="5">852.88</td></tr>
<tr><td>λ method</td><td>147.37</td></tr>
<tr><td rowspan="3">sand</td><td>Coyle and Castello</td><td>1507.85</td><td rowspan="3">753.34</td></tr>
<tr><td>Field Corr. (Meyer)</td><td>399.57</td></tr>
<tr><td>Field Corr. (Briaud)</td><td>352.60</td></tr>
</table>

Table 4: Bearing Capacity of the Single Pile

At the end, Q_{ult} is calculated by sum of end bearing and skin resistance and is found as 3003.46 kip. Q_{all} is calculated by dividing Q_{ult} to factor of safety and is calculated as 1201.38 kip.

Load due to the building is calculated in different way than the calculations from mat foundation. At this time, load is calculated by using the figure of loads and pressures along short axis of mat and building plan. Loads for core, external wall and garage wall is calculated by separately by multiplying pressures with dimensions.

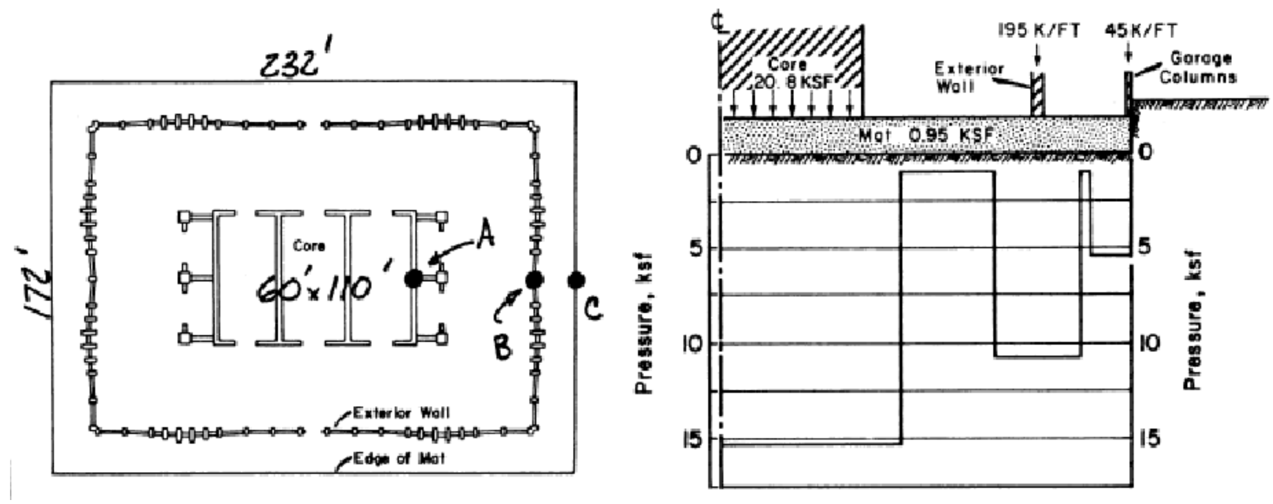


Figure 6: Pressure Distribution of the Building

	B (ft)	L (ft)	Perimeter (ft)	Area (ft^2)	q (ksf or k/ft)	Q_{net} (kip)
Q_{core}	60	110	340	6600	15.5	102300
$Q_{extwall}$	134	200	668	26800	11	7348
$Q_{garwall}$	172	232	808	39904	5.5	4444
				Q_{net} (kip)	Sum	**114092**

Table 5: Calculation of Loading due to the Building

Pressures are taken by using pressure diagram figure and lengths of core, exterior and garage wall are measured by using scale and at the end, Q_{net} is calculated as 114092 kip.

Number of piles is calculated as:

$$\#\ of\ piles = \frac{Q_{net}}{Q_{all}} = \frac{114092}{1201.38} = 95$$

If the pile number is calculated for different places, the core is going to take the most number of piles. Although total number of piles for different places (under core, exterior wall and garage wall) is 95, 85 piles have to be constructed under the core. Remaining 10 piles under exterior and external wall theoretically resist loading due to these walls loading; however 10 piles are not reasonable for huge dimensions. Therefore, by decreasing pile dimensions, number of piles under exterior wall and garage wall can be increased.

For increasing number of piles, pile dimensions under exterior and garage wall is that diameter and length are 1 and 10 ft, respectively.

Like diameter of 2.5 ft and length of 40 ft pile foundation, after calculations are done, total bearing capacity of one single pile (D=1 ft, L=10 ft) is found as **115.64 kip**.

The table below is the summary of the calculations for single pile (D=1 ft, L=10 ft):

<table>
<tr><th colspan="2">End Bearing Capacity</th><th>Q_p (kip)</th><th>Average (kip)</th><th>Total (kip)</th><th>Q_{ult} (kip)</th><th>F.S</th><th>Q_{all} (kip)</th></tr>
<tr><td colspan="2">Meyerhof</td><td>183.26</td><td rowspan="4">222.38</td><td rowspan="4">222.38</td><td rowspan="12">289.11</td><td rowspan="12">2.50</td><td rowspan="12">115.64</td></tr>
<tr><td colspan="2">Vesic</td><td>253.02</td></tr>
<tr><td colspan="2">Coyle and Castello</td><td>282.71</td></tr>
<tr><td colspan="2">Field Corr. (Meyer)</td><td>170.51</td></tr>
<tr><td colspan="5"></td></tr>
<tr><td colspan="2">Side Friction</td><td>Q_s (kip)</td><td>average (kip)</td><td>Total (kip)</td></tr>
<tr><td rowspan="2">clay</td><td>α method</td><td>20.68</td><td rowspan="2">39.81</td><td rowspan="5">66.73</td></tr>
<tr><td>λ method</td><td>58.95</td></tr>
<tr><td rowspan="3">sand</td><td>Coyle and Castello</td><td>45.36</td><td rowspan="3">26.92</td></tr>
<tr><td>Field Corr. (Meyer)</td><td>18.80</td></tr>
<tr><td>Field Corr. (Briaud)</td><td>16.59</td></tr>
</table>

Table 6: Bearing Capacity Calculation for Pile Type 2

At the end, dimensions of pile under core: D = 2.5 ft, L = 40 ft (Pile Type 1);

Dimensions of pile under ext. and garage wall: D = 1 ft, L = 10 ft (Pile Type 2);

In terms of pile capacity and loading due to the building, the number of piles should be:

	B (ft)	L (ft)	Perimeter (ft)	Area (ft^2)	q (ksf or k/ft)	Q_{net} (kip)	Q_{all} (kip)	# of piles
Q_{core}	60	110	340	6600	15.5	102300	1201	85
$Q_{extwall}$	134	200	668	26800	11	7348	116	64
$Q_{garwall}$	172	232	808	39904	5.5	4444	116	38
				Q_{net} (kip)	Sum	114092		187

Table 7: Number of Different Types of Piles

At Appendices E, the foundation layout in detailed is shown. The number of piles under core, exterior wall and garage wall is table as shown below:

	# of piles	Diameter (ft)	Length (ft)
Q_{core}	84	2.5	40
$Q_{extwall}$	66	1	10
$Q_{garwall}$	40	1	10

Table 8: Number of Piles

Assume that load under the core does not go down vertically directly, the loading also spreading horizontally. Therefore, 84 numbers of piles are not placed exactly in the core area.

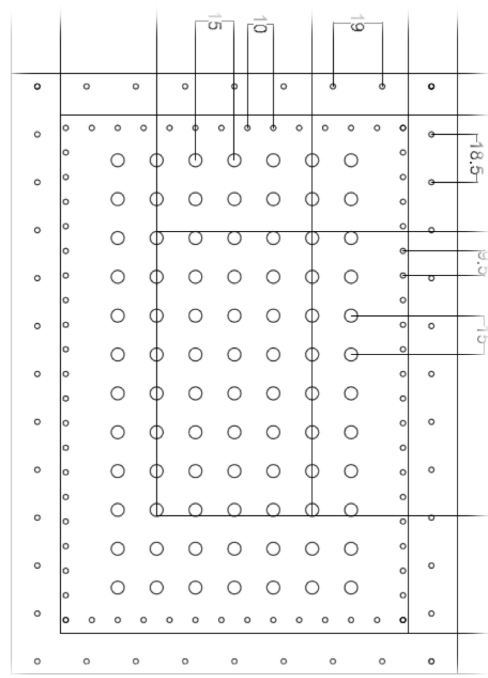


Figure 7: Layout of Piles

Lateral Loading

Another issue that needs to be checked is the total resistance of piles to the applied horizontal load. Applied horizontal load on the building due to the hurricane loading is calculated as 10981 kip (At the beginning of Appendices C)

The lateral resistance of single pile type 1 and 2 are calculated at Appendices F. After the sum of lateral load resistance of pile type 1 and 2, lateral load resistance can resist the existing horizontal load which means lateral loading of pile is not going to be a problem for future.

After calculations are done in Appendices F for both types of piles:

		# of piles	$Q_{u(g)}$ (kip)	Total $Q_{u(g)}$ (kip)	Q_h (kip)
Q_{core}	**Type 1**	85	282	24013	
$Q_{extwall}$	**Type 2**	64	84	5337	10981
$Q_{garwall}$	**Type 2**	38	84	3228	
			Sum (kip)	32578	

Table 9: Lateral Resistance and Loading

Total lateral resistance force is calculated as 32578 kip which is higher than lateral force acting on the foundation which is 10981 kip. Therefore, the pile foundation is safe in terms of lateral loading condition.

Group Effect

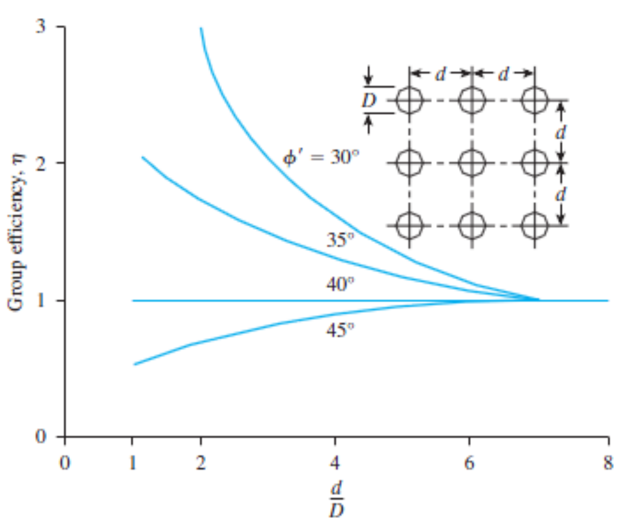


Figure 8: Group Effect in Bearing Capacity for Sand Layer

The pile places in the sand layer and end of the piles are in the sand layer. The friction angle of sand layer (z = 24 ft to 60 ft) is about 40^0. The figure indicates that no matter what distance between piles and diameter of piles are, if the end tip of the pile is in the sand layer and he friction angle is equal to 40 degree, group efficiency factor is 1 which means there is no group effect in terms of bearing capacity.

Total bearing capacity is calculated by one by which was done at bearing capacity of single pile section and number of piles actually was calculated from bearing capacity of single pile. Therefore, in terms of bearing capacity, there is no group effect and the pile foundation stays safe side.

In terms of settlement aspect for group of pile, there is two type of settlement: elastic and consolidation settlement. Since the main loading to the soil coming from core (about 92% of total), during settlement calculation, only piles under core and core loading is going to be taken into consideration.

After immediate and consolidation settlement calculations are done for group piles, total settlement of group piles is found as 5.7 inches that is acceptable value such a huge building.

RESULTS

There are two foundation options examined for fifty-one-story building. The first option is mat foundation with deep excavation. At the end of the calculations, it is found that deep excavation takes so much stresses on the soil and therefore, the building is going to stand on the soil layer after excavation. It is expected that there is not going to face any problems related with bearing capacity and settlement. The table below indicates summary of the calculations in the aspect of bearing capacity and settlement condition.

BEARING CAPACITY CONDITION		
$q_{net(all)}$ =	kip / ft^2	0.681
q_{net} =	kip / ft^2	0.664
F.S =		3.00
SETTLEMENT CONDITION		
Immediate Sett.	**inches**	0.12
Consolidation Sett.	**inches**	1.294

Table 10: Summary of Mat Foundation

The other foundation choice is constructing piles without deep excavation. The main core of the building exerted the main load to the soil, and compared to main core, external and garage wall exerts smaller loads to the soil. Due to that inequality, two different piles are designed. The pile type 1 is bigger than pile type 2. The table indicates dimensions and numbers of these two types of piles.

	# of piles	Diameter (ft)	Length (ft)
Type 1	84	2.5	40
Type 2	106	1	10

Table 11: Number of Piles

The behaviors of piles are examined in single itself and also in-group piles. The pile design consisting of two different types of piles also resists the loading coming from main core, external and garage wall in terms of bearing capacity, lateral loading and settlement in single and in-group behavior.

BEARING CAPACITY CONDITION		**Type 1**	**Type 2**
End Bearing	**kip**	2150.6	228.4
Skin Resistance	**kip**	852.9	66.7
Total		3003.5	289.1
F.S		2.5	2.5
Q_{net} =	**kip**	1201.4	115.6
SETTLEMENT CONDITION (Group)		**Type 1**	
Immediate Sett.	**inches**	0.94	
Consolidation Sett.	**inches**	4.8	

Table 12: Summary of Pile Foundation System

To conclude, both mat foundation and pile foundation can resist the building load in terms of geotechnical term.

APPENDICIES

A. Soil Profile and Properties

Layer Thickness (ft)	Depth (ft)	Description	N	e (Void Ratio)	w_L	PL	LL	γ_{sat}	s_u	σ'_0	N_{60}
	0		blow/ft		%	%	%	kip / ft^3	kip / ft^2	kip / ft^2	blow/ft
24	6	*Firm to very stiff clay*	7		24	24	76	0.128	2.2	0.769	7
	12		15							1.538	16
	18		13							2.307	14
	24		17							3.076	18
36	29	*Dense sandy Silt and silty Sand with clay layer*	38		20	--	--	0.131	--	3.665	40
	33		31							4.254	33
	38		25							4.842	26
	42		28							5.431	29
	47		46							6.020	48
	51		22							6.608	23
	56		33							7.197	35
	60		50							7.786	53
12	66	*Very Stiff Clay*	36	0.535	24	20	53	0.128	2.9	8.555	38
	72		24							9.324	25
8	75	*Dense sandy Silt*	75		27	--	--	0.123	--	9.694	79
	80		--							10.311	--
25	86	*Very Stiff Clay*	--	0.723	29	24	75	0.123	3.5	11.080	--
	93		--							11.850	--
	99		--	0.564						12.620	--
	105		--							13.389	--
13	112	*Dense sandy Silt*	--		25	--	--	0.125	--	14.204	--
	118		--							15.018	--
58	125	*Very stiff clay and sandy clay*	--	0.504	20	17	52	0.136	--	16.006	--
	133		--							16.994	--
	140		--							17.982	--
	147		--							18.970	--
	154		--	0.616						19.958	--
	162		--							20.946	--
	169		--							21.934	--
	176		--							22.921	--
15	181	*Dense sandy Silt*	100		20	--	--	0.131	--	23.603	105
	186		74							24.284	78
	191		>100							24.965	--
9	200	*Very Stiff Clay*	--	0.634	20	20	53	0.133	--	26.161	--

***Note that for water content, PL and LL, averages are taken.**

Type of soil	G_s
Quartz sand	2.64–2.66
Silt	2.67–2.73
Clay	2.70–2.9
Chalk	2.60–2.75
Loess	2.65–2.73
Peat	1.30–1.9

$$\gamma_{sat} = \left(\frac{1+w}{1+wG_s}\right) G_s \gamma_w \qquad ()$$

For clay and sandy silt layers, G_s values are assumed to be as 2.75 and 2.69 respectively. For instance, for the first layer:

$$\gamma_{sat} = \left(\frac{1+0.24}{1+0.24x2.75}\right) 2.75x0.0624 = 0.128 \ kip/ft^3$$

SPT-N values are directly taken from the field. Therefore, it has to be corrected by using correction factors. The table below indicates the SPT-N properties and correction factors depending on these properties.

SPT Properties			
η_H	η_B	η_S	η_R
Hammer eff	**Borehole dia**	**Sampler**	**Rodlength**
60	1.05	1	1
Safety U.S	150 mm	Standard	>10

N_{60} values are calculated by:

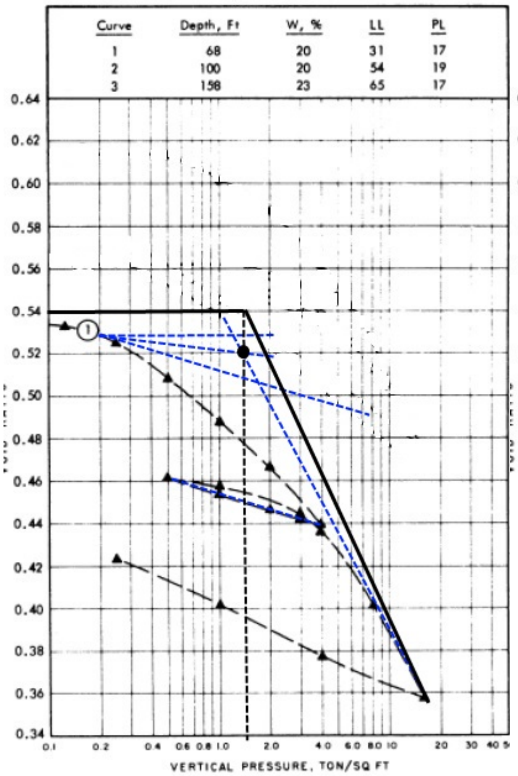
$$N_{60} = \frac{N\ x\ \eta_H\ x\ \eta_B\ x\ \eta_S\ x\ \eta_R}{60} \qquad ()$$

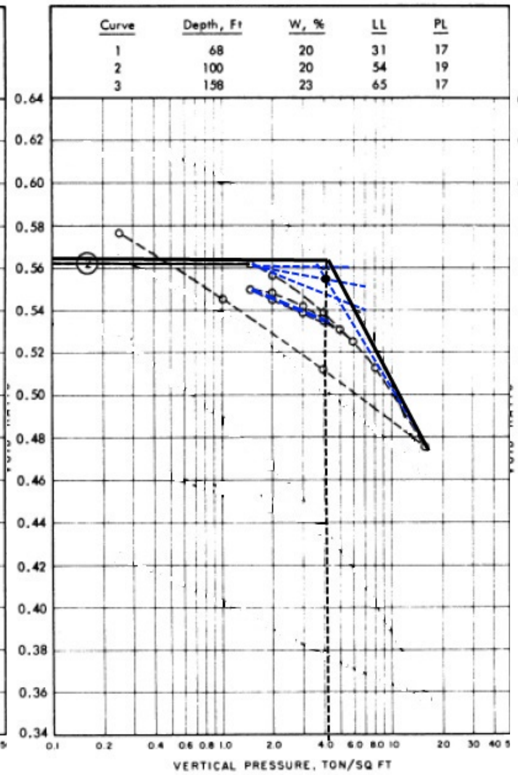
For instance;

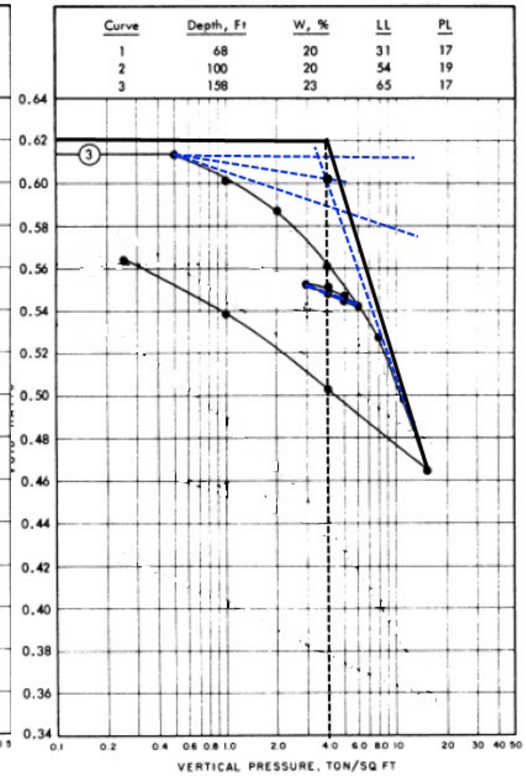
At the depth of 18 meter -> $N = 13\ blow/ft \rightarrow N_{60} = \frac{13\ x\ 60\ x\ 1.05\ x\ 1\ x\ 1}{60} = 14$

N_{60} values at different depths are calculated as above example.

B. Soil Consolidation Test Data







$$\boldsymbol{At\ a\ depth\ of\ z = 68\ ft}$$

$Fully\ Saturated\ soil\ \rightarrow e_0 = G_s\ x\ w$

$e_0 = 2.7\ x\ 0.20 = 0.54$

$\sigma'_v = 24\ x\ 0.128 + 36\ x\ 0.131 + 6\ x\ 0.128$

$\boldsymbol{\sigma'_v = 8.556\ ksf}$

$\boldsymbol{\sigma'_c = 1.5\ tsf = 3\ ksf}$

$OCR = \frac{\sigma'_c}{\sigma'_v} \rightarrow OCR = \frac{3}{8.556}$

$OCR = 0.35 < 1 \rightarrow Underconsoldiated$

$$\boldsymbol{At\ a\ depth\ of\ z = 100\ ft}$$

$Fully\ Saturated\ soil\ \rightarrow e_0 = G_s\ x\ w$

$e_0 = 2.70\ x\ 0.21 = 0.567$

$\sigma'_v = \sigma'_{v(z=68\ ft)} + 4\ x\ 0.128 + 8\ x\ 0.123$

$+ 20\ x\ 0.123$

$\boldsymbol{\sigma'_v = 12.743\ ksf}$

$\boldsymbol{\sigma'_c = 4\ tsf = 8\ ksf}$

$OCR = \frac{\sigma'_c}{\sigma'_v} \rightarrow OCR = \frac{8}{12.743}$

$OCR = 0.63 < 1 \rightarrow Underconsoldiated$

$$\boldsymbol{At\ a\ depth\ of\ z = 168 ft}$$

$Fully\ Saturated\ soil\ \rightarrow e_0 = G_s\ x\ w$

$e_0 = 2.70\ x\ 0.21 = 0.567$

$\sigma'_v = \sigma'_{v(z=100\ ft)} + 5\ x\ 0.123 + 13\ x\ 0.125$

$+ 50\ x\ 0.136$

$\boldsymbol{\sigma'_v = 21.798\ ksf}$

$\boldsymbol{\sigma'_c = 4\ tsf = 8\ ksf}$

$OCR = \frac{\sigma'_c}{\sigma'_v} \rightarrow OCR = \frac{8}{21.798}$

$OCR = 0.367 < 1 \rightarrow Underconsoldiated$

C. Deep Excavation and Mat Foundation

Bearing Capacity

Due to the hurricane loading, horizontal force acted on the building and it returns gross pressure on the foundation. However, the horizontal load acting on the foundation can be found by following equations:

$$\sigma_{gross} = \frac{P}{A} \pm \frac{Mc}{I} \; where$$

$$A = 232 \; x \; 172 = 39904 \; ft^2 \; , c = 8.5 \; ft \; , \; I_{weak} = \frac{1}{12} x \; 232 \; x \; 172^3 = 98376661 \;\; ft^3$$

$$P = vertical \; load \; acting \; on \; foundation = \; P_{building} + W_{foundation}$$

$$P = 8.45 \; x \; 39904 + 0.155 \; x \; 39904 \; x \; 8.5 = 389762 \; kip$$

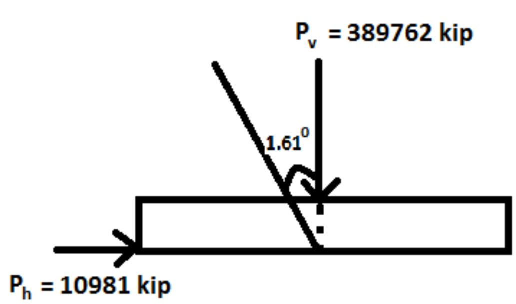
$$M = (\sigma_{gross} - \frac{P}{A}) \; x \; I/c = (9.35 - 389762/39904) \; x \; 98376661/8.5 = 4832030 \; kip.ft$$

We can assume the horizontal point load acting on the building at $\frac{2}{3}$ height of the building above the ground surface:

$$M = V \; x \; h \; where \; h = building \; height \; x \frac{2}{3} = 12 \; x \; 51 \; x \; \frac{2}{3} = 408 \; ft \; (1 \; story \; height = 12ft)$$

$$V = \frac{4832030}{408} = 10981 \; kip \; (horizontal \; load \; acting \; on \; mat \; foundation \; at \; the \; edge)$$

$$\beta = tan^{-1} = \frac{10981}{389762} = 1.61^0$$



$$F_{ci} \qquad (inclination \; factor) = \left(1 - \frac{\beta}{90^2}\right)^2$$

$$F_{ci} = \left(1 - \frac{\beta}{90^0}\right)^2 = 0.96$$

$q = 8.45\ ksf$

$\sigma'_v = 7.786\ ksf\ (vertical\ pressure\ at\ depth\ of\ z = 60\ ft)$

$q_{net} = q - \sigma'_v \quad \rightarrow q_{net} = 8.45 - 7.786 \ \rightarrow \boldsymbol{q_{net} = 0.664\ ksf}$

$c_u = 2.9\ ksf (undrained\ shear\ stress), B\ (foundation\ width) = 172\ ft,$

$L(foundation\ length) = 232\ ft, \qquad D_f(foundation\ depth) = 60\ ft$

$$q_{net(u)} = 5.14c_u\left(1 + \frac{0.195B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right)(F_{ci})$$

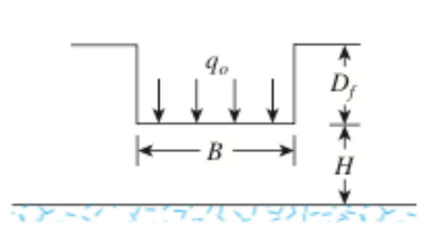
$$q_{net(u)} = 5.14\ x\ 2.9\ x\ \left(1 + \frac{0.195\ x\ 172}{232}\right)\left(1 + 0.4\frac{60}{172}\right)(0.96) \rightarrow \boldsymbol{q_{net(u)} = 18.664\ ksf}$$

$$F.S = \frac{q_{net(u)}}{q_{net(all)}} \rightarrow q_{net(all)} = \frac{18.664}{3} \rightarrow \boldsymbol{q_{net(all)} = 6.22\ ksf}$$

$\boldsymbol{q_{net(all)} > q_{net}}$

Settlement

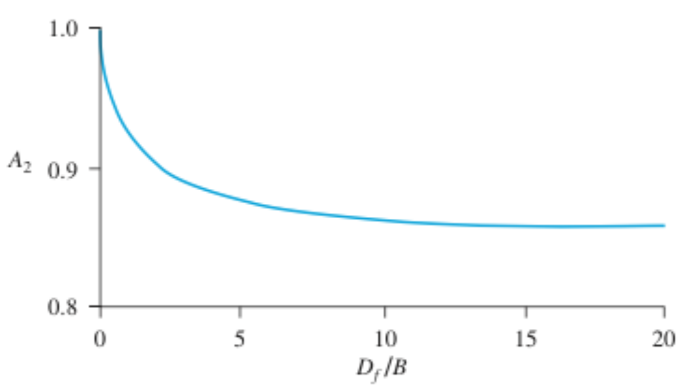
Immediate Settlement

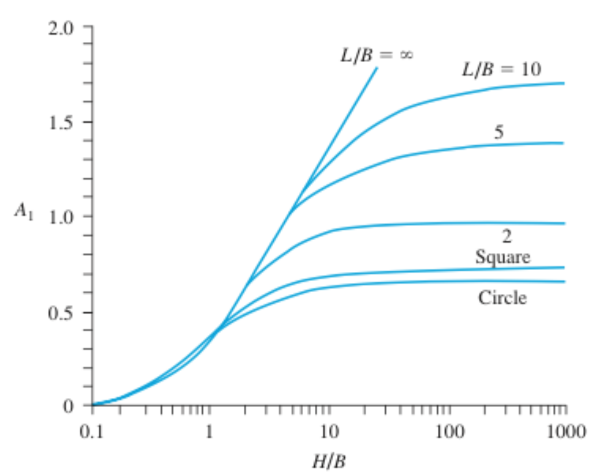


$S_e = A_1 x A_2 \frac{q_0 x B}{E_s}$

In order to find A_1 and A_2, (H =182 ft (N>100))

$D_f/B = 0.35, H/B = 1.06, L/B = 1.35$





From the graphs $\boldsymbol{A_1 = 0.35}$ & $\boldsymbol{A_2 = 0.98}$

For finding modulus of elasticity in clays,

$E_s = \beta \; x \; c_u \; where \; \beta: function \; of \; placticity \; index \; and \; OCR, \; c_u: undrained \; shear \; stress$

Before starting calculation of the OCR at a depth of 60 ft, the table below indicates ome parameters being used during the calculation.

z	e_0	W_L	LL	PL	LI	PI	c_u	σ'$_v$	p_A
(ft)	--	(%)	(%)	(%)	--	--	kip / ft^2	kip / ft^2	kip / ft^2
60	0.54	20	31	17	0.214	14	2.9	7.786	2.088

*Note that e_0, WL, LL, PL and c_u is taken from consolidation test data.

$$Plasticity\; Index\; (PI) = LL - PL \rightarrow PI = 31 - 17 \rightarrow PI = 14$$

$$Liquid\; Index\; (LI) = \frac{W_L - PL}{LL - PL} = \frac{20 - 17}{31 - 17} = 0.214$$

$$\sigma'_v = 7.786 \text{ ksf (from soil profile table)}$$

$$OCR = \frac{\sigma'_c}{\sigma'_v}$$

There are different ways for calculating σ'_c,

Stas and Kulhawy (1984)

$$\frac{\sigma'_c}{p_A} = 10^{(1.11 - 1.62 \; x \; LI)}$$

$$\sigma'_c = 2.088 \; x \; 10^{(1.11 - 1.62 \; x \; 0.214)} = 12.11 \text{ ksf}$$

$$OCR = \frac{\sigma'_c}{\sigma'_v} = \frac{12.11}{7.786} \rightarrow \boldsymbol{OCR = 1.55}$$

Kulhawy and Mayne (1990)

$$\sigma'_c = \sigma'_0 \left\{ 10^{\left(1-(2.5\ x\ LI)-\left(1.25\ x\ \log\frac{\sigma'_0}{p_A}\right)\right)} \right\}$$

$$\sigma'_c = 7.786 \left\{ 10^{\left(1-(2.5\ x\ 0.214)-\left(1.25\ x\ \log\frac{7.786}{2.088}\right)\right)} \right\} \rightarrow \boldsymbol{\sigma'_c = 4.38\ ksf}$$

$$OCR = \frac{\sigma'_c}{\sigma'_v} = \frac{4.38}{7.786} \rightarrow \ \boldsymbol{OCR = 0.56}$$

Nagaraj and Murthy(1985)

$$\sigma'_c = 10^{\left(\frac{1.22-\frac{e_0}{e_L}-0.0463 log\sigma'_0}{0.188}\right)}\ where\ e_L = \frac{LL}{100} x G_s = \frac{31}{100} x 2.75 = 0.85$$

$$\sigma'_c = 10^{\left(\frac{1.22-\frac{0.54}{0.85}-0.0463\log(7.786\ x\ 47.88)}{0.188}\right)} x\ 0.0208 \rightarrow \sigma'_c = \mathbf{1.93}\ \boldsymbol{ksf}$$

$$OCR = \frac{\sigma'_c}{\sigma'_v} = \frac{1.93}{7.786} \rightarrow \ \boldsymbol{OCR = 0.25}$$

OCR is also corralated like equation as below

$$OCR = \beta\ x\ \frac{c_{u(field)}}{\sigma'_0}$$

Mayne and Mitchell (1988)

$$\beta = \ 22\ x\ PI^{-0.48}$$

$$\beta = \ 22\ x\ 14^{-0.48} = 6.198$$

$$OCR = \ \beta\ x\ \frac{c_{u(field)}}{\sigma'_0} = 6.198\ x\ \frac{2.9}{7.786} \rightarrow \boldsymbol{OCR = 2.31}$$

Hansbo (1957)

$$\beta = \frac{222}{w(\%)} = \frac{222}{20} = 11.1$$

$$OCR = \beta\ x\ \frac{c_{u(field)}}{\sigma'_0} = 11.1\ x\ \frac{2.9}{7.786} \rightarrow \boldsymbol{OCR = 4.13}$$

Larrson (1980)

$$\beta = \frac{1}{0.08 + 0.0055\ x\ PI} = \frac{1}{0.08 + 0.0055\ x\ 14} = 6.37$$

$$OCR = \beta\ x\ \frac{c_{u(field)}}{\sigma'_0} = 6.37\ x\ \frac{2.9}{7.786} \rightarrow \boldsymbol{OCR = 2.37}$$

(ksf)	Stas and Kulhawy (1984)	Kulhawy and Mayne (1990)	Nagaraj and Murthy (1985)	Mayne and Mitchell (1988)	Hansbo (1957)	Larrson (1980)	From Consolidation Test Data	Average
$\boldsymbol{OCR}$	1.55	0.56	0.25	2.31	4.13	2.37	0.35	
$\boldsymbol{\sigma'_0}$	7.786							7.786
$\boldsymbol{\sigma'_c}$	12.11	4.38	1.93	17.98	32.15	18.45	22.24	

OCR and PI is taken as 1.86 and 14 respectively. By interpolation from the table below,

$\boldsymbol{\beta = 1397}$

Plasticity Index	β OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
<30	1500–600	1380–500	1200–580	950–380	730–300
30 to 50	600–300	550–270	580–220	380–180	300–150
>50	300–150	270–120	220–100	180–90	150–75

[a]Interpolated from Duncan and Buchignani, (1976)

$$E_s = \beta\ x\ c_u = 1397\ x\ 2.9 = 4051.3\ ksf$$

To summary;

A_1	--	0.35
A_2	--	0.98
q_0	ksf	0.664
B	ft	172
E_s	ksf	4051.3

$$S_e = A_1 x A_2 \frac{q_0 x B}{E_s}$$

$$S_e = 0.35\ x\ 0.98\ x\ \frac{0.664\ x\ 172}{4051.3}$$

$$S_e = 0.009\ ft$$

$$\boldsymbol{S_e = 0.12\ inch}$$

Consolidation Settlement

During calculation of C_c and C_s, consolidaton test data parameters are used. The table as below indicates parameters being used.

Clay Layer No	z (ft)	e_0	W_L (%)	LL (%)	PL (%)	LI	PI
1	68	0.54	20	31	17	0.214	14
2	100	0.565	20	54	19	0.029	35
3	158	0.62	23	65	17	0.125	48

There are some different methods for calculating C_c

- **Skempton (1944)**

$$C_c = 0.009(LL(\%) - 10)$$

Clay Layer 1 -> $C_c = 0.009(31 - 10) = 0.189;$

Clay Layer 2 -> $C_c = 0.009(54 - 10) = 0.396$

Clay Layer 3 -> $C_c = 0.009(65 - 10) = 0.495$

- **Renden Herrero (1983)**

$$C_c = 0.141 \ x \ {G_s}^{1.2} \ x \ \left(\frac{1 + \ e_0}{G_s}\right)^{2.38}$$

Clay Layer 1 -> $C_c = 0.141 \ x \ 2.75^{1.2} \ x \ \left(\frac{1+ 0.54}{2.75}\right)^{2.38} = 0.119$

Clay Layer 2 -> $C_c = 0.141 \ x \ 2.75^{1.2} \ x \ \left(\frac{1+ 0.565}{2.75}\right)^{2.38} = 0.124$

Clay Layer 3 -> $C_c = 0.141 \ x \ 2.75^{1.2} \ x \ \left(\frac{1+ 0.62}{2.75}\right)^{2.38} = 0.135$

- **Nagaraj and Murty (1985)**

$$C_c = 0.2343 \ x \ \left[\frac{LL(\%)}{100}\right] \ x \ G_s$$

Clay Layer 1 -> $C_c = 0.2343 \ x \ \left[\frac{31}{100}\right] \ x \ 2.75 = 0.200$

Clay Layer 2 -> $C_c = 0.2343 \ x \ \left[\frac{54}{100}\right] \ x \ 2.75 = 0.428$

Clay Layer 3 -> $C_c = 0.2343 \ x \ \left[\frac{65}{100}\right] \ x \ 2.75 = 0.419$

- **Wroth and Wood (1978)**

$$C_c = 0.5 \ x \ G_s \ x \ \left(\frac{PI(\%)}{100}\right)$$

Clay Layer 1 -> $C_c = 0.5 \ x \ 2.75 \ x \ \left(\frac{14}{100}\right) = 0.193$

Clay Layer 2 -> $C_c = 0.5 \ x \ 2.75 \ x \ \left(\frac{35}{100}\right) = 0.481$

Clay Layer 3 -> $C_c = 0.5 \ x \ 2.75 \ x \ \left(\frac{48}{100}\right) = 0.66$

- **Kulhawy and Mayne (1990)**

$$C_c = \frac{PI(\%)}{74}$$

Clay Layer 1 -> $C_c = \frac{PI(\%)}{74} = \frac{14}{74} = 0.189$

Clay Layer 2 -> $C_c = \frac{PI(\%)}{74} = \frac{35}{74} = 0.473$

Clay Layer 3 -> $C_c = \frac{PI(\%)}{74} = \frac{48}{74} = 0.649$

For C_s calculation, Kulhawy and Mayne method is used.

$$C_s \approx \frac{PI(\%)}{370}$$

The summary table is shown as below:

	C_c							C_s
	Skempton	**Renden Herrero**	**Nagaraj and Murty**	**Wroth and Wood**	**Kulhawy and Mayne**	**Consolidation Test Data**	**Average**	
Layer 1	0.189	0.119	0.200	0.193	0.189	0.173	0.177	0.038
Layer 2	0.396	0.124	0.428	0.481	0.473	0.143	0.328	0.095
Layer 3	0.495	0.135	0.419	0.66	0.649	0.246	0.434	0.130

For each layer, average values are taken for consolidation settlement calculation.

The loading due to the building is not same at every depth of soil value. The stress values due to the building loading at the mid depth of clay layer or sub layers are calculated by Boussinesq's equation.

For this foundation, the vertical stress is below the center of mat foundation is significant to calculate.

$$\Delta\sigma = q_0 I_c \ where$$

$$I_c = \frac{2}{\pi}\left[\frac{m_1 n_1}{\sqrt{1+m_1{}^2+n_1{}^2}}\frac{1+m_1{}^2+n_1{}^2}{(1+n_1{}^2)(m_1{}^2+n_1{}^2)} + sin^{-1}\left(\frac{m_1}{\sqrt{m_1{}^2+n_1{}^2}\sqrt{1+n_1{}^2}}\right)\right]$$

$$m_1 = \frac{L}{B} = \frac{172}{232} = 0.74 \qquad n_1 = \frac{z}{(\frac{B}{2})}$$

For different depth values, influence factor and corresponding stress changes are calculated as:

Z (ft)	m = L/B	n = z/(B/2)	I_c	q_{net} (ksf)	$\Delta\sigma'_0$ (ksf)	σ'_0 (ksf)	$\Delta\sigma'_0+\sigma'_0$ (ksf)
6	0.74	0.05	0.96	0.664	0.641	7.89	8.53
26.25		0.23	0.85		0.564	9.15	9.71
38.75		0.33	0.78		0.520	9.91	10.43
65.25		0.56	0.67		0.443	11.64	12.08
79.75		0.69	0.62		0.409	12.71	13.12
94.25		0.81	0.57		0.380	13.78	14.16
108.75		0.94	0.54		0.356	14.85	15.21

For all clay layers, since $\sigma'_0 + \Delta\sigma' < \sigma'_c$, the formula as below is used and consolidation settelement is calculated as table below:

$$S_c = \frac{C_s H_c}{1+e_0} log \frac{\sigma'_0 + \Delta'\sigma_{avg}}{\sigma'_0}$$

Actual z	Layer	Below found. z	$\Delta\sigma'_0+\sigma'_0$ (ksf)	σ'_c (ksf)	H_c ft	e_0 (G_s x w)	C_s	C_c	S_c (ft)	S_c (in)	Total (in)
66	Very Stiff Clay	6	8.53	13.42	12	0.535	0.038	0.177	0.0100	0.1204	1.294
86.25	Very Stiff Clay	26.25	9.71	21.37	12.5	0.723	0.095	0.328	0.0178	0.2138	
98.75		38.75	10.43	21.37	12.5	0.564	0.095	0.328	0.0168	0.2016	
125.25	Very stiff clay and sandy clay	65.25	12.08	21.98	14.5	0.504	0.130	0.434	0.0203	0.2433	
139.75		79.75	13.12	21.98	14.5	0.504	0.130	0.434	0.0172	0.2063	
154.25		94.25	14.16	21.98	14.5	0.616	0.130	0.434	0.0138	0.1650	
168.75		108.75	15.21	21.98	14.5	0.616	0.130	0.434	0.0120	0.1437	

D. PILE FOUNDATION

Before end bearing capacity calculation, friction angle of sand layer at which pile end exists is calculated by 3 different approaches and then average of these values are taken.

By Peck, Hanson, and Thurburn (1974);

$$\emptyset'(\deg) = 27.1 + 0.3\,(N_1)_{60} - 0.00054((N_1)_{60})^2$$

$$For\ instace, at\ z = 28.5\ ft \rightarrow \emptyset' = 27.1 + 0.3\ \times 29.5 - 0.00054(29.5)^2 \rightarrow \emptyset' = 35.48^0$$

By Schmertmann (1975);

$$\emptyset' = tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_0}{p_A}\right)}\right]^{0.34}$$

$$For\ instace, at\ z = 28.5\ ft \rightarrow \emptyset' = tan^{-1}\left[\frac{39.9}{12.2 + 20.3\left(\frac{3.7}{2.088}\right)}\right] \rightarrow \emptyset' = 42.85^0$$

By Hatanaka and Uchida (1996);

$$\emptyset'(\deg) = \sqrt{20(N_1)_{60}} + 20$$

$$For\ instace, at\ z = 28.5\ ft \rightarrow \emptyset'(\deg) = \sqrt{20 \times 29.5} + 20 \rightarrow \emptyset' = 44.28^0$$

Depth (ft)	Description	N	γ_{sat}	σ'_0	N_{60}	$(N_1)_{60}$	ϕ	ϕ	ϕ	ϕ
		blow/ft	kip / ft^3	kip / ft^2	blow/ft	blow/ft	Peck, Hanson and Thorburn	Schemertmann	Hatanaka and Uchida	Average
28.5	Dense sandy Silt and silty Sand with clay layer	38	0.131	3.7	39.9	29.5	35.5	42.9	44.3	40.9
33		31		4.3	32.6	22.3	33.5	39.9	41.1	38.2
37.5		25		4.8	26.3	16.9	32.0	36.8	38.4	35.7
42		28		5.4	29.4	17.8	32.3	37.0	38.9	36.1
46.5		46		6.0	48.3	27.8	35.0	41.0	43.6	39.9
51		22		6.6	23.1	12.7	30.8	33.3	35.9	33.4
55.5		33		7.2	34.7	18.3	32.4	36.4	39.1	36.0
60		50		7.5	52.5	27.1	34.8	40.0	43.3	39.4

Pile Type 1

End Bearing Capacity

- ***Meyerhof Method***

For piles in sand, c' = 0,

$$Q_p = A_p q_p = A_p q' N^*_q$$

N^*_q changes with ϕ' shown in figure.

Q_p should not exceed the limiting value $A_p q_l$; that is:

$$Q_p = A_p q' N^*_q \leq A_p q_l \qquad where$$

$$q_l = 0.5\, p_A N^*_q tan\emptyset' \qquad where$$

$$p_A = \ atmospheric\ pressure$$

$$\emptyset' = effective\ soil\ friction\ angle\ of\ the\ bearing\ stratum$$

$$q' = \ Oveburden\ pressure\ at\ x = 58\ ft \rightarrow q' = 7.52\ ksf$$

$$A_p = \frac{\pi\, D^2}{4}\ where\ D = 2.5\ ft\ \rightarrow A_p = \ 4.91\ ft^2$$

$$\emptyset' = 39\ (at\ the\ end\ of\ pile\ z = 58\ ft) \rightarrow N^*_q = 276$$

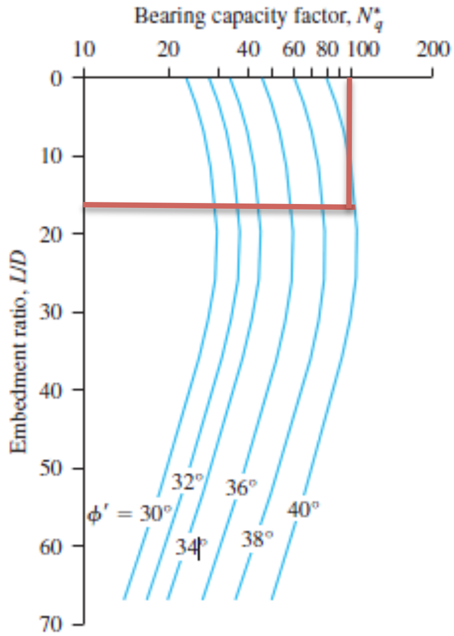
$$Q_p = A_p q_p = A_p q' N^*_q = 4.91 \times 7.52 \times 276 \rightarrow Q_p = 10194.1\ kip$$

$$A_p q_l = 4.91 \times 0.5 \times 2 \times 276 \times \tan(39) \rightarrow A_p q_l = 1145.4\ kip$$

$$Q_p > A_p q_l \rightarrow Therefore,\ \boldsymbol{Q_p = 1145.4\ kip}$$

Soil friction angle, ϕ (deg)	N^*_q
20	12.4
21	13.8
22	15.5
23	17.9
24	21.4
25	26.0
26	29.5
27	34.0
28	39.7
29	46.5
30	56.7
31	68.2
32	81.0
33	96.0
34	115.0
35	143.0
36	168.0
37	194.0
38	231.0
39	276.0
40	346.0
41	420.0
42	525.0
43	650.0
44	780.0
45	930.0

- ***Coyle and Castello Method***



$$Q_p = A_p q' N^*_q$$

$$\frac{L}{D} = \frac{40}{2.5} = 16 \rightarrow N^*_q = 100$$

$$q' = \ Oveburden\ pressure\ at\ x = 58\ ft \rightarrow q' = 7.52\ ksf$$

$$A_p = \frac{\pi\ D^2}{4}\ where\ D = 2.5\ ft\ \rightarrow A_p = \ 4.91\ ft^2$$

$$Q_p = 4.91\ x\ 7.52\ x\ 100 \rightarrow \boldsymbol{Q_p = 3693.5\ kip}$$

- ***Vesic Method***

$$Q_p = \ A_p q_p = A_p \bar{\sigma}'_o N^*_\sigma\ where$$

$$\bar{\sigma}'_o = \left[\frac{1 + 2(1 - sin\emptyset')}{3}\right] q' = \left[\frac{1 + 2(1 - sin39)}{3}\right] 7.52 \rightarrow \bar{\sigma}'_o = 4.37\ ksf$$

$For\ N^*_\sigma\ ;$

$$\frac{E_s}{p_A} = m\ where\quad m = \begin{cases} 100 \text{ to } 200 (\text{loose soil}) \\ 200 \text{ to } 500 (\text{medium dense soil}) \\ 500 \text{ to } 1000 (\text{dense soil}) \end{cases}$$

$$Soil\ is\ dense\ sand: \frac{E_s}{2.088} = 1000\ \rightarrow E_s = 2088\ ksf$$

$$\mu_s = 0.1 + 0.3\left(\frac{\emptyset' - 25}{20}\right) = 0.1 + 0.3\left(\frac{39 - 25}{20}\right) \rightarrow \mu_s = 0.31$$

$$I_r = \frac{E_s}{2(1 + \mu_s) q' tan\emptyset'} = \frac{2088}{2(1 + 0.31)\ x\ 7.52\ x\ tan39} \rightarrow I_r = 130.8$$

$$\Delta = 0.05\left(1 - \frac{\emptyset' - 25}{20}\right)\left(\frac{q'}{p_A}\right) = 0.05\left(1 - \frac{39 - 25}{20}\right)\left(\frac{7.52}{2.088}\right) = 0.0054$$

$$I_r = \frac{I_r}{1 + I_r\Delta} = \frac{130.8}{1 + 130.8\ x\ 0.0054} = 76.6$$

	I_{rr}									
ϕ'	10	20	40	60	80	100	200	300	400	500
25	12.12	15.95	20.98	24.64	27.61	30.16	39.70	46.61	52.24	57.06
26	13.18	17.47	23.15	27.30	30.69	33.60	44.53	52.51	59.02	64.62
27	14.33	19.12	25.52	30.21	34.06	37.37	49.88	59.05	66.56	73.04
28	15.57	20.91	28.10	33.40	37.75	41.51	55.77	66.29	74.93	82.40
29	16.90	22.85	30.90	36.87	41.79	46.05	62.27	74.30	84.21	92.80
30	18.24	24.95	33.95	40.66	46.21	51.02	69.43	83.14	94.48	104.33
31	19.88	27.22	37.27	44.79	51.03	56.46	77.31	92.90	105.84	117.11
32	21.55	29.68	40.88	49.30	56.30	62.41	85.96	103.66	118.39	131.24
33	23.34	32.34	44.80	54.20	62.05	68.92	95.46	115.51	132.24	146.87
34	25.28	35.21	49.05	59.54	68.33	76.02	105.90	128.55	147.51	164.12
35	27.36	38.32	53.67	65.36	75.17	83.78	117.33	142.89	164.33	183.16
36	29.60	41.68	58.68	71.69	82.62	92.24	129.87	158.65	182.85	204.14
37	32.02	45.31	64.13	78.57	90.75	101.48	143.61	175.95	203.23	227.26
38	34.63	49.24	70.03	86.05	99.60	111.56	158.65	194.94	225.62	252.71
39	37.44	53.50	76.45	94.20	109.24	122.54	175.11	215.78	250.23	280.71
40	40.47	58.10	83.40	103.05	119.74	134.52	193.13	238.62	277.26	311.50
41	43.74	63.07	90.96	112.68	131.18	147.59	212.84	263.67	306.94	345.34
42	47.27	68.46	99.16	123.16	143.64	161.83	234.40	291.13	339.52	382.53
43	51.08	74.30	108.08	134.56	157.21	177.36	257.99	321.22	375.28	423.39
44	55.20	80.62	117.76	146.97	172.00	194.31	283.80	354.20	414.51	468.28
45	59.66	87.48	128.28	160.48	188.12	212.79	312.03	390.35	457.57	517.58

By Interpolation;

$N^*_\sigma = 106.7 \ \rightarrow Q_p = 4.91\ x\ 4.37\ x\ 106.7 \rightarrow \ \boldsymbol{Q_p = 2287.13\ kip}$

- ***Basis of field observation, Meyerhof (1976)***

$$q_p = 0.4\ x\ p_A x N_{60} x \frac{L}{D} \leq 4\ x\ p_a x N_{60}$$

$N_{60}\ values\ are\ taken\ between\ 10xD\ above\ and\ 4D\ below\ pile\ end\ and\ average\ of\ these\ values\ are\ taken\ and\ is\ found\ as\ (N_{60})_{avg} = 36\ blow/ft$

$$q_p = 0.4\ x\ p_A x N_{60} x \frac{L}{D} = 0.4\ x\ 2.088\ x\ 36\ x\ \frac{40}{2.5} \rightarrow q_p = 481.2\ ksf$$

$$4\ x\ p_a x N_{60} = \ 4\ x\ 2.088\ x\ 36 = 300.75\ ksf$$

Due to the condition: $q_p\ = 0.4\ x\ p_A x N_{60} x \frac{L}{D} \leq 4\ x\ p_a x N_{60}\ \rightarrow q_p\ = 300.75\ ksf$

$$Q_p\ = \ A_p\ x\ q_p\ = 4.91\ x\ 300.75 \rightarrow \boldsymbol{Q_p\ = 1476.3\ kip}$$

Skin Resistance

- ***Clay Layer (z = 18 to 24 ft)***

  - **α Method**

$f\ = \ \alpha\ x\ c_u\ where$

$c_u\ at\ this\ clay\ layer\ is\ 2.2\ ksf$

$Sladen\ \alpha = C\left(\frac{\bar{\sigma}'_o}{c_u}\right) where\ C \geq 0.5\ for\ driven\ piles.$

$Assume\ C\ = 0.5\ \ and\ \bar{\sigma}'_o\ = \left(\frac{6\ x\ \sigma + 6\ x \frac{\Delta\sigma}{2}}{2}\right) = 2.69\ ksf$

$\alpha = C\left(\frac{\bar{\sigma}'_o}{c_u}\right) = 0.5\left(\frac{2.69}{2.2}\right) = 0.55\ (Sladen)$

$\frac{c_u}{p_a}$	α
≤ 0.1	1.00
0.2	0.92
0.3	0.82
0.4	0.74
0.6	0.62
0.8	0.54
1.0	0.48
1.2	0.42
1.4	0.40
1.6	0.38
1.8	0.36
2.0	0.35
2.4	0.34
2.8	0.34

$\frac{c_u}{p_A} = \frac{2.2}{2.088} = 1.1$

$Terzaghi, Peck\ and\ Mesri\ \alpha = 0.45\ (from\ table)$

$Perimeter, p\ = \ \pi\ x\ D = \pi\ x\ 2.5 = 7.85\ ft$

$f\ = \ 0.55\ x\ 2.2\ \ = 1.21\ ksf\ (Sladen) \rightarrow Q_s = f\ x\ p\ x\ L =$

$1.21\ x\ 7.85\ x\ 6 = 56.76\ kip$

$f\ = \ 0.45\ x\ 2.2\ \ = 0.99\ ksf\ (Terzaghi) \rightarrow Q_s = f\ x\ p\ x\ L = 0.99x\ 7.85\ x\ 6 = 46.65\ kip$

$Average \rightarrow \boldsymbol{Q_s = 51.71\ kip\ for\ clay}$

- **λ method**

Embedment length, *L* (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

$$Q_s = p\ x\ L\ x\ f_{av}\ where$$

$$f_{av} = \lambda(\bar{\sigma}'_o + 2c_u)\ where\ \bar{\sigma}'_o = 2.69\ ksf, c_u = 2.2\ ksf$$

Embedment length, L = 6 ft = 1.8 m

By interpolation, λ = 0.44

$$f_{av} = \lambda(\bar{\sigma}'_o + 2c_u) = 0.44\ (2.69 + 2x2.2) = 3.13\ ksf$$

$$Q_s = p\ x\ L\ x\ f_{av} = 7.85\ x\ 6\ x\ 3.13 = 147.4\ kip$$

To summary for skin resistance of clay layer:

Method	Q_s (kip)	Average (kip)
α method	51.71	99.54
λ method	147.37	

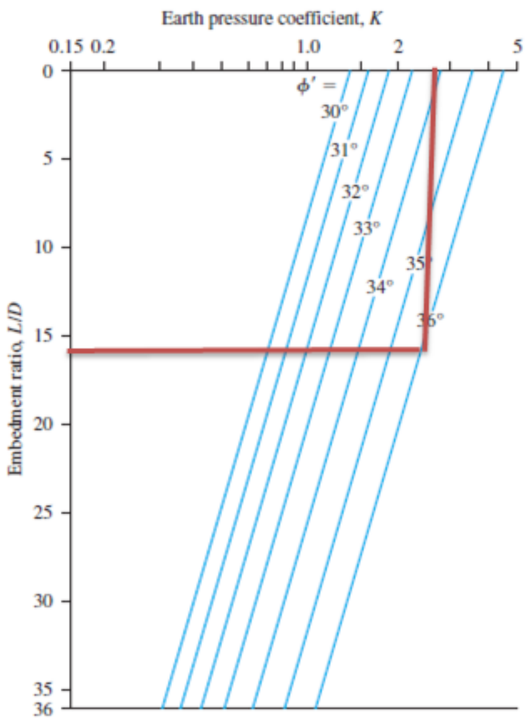
- *Sand Layer (z = 24 to 58 ft)*

During calculations, the average of ϕ values is used which is 39^0. Since the pile foundation does not start at the sand layer, and so unit frictional resistance diagram does not start at 0, a conservative estimate (L' = 15D) is not applied and assume frictional resistance increasing with depth.

$$f = K\ x\ \sigma'_o\ x\ tan\delta$$

$$K = average\left(1 - sin39, 1.4\ x\ (1 - sin39)\right) \rightarrow K = 0.77$$

Pile type	*K*
Bored or jetted	$\approx K_o = 1 - \sin\phi'$
Low-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.4K_o = 1.4(1 - \sin\phi')$
High-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.8K_o = 1.8(1 - \sin\phi')$



The other way of obtaining K is using figure:

L/D = 40/2.5 = 16;

K = 2.8 from figure

Average of K values is (0.77 + 2.8)/2 = 1.79

$$\delta = 0.8\, \emptyset' = 0.8\ x\ 39 \rightarrow \delta = 31.2^0$$

Layer	z	σ'₀	f_s	f_s	A_s	Q_s
ft	ft	ksf	ksf	ksf	ksf	kip
34	24	3.08	3.33	5.65	267.0354	**1507.85!**
	58	7.37	7.97			

- **Field observations and correlation with SPT**

Meyerhof: for high displacement piles

$$f_{av} = 0.02\ x\ p_A\ x\ (\overline{N_{60}}) = 0.02\ x\ 2.088\ x\ 36 \rightarrow f_{av} = 1.5\ ksf$$

$$Q_s = f_{av}\ x\ p\ x\ L = 1.5\ x\ 7.85\ x\ 34 = 399.6\ kip \rightarrow \boldsymbol{Q_s = 399.6\ kip}$$

Briaud:

$$f_{av} = 0.224\ x\ p_A\ x\ (\overline{N_{60}})^{0.29} = 0.02\ x\ 2.088\ x\ 36^{0.29} \rightarrow f_{av} = 1.32\ ksf$$

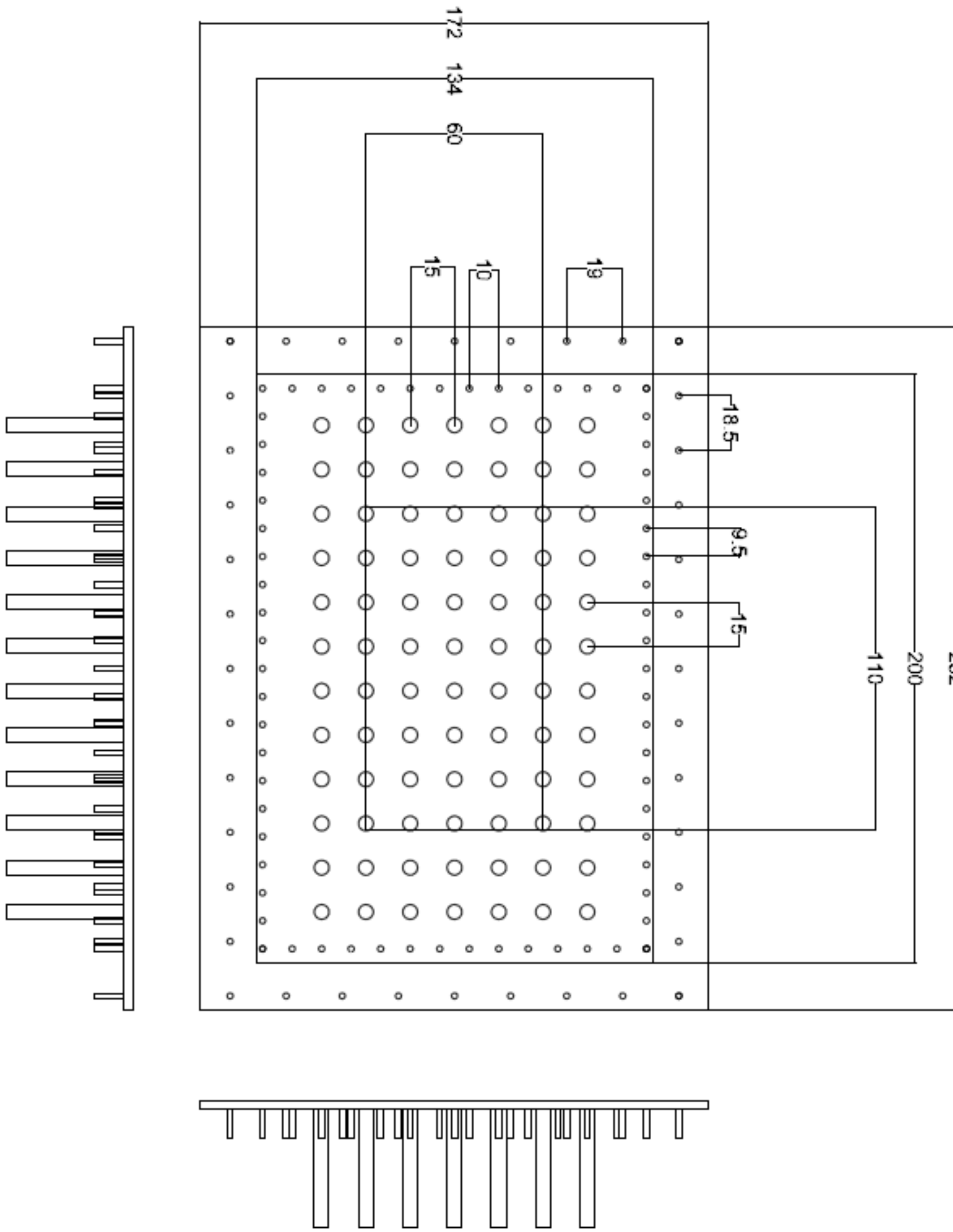
$$Q_s = f_{av}\ x\ p\ x\ L = 1.32\ x\ 7.85\ x\ 34 = 352.6\ kip \rightarrow \boldsymbol{Q_s = 352.6\ kip}$$

Average:

$$\boldsymbol{Q_s} = \left(\frac{\mathbf{399.6 + 352.6}}{\mathbf{2}}\right) \rightarrow \boldsymbol{Q_s = 376.1\ kip}$$

E. FOUNDATION LAYOUT

All dimensions are in ft.



F. LATERALLY LOADED PILE

The lateral resistance of single pile is calculated by Broom's Method.

There are two type of pile used for design and for calculation of lateral resistance of each type, each of them is determined whether short or long pile:

$$T = \sqrt{\frac{EI}{n_H}} \;\; \textbf{if} \;\; \frac{L}{T} \geq 5 \rightarrow long\ pile\ \textbf{else}\ \rightarrow short\ pile\ where\ n_H = 10000\ kN/m^3 (36.84\ lb/in^3\ )$$

Soil	n_h kN/m³
Dry or moist sand	
Loose	1800–2200
Medium	5500–7000
Dense	15,000–18,000
Submerged sand	
Loose	1000–1400
Medium	3500–4500
Dense	9000–12,000

$$Modulus\ of\ elasticity\ of\ piles\ is\ assumed\ to\ be\ 4351132\frac{lb}{in^2}\ (30\ GPa)$$

$$For\ pile\ type\ 1\ \rightarrow I = \frac{1}{64} x\ \pi\ x\ D^4 = \frac{1}{64} x\ \pi\ x\ 2.5^4 = 1.92\ ft^4$$

$$For\ pile\ type\ 2\ \rightarrow I = \frac{1}{64} x\ \pi\ x\ D^4 = \frac{1}{64} x\ \pi\ x\ 1^4 = 0.05\ ft^4$$

$$For\ pile\ type\ 1 \rightarrow\ T = \left[\frac{4351132\ x\ 1.92}{36.84}\right]^{0.2} = 7.16\ ft$$

$$For\ pile\ type\ 2 \rightarrow\ T = \left[\frac{4351132\ x\ 0.05}{36.84}\right]^{0.2} = 2.9\ ft$$

$$For\ pile\ type\ 1 \rightarrow \frac{L}{T} = 5.6\ \rightarrow \boldsymbol{Long\ Pile}$$

$$For\ pile\ type\ 2 \rightarrow \frac{L}{T} = 2.9 \rightarrow \boldsymbol{Short\ Pile}$$

To summary;

	Unit	Pile Type 1	Pile Type 2
D	ft	2.5	1
L	ft	40	10
I	ft^4	1.92	0.05
	in^4	39760.78	1017.88
E	lb/in^2	4351132.072	4351132.072
T	in	86	41
	ft	7.16	3.44
L/T	--	5.6	2.9
Situation	Restrained Pile	Long Pile	Shor tPile

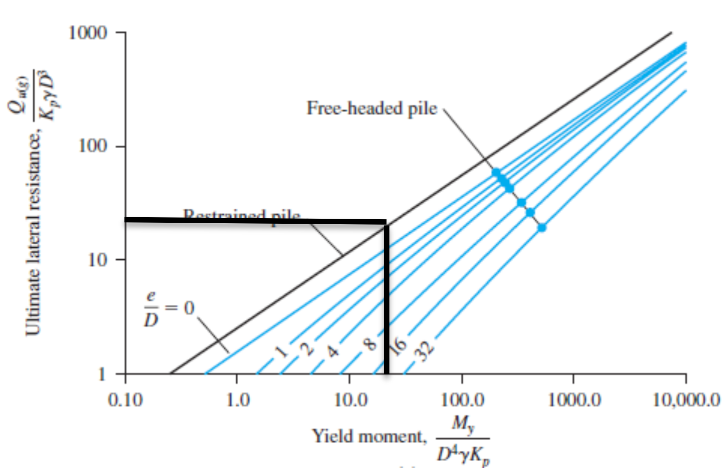
Pile Type 1

$$I = \frac{1}{64} x\ \pi\ x\ D^4 = \frac{1}{64} x\ \pi\ x\ 2.5^4 = 1.92\ ft^4 = 39760\ in^4$$

$$Assume\ yield\ strength\ of\ pile\ F_y = 3500\ psi;\ Diameter = 30\ inch$$

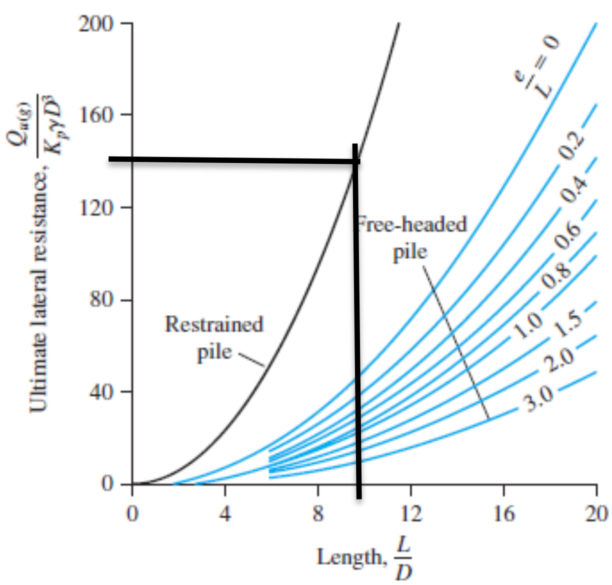
$$Yield\ Moment\ M_y = \left[\frac{I_p}{\frac{D}{2}}\right] x\ F_y = \left[\frac{39760}{\frac{30}{2}}\right] x\ 3500 = 9277515\ lb.in$$

$$\frac{M_y}{D^4 x\ \gamma\ x\ K_p} = \frac{M_y}{D^4 x\ \gamma\ x\ tan^2(45 + \emptyset/2)} = \frac{9277515}{30^4 x\ 0.075\ x\ tan^2(45 + 40/2)} = 33$$



$$\frac{Q_{u(g)}}{D^3 x\, \gamma\, x\, K_p} = 30 \;\rightarrow Q_{u(g)} = 30\ x\ 30^3 x\ 0.075\ x\ tan^2\left(45 + \frac{40}{2}\right)/1000 \rightarrow \boldsymbol{Q_{u(g)} = 282\ kip}$$

Pile Type 2



$\frac{L}{D} = 10$

$$\frac{Q_{u(g)}}{D^3 x\, \gamma\, x\, K_p} = 140$$

$$\rightarrow Q_{u(g)} = 140\ x\ 1^3 x\ 0.131\ x\ tan^2\left(45 + \frac{40}{2}\right)$$

$$\rightarrow \boldsymbol{Q_{u(g)} = 84\ kip}$$

At the end;

	# of piles	$Q_{u(g)}$ (kip)	Total $Q_{u(g)}$ (kip)	Q_h (kip)
Q_{core}	85	282	24013	10981
$Q_{extwall}$	64	84	5337	
$Q_{garwall}$	38	84	3228	
		Sum (kip)	32578	

G. GROUP EFFECT OF PILES

- Settlement

Immediate Settlement

Elastic settlement of group piles can be calculated according to empirical relation for Meyerhof (1976);

$$s_{g(e)}(mm) = \frac{0.96\ x\ q\ x\ \sqrt{B_g}\ x\ I}{N_{60}}$$

$$The\ averages\ of\ N_{60}\ below\ the\ pile\ foundation\ is\ found\ as\ 63.$$

$$B_g = (n_1 - 1)\ x\ d + D = (7-1)\ x\ 15 + 2.5 = 92.5\ ft = 28.194\ meter$$

$$L_g = (n_2 - 1)\ x\ d + D = (11-1)\ x\ 15 + 2.5 = 167.5\ ft = 51.054\ meter$$

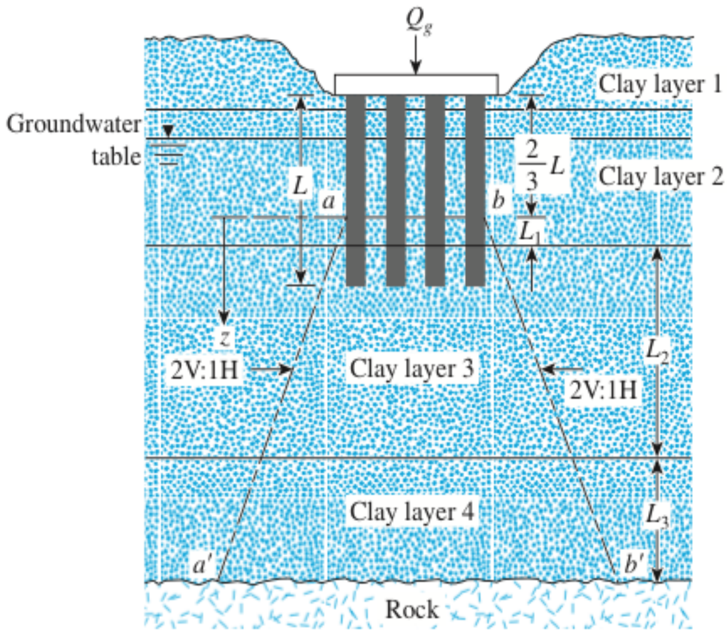
$$d\ = 15\ ft = 4.572\ meter\ , D = 2.5\ ft = 0.762\ meter;\ Q_{core} = 455053\ \mathrm{kN}$$

$$q = \frac{Q_{core}}{B_g\ x\ L_g} = \frac{455053}{28\ x\ 51} = 314.2\ kN/m^2; I = 1 - \frac{L}{8\ x\ B_g}\ =\ 1 - \frac{40}{8\ x\ 95} = 0.95;$$

$$s_{g(e)} = \frac{0.96\ x\ q\ x\ \sqrt{B_g}\ x\ I}{N_{60}} = \frac{0.96\ x\ 314.2\ x\ \sqrt{28.194}\ x\ 0.95}{N_{60}} = 24\ mm = 0.94\ inch$$

Consolidation Settlement

The soil properties does not change for calculating consolidation settlement of group pile. In other words, soil parameters are as same as the parameters that are used for calculating consildation settelement dor mat foundation. Changing paramater is starting point of spreading load (2:1 method); in other words, change in stress due to the loading and loading itself change.



The situation at the figure is not similar to project. But the figure indicates at where uniform loading begins to spread by 2:1 method. For the case, length of piles is 40 ft and so loading starts at a depth of 45 (2/3*40 +15(excavation) + 3 (slab thickness)) ft below from the ground.

$$q_{core} = \frac{Q_g}{L_g B_g} = \frac{102300}{167.5\ x\ 92.5} = 6.6\ ksf;\ q_{excavation} = 0.131\ x\ 15 = 1.965\ ksf;\ q_{net} = 4.64\ ksf$$

Actual z (ft)	Layer	z (ft)	qnet (ksf)	Δσ'0 (ksf)	σ'0 (ksf)	Δσ'0+σ'0 (ksf)	σ'c (ksf)	Hc ft	e0 (Gs x w)	Cs	Cc	Sc (ft)	Sc (in)	Total (inch)
66	Very Stiff Clay	21	4.64	3.4	7.9	11.2	13.4	12	0.535	0.038	0.176	0.046	0.547	
86.25	Very Stiff Clay	41.25	4.64	2.6	9.1	11.7	21.4	12.5	0.723	0.095	0.325	0.074	0.886	
98.75		53.75	4.64	2.2	9.9	12.1	21.4	12.5	0.564	0.095	0.325	0.066	0.797	
125.25	Very stiff clay and sandy clay	80.25	4.64	1.7	11.6	13.3	22.0	14.5	0.504	0.130	0.431	0.073	0.878	**4.808575**
139.75		94.75	4.64	1.5	12.7	14.2	22.0	14.5	0.504	0.130	0.431	0.059	0.710	
154.25		109.25	4.64	1.3	13.8	15.1	22.0	14.5	0.616	0.130	0.431	0.045	0.541	
168.75		123.75	4.64	1.1	14.9	16.0	22.0	14.5	0.616	0.130	0.431	0.037	0.449	

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