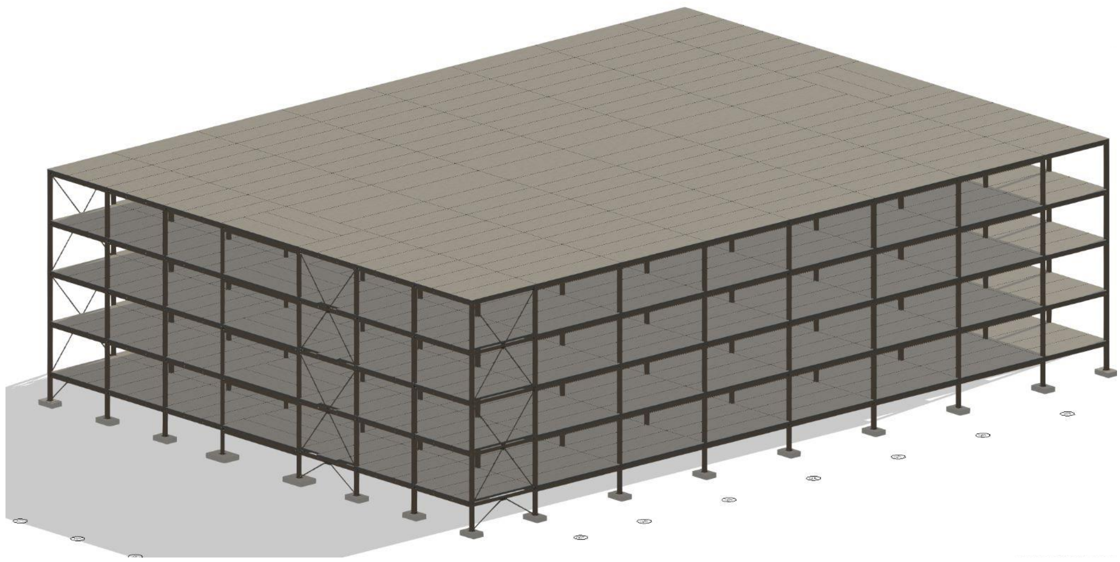
The University of Manchester

CIVL31001 Design 3: Geotechnics

Concepts and Options for the Foundation of Building A



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1. Soil Profile

In this report, the Normalized SBT Chart was used to identify the soil type of each layer. Calculations were performed to obtain the value of the normalized cone resistance (Q_{tn}) and normalized friction ratio (F_r) which were then used to find the zone number of each soil layer. The full results of each soil layer type can be found in the Appendix. However, a more thorough calculation is also described below to allow better understanding on how the soil type was identified.

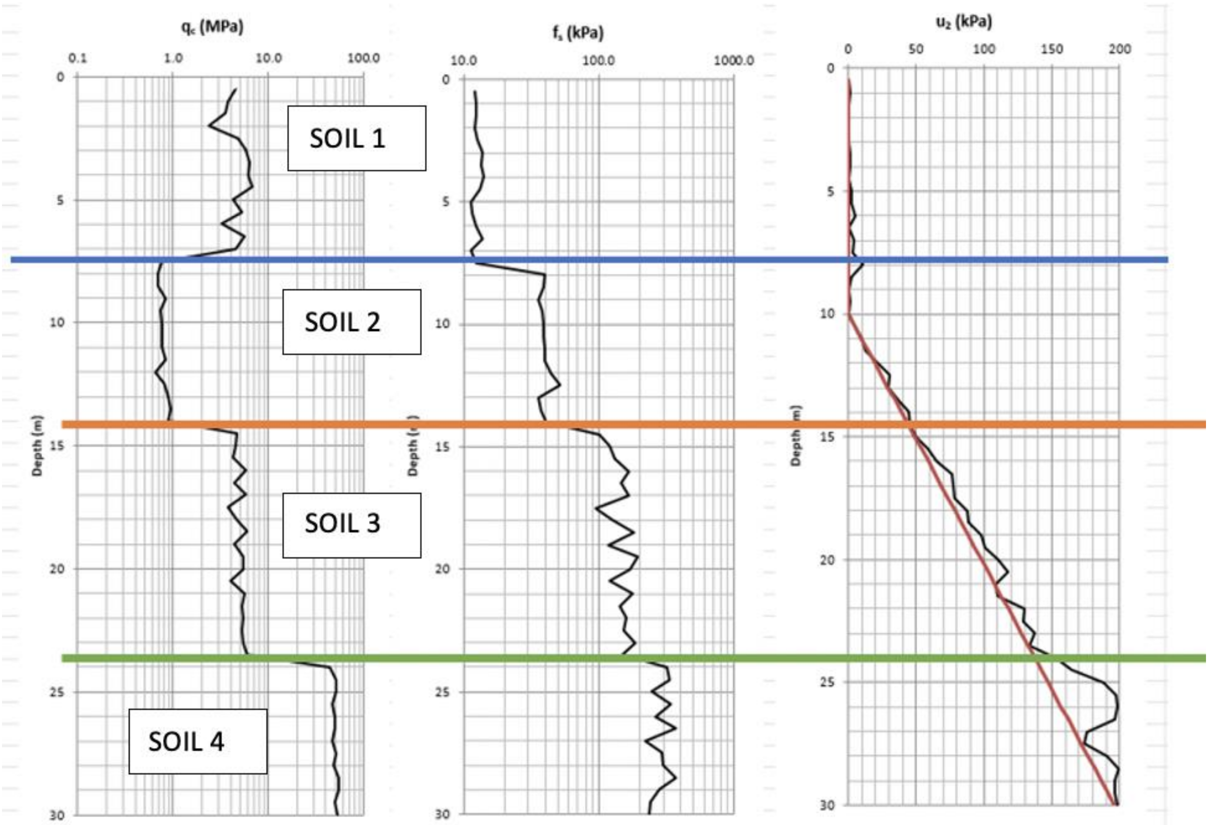


Figure 1.1: Dividing Data into Multiple Sections

As can be seen from Figure 1.1, the given data in excel is divided into multiple sections to allow easier soil type identification. This part of the report discusses how to obtain the soil type for soil section 1 in which the method can also be used to find soil type in another section (e.g., soil 2, soil 3 and soil 4).

First, the cone resistance (q_c) from the excel data is converted from MPa to kPa through multiplication by 1000. This was done to ensure uniform units are used throughout the calculation. After that, the data of u_2 was extracted from chart provided in excel as shown in Table 1 below. (Soil 1 reaches a depth of 7.5m).

Table 1: Values of u_2 for Soil Section 1

u2 (kPa)
0
0
0
0
0
0
0
0
0
0
0
0
0
0
0

Then, the corrected cone resistance (q_t) can be calculated as below

$$q_t = q_c + u_2(1 - a)$$

Where:

q_c : $cone\ resistance\ (kPa)$

u_2 : $water\ pressure\ at\ base\ of\ sleeve\ (kPa)$

a : $net\ area\ ratio\ from\ laboratory\ calibration\ with\ typical\ value\ of\ 0.70 - 0.85$

The chosen value of net area ratio is 0.775

The results are shown in Table 2 below:

Table 2: Results of q_c and q_t in Soil Section 1

qc (kPa)	qt (kPa)
4500	4500
3800	3800
3500	3500
2400	2400
4800	4800
5700	5700
6300	6300
6100	6100
6800	6800
4200	4200
5300	5300
3200	3200
5600	5600
4500	4500
770	770

Then, the friction ratio (R_f) can be calculated using the formula below:

$$R_f = \frac{f_s}{q_t} \times 100\%$$

Where:

R_f : $friction\ ratio$

f_s : $sleeve\ friction\ (kPa)$

q_t : $corrected\ cone\ resistance\ (kPa)$

The results of R_f are shown in Table 3 below:

Table 3: Results of R_f in Soil Section 1

Rf
0.270
0.326
0.351
0.504
0.266
0.243
0.215
0.234
0.195
0.272
0.218
0.391
0.246
0.256
1.623

Furthermore, the unit weight of the soil (γ_s) can be found by using the formula below:

$$\frac{\gamma_s}{\gamma_w} = 0.27\log(R_f) + 0.36\log\left(\frac{q_t}{P_a}\right) + 1.236$$

Where:

γ_w: $unit\ weight\ of\ water\ \left(\frac{kN}{m^2}\right)$

R_f : $friction\ ratio$

P_a : $atmospheric\ pressure\ (100\ kPa)$

q_t : $corrected\ cone\ resistance\ (kPa)$

Hence, the results can be seen in Table 4 as presented below:

Table 4: Results of Unit Weight of Soil Section 1

γ_s (kN/m3)
16.778
16.734
16.692
16.525
16.861
17.023
17.037
17.085
17.040
16.679
16.781
16.676
17.011
16.712
16.119

Hence, this unit weight can be used to obtain the total vertical stress (σ_{vo}) and effective vertical stress (σ'_{vo}) using the below formula:

$$\sigma_{vo} = \gamma_s \times D$$

$$\sigma'_{vo} = \sigma_{vo} - u$$

Where:

σ_{vo} : $total\ vertical\ stress\ (kPa)$

γ_s : $unit\ weight\ of\ soil\ \left(\frac{kN}{m^2}\right)$

D : $depth\ of\ soil\ (m)$

σ'_{vo} : $effective\ vertical\ stress\ (kPa)$

u : $pore\ water\ pressure\ (kPa)$

The results of total and effective vertical stress for every depth in soil section 1 are provided in Table 5 below:

Table 5: Results of σ_{vo} and σ'_{vo} of Soil Section 1

σ_{vo} (kPa)	σ'_{vo} (kPa)
8.389	7.889
16.734	15.734
25.039	24.839
33.049	32.749
42.152	42.052
51.069	50.769
59.628	58.428
68.338	67.338
76.681	76.181
83.393	81.393
92.295	90.295
100.058	95.058
110.572	110.472
116.982	112.982
120.896	117.696

The final step of the calculation is to compute Q_t and F_r which will then be plotted in the Normalized SBT Chart. The formula for Q_t and F_r is shown below:

$$Q_t = (q_t - \sigma_{vo})/\sigma'_{vo}$$

$$F_r = [\frac{f_s}{q_t - \sigma_{vo}}]100\%$$

The final results are shown in Table 6 below:

Table 6: Results of Q_t and F_r in Soil Section 1

Qt	Fr (%)
569.345	0.271
240.451	0.328
139.902	0.354
72,.74	0.511
113.142	0.269
111.267	0.245
106.805	0.217
89.573	0.237
88.255	0.197
50.577	0.278
57.675	0.222
32.611	0.403
49.691	0.251
38.794	0.262
5.515	1.926

The mean of Q_t and F_r were then calculated to obtain one value which will be plotted in the Normalized SBT Charts. The mean of Q_t and F_r were found to be 117.725 and 0.398 respectively. The values for each soil were then plotted in Normalized SBT Charts as shown in Figure 1.2 below:

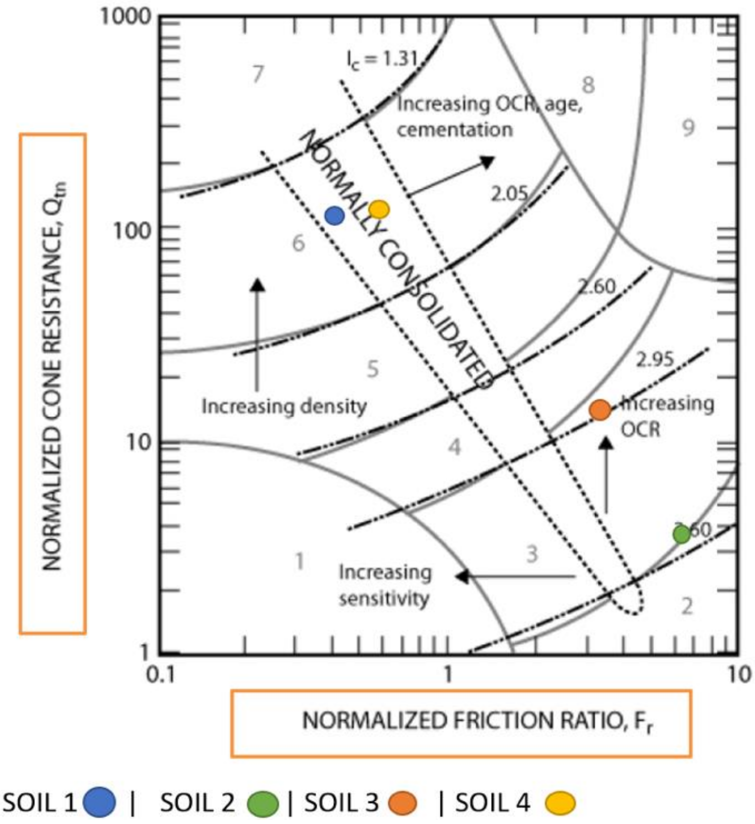


Figure 1.2: Plotted Points in Normalized SBT Charts

According to Figure 1.2, it can be observed that the soils are found to be in zone 3 and 6, which represents sand and clay, as described in Figure 1.3 below:

Zone	*Soil Behavior Type*	I_c
1	*Sensitive, fine grained*	N/A
2	*Organic soils – clay*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6
6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

Figure 1.3: Soil Behaviour Type and Zone

Therefore, a full visualisation of the soil layer can be described in Figure 1.4 as shown below:

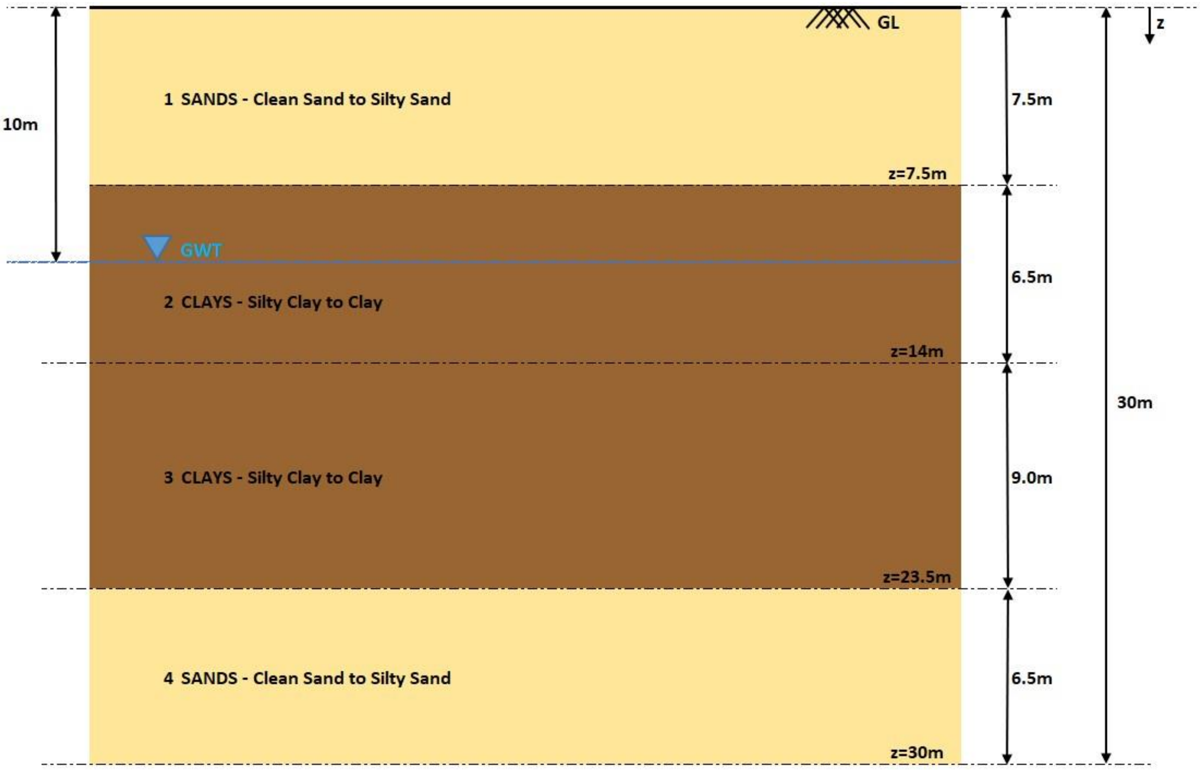


Figure 1.4: Full Visualisation of the Soil Layer

As shown in Figure 1.4, the soil has four layers which is mainly formed by clay and sand layers. The sand layer is located near the ground surface down to the depth of 7.5m. Meanwhile, the clay layer extends from depth of 7.5m until 23.5m. The soil behaviour type index (I_c) for each layer can also be identified from the chart. For section 1 and 4, the value of I_c is found to be 1.31. Furthermore, the value of I_c of section 2 and 3 are 3.5 and 2.9 respectively. This identifies Soils 1 and 4 (top and bottom) as Clean Sand to Silty sand, and Soils 2 and 3 as Silty Clay to Clay.

The shallow foundations will sit within Soil 1, with Z(=B) never reaching the clay layer. This is important for the soil bearing capacity calculations as it dictates which equation is suitable, which is discussed further in Section 2.1.

With the Groundwater Table (GWT) being 10m below the surface, the soil that contains the shallow foundation will be dry. This is important to know for the immediate settlement calculations in Section 4 as this procedure requires the effective stress to be known for the soil zone in question.

2. Bearing Capacity Shallow Foundations

2.1 Direct Calculation Method Using CPT Results

When calculating the bearing capacity, a few design assumptions must be made before calculations can commence. These are aligned with the client's requirements and the first is to only consider square footings for the design of shallow foundations.

In order to protect the shallow foundations from potential damage caused by frost, erosion, trees, and poor soils, a minimum depth must be selected. In this case, the depth of the foundation footing is D=1.0m below ground level. When conducting the design calculations, a range of footing widths from 1.5m to 3.0m have been considered, in 0.5m increments. It is also assumed that each foundation footing only supports one structural column.

The layout considered for the following calculations is portrayed in Figure 2.1 and accounts for the fact that the shallow foundation will only lie within Soil 1 in Figure 1.4, due to the fact that the investigated depth, Z is equal to the width of the footing, B, which at its maximum is 3.0m, so even at the widest footing only the first Soil type has been included in calculations. The depth Z defines the depth of soil which will contribute to the bearing capacity, which is why the soil below need not be considered for these calculations.

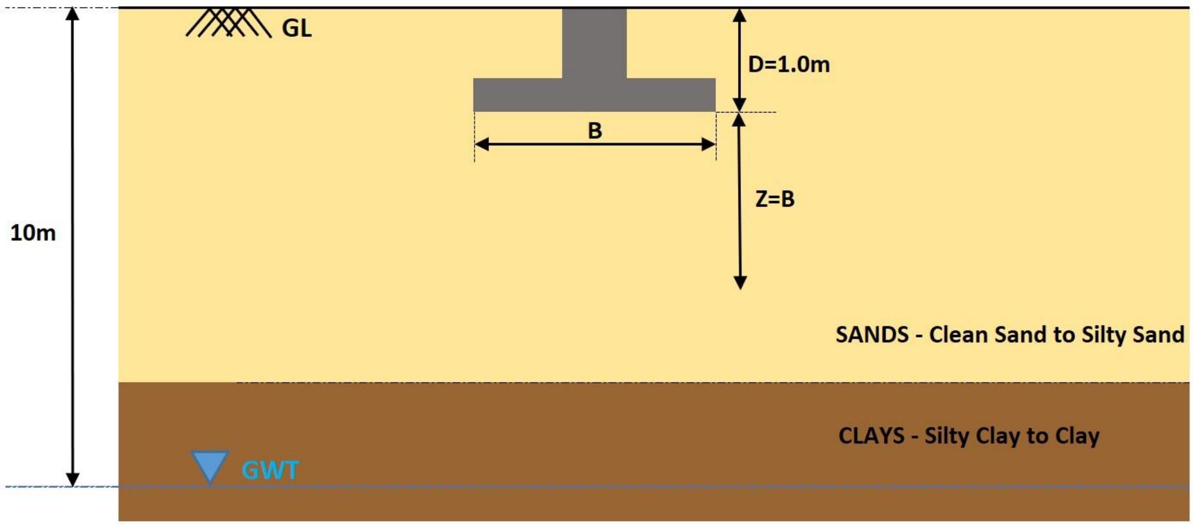


Figure 2.1: Layout of Shallow Foundation Conditions

Due to Soil 1 being Sand, equation (1) is suitable for determining its ultimate bearing capacity.

$$q_u = k_{\emptyset} q_{c(av)} \tag{1}$$

In order to reach this stage, parameters including $k_{\emptyset}$ and $q_{c(av)}$ must first be determined. $k_{\emptyset}$ is found using the graph in Figure 2.2 and in this case $k_{\emptyset}$ has been identified as 0.23. Next, $q_{c(av)}$ is calculated by finding the value of q_c for each depth at increments of 0.5m, and finding their mean for the given value of B. This value is the average CPT penetration resistance below the depth of the footing. It is worth noting here that the depth in question is equal to D + Z, with D=1.0m for all cases and Z being equal to the footing width B. For example, for a footing width of 1.5m, the depth used to calculate $q_{c(av)}$ would be 2.5m (1m+1.5m=2.5m).

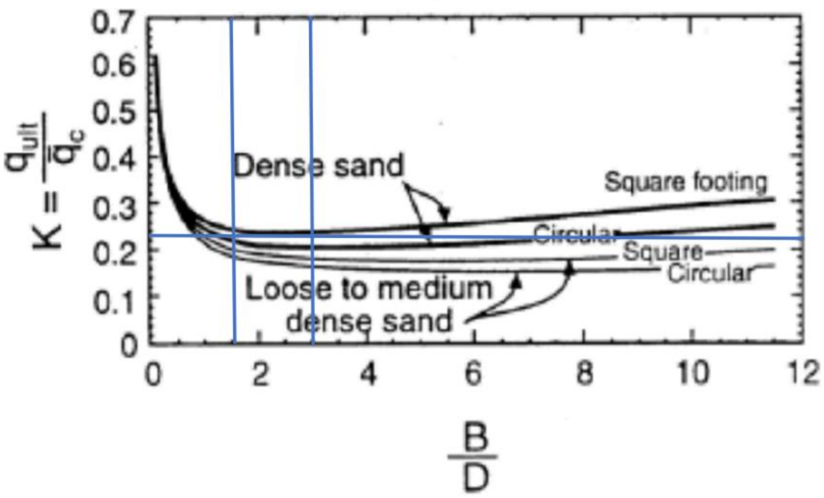


Figure 2.2: Determination of k_{ϕ}

The results from these calculations are summarised in Table 7.

Table 7: Calculation of $q_{c(av)}$

				B/D			
				1.5	2.0	2.5	3.0
Depth (m)	q_c (kPa)	f_s (kPa)	f_r	q_c (kPa)	q_c (kPa)	q_c (kPa)	q_c (kPa)
0.5	4500	12.2	0.00027	4500	4500	4500	4500
1.0	3800	12.4	0.00033	3800	3800	3800	3800
1.5	3500	12.3	0.00035	3500	3500	3500	3500
2.0	2400	12.1	0.00050	2400	2400	2400	2400
2.5	4800	12.8	0.00027	4800	4800	4800	4800
3.0	5700	13.9	0.00024	-	5700	5700	5700
3.5	6300	13.6	0.00022	-	-	6300	6300
4.0	6100	14.3	0.00023	-	-	-	6100
			$q_{c(av)}$ (kPa)	3800	4117	4429	4638

Having found k_{ϕ} and $q_{c(av)}$, the ultimate bearing capacity, q_u can be calculated for each ratio of B/D using equation (1). For bearing capacity calculations, q_{all} can be calculated using equation (2). This is the allowable bearing capacity which takes into account the chosen Factor of Safety of 3 in this case to provide a value of the allowable stress to which the soil can be subjected to before undergoing failure.

$$q_{all} = \frac{q_u}{FoS} \tag{2}$$

The values of q_{all} for each B/D ratio are provided in Table 8. In this case, D=1.0m, so B/D is equivalent to the width of the footing.

Table 8: Calculation of q_{all}

B/D	$q_{c(av)}$ (kPa)	k_{ϕ}	q_u (kPa)	q_{all} (kPa)
1.5	3800	0.23	874	291
2.0	4117	0.23	947	316
2.5	4429	0.23	1019	340
3.0	4638	0.23	1067	356

To visualise these results more clearly, Figure 2.3 depicts a graph of Allowable Bearing Capacity against Footing Width to demonstrate that a greater footing width provides a larger resistance to soil failure.

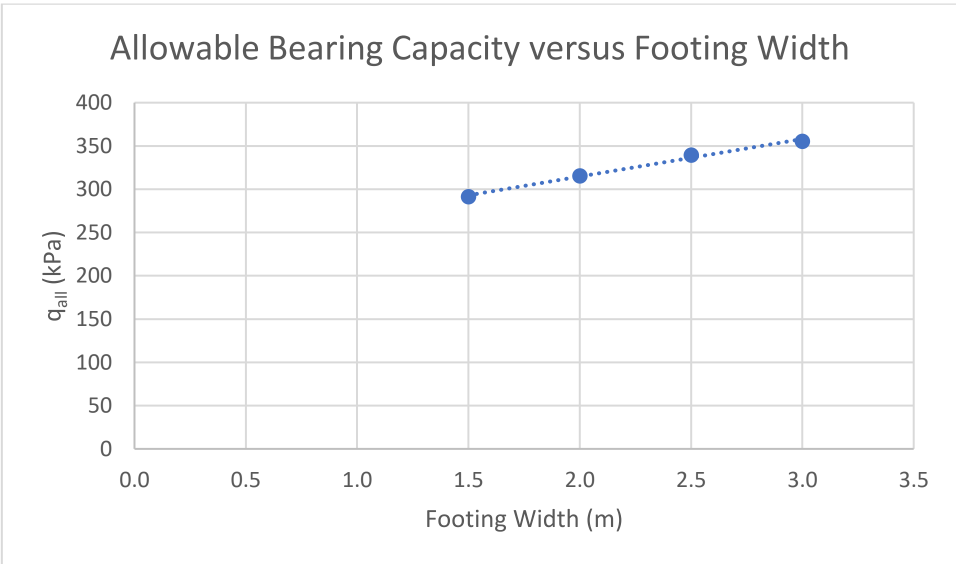


Figure 2.3: Allowable Bearing Capacity versus Footing Width for the Direct Method

This shows that, as expected, the bearing capacity increases with increasing footing width. To determine the required footing width to support the designed structure, settlement calculations are completed in Section 3 and a comparison is made with these Bearing Capacity figures to see which is dominant, and thus which should dominate the design procedure.

2.2 Indirect Calculation Methods Using CPT Results

Indirect calculation methods using CPT results are used to calculate the bearing capacities of the shallow foundations. It also provided a base for comparison with the results obtained via the direct method, as a means for verification. In Section 2.2.1, the verification of the results will be discussed.

Similar to the direct calculation methods using CPT results, the indirect calculation methods follow the same design assumptions of using square footings for the design of shallow foundations and the foundation footings are located at a depth D=1m below the ground level. A range of footing widths from 1.5m to 3.0m have been considered with an increment of 0.5m.

Before calculating the bearing capacity of the shallow foundations using equation (3) shown below, some assumptions of the soil type are made. As the foundation footings only sit on Soil 1 which is sand, the soil is in drained condition, the cohesion part in equation (3) is ignored because the cohesion is 0 (sand is considered a cohesionless soil). In addition, the load inclination factors are assumed to be 1 because the load is vertically acting downward. The simplified equation is shown in equation (4).

$$q_u = 0.5\gamma B N_\gamma s_\gamma i_\gamma + cN_c s_c i_c + \gamma D N_q s_q i_q \tag{3}$$

It simplifies as:

$$q_u = 0.5\gamma B N_\gamma s_\gamma + \gamma D N_q s_q \tag{4}$$

Where:

$\gamma :\ unit\ weight\ of\ soil$

$B :\ footing\ width$

$c :\ cohesion$

$D :\ depth\ of\ footing$

$N_\gamma, N_c, N_q :\ bearing\ capacity\ factors$

$s_\gamma, s_c, s_q :\ shape\ factors$

$i_\gamma,\ i_c, i_q : load\ inclination\ factors$

The first step is to calculate the soil friction angle using the CPT results. For example, when B = 1.5m, use the CPT results of soil layers from 0m to 2.5m.

$$tan\phi' = \frac{1}{2.68}\left[log\left(\frac{q_c}{\sigma'}\right) + 0.29\right]$$

Where:

q_c : $q_{c\,(av)}$ $from\ Table\ 9$

σ' : $average\ effective\ stress$

Table 9: Results of Soil Friction Angles from Width 1.5m to 3m

footing width (m)	$q_{c\,(av)}$ (kPa)	effective stress (kPa)	Soil friction angle (degrees)
1.5	3800	24.65	43
2.0	4117	29.01	42
2.5	4429	33.21	42
3.0	4638	37.47	41

Soil friction angles are rounded up to integers as it is easier to check the bearing capacity factors and use them in equation (4). N_γ, N_q will be found with varying soil friction angles in a table in Appendices.

Next, shape factors s_γ, s_q for square footings are calculated using the Eurocode 7 approach:

$$s_\gamma = 0.7$$

$$s_q = 1 + sin\phi'$$

Finally, the ultimate bearing capacity q_u of footing width from 1.5m to 3m are calculated by applying the results of different parameters into equation (4). The allowable bearing capacity q_{all} considers a Factor of Safety of 3 (refer to equation 2) in this case. The results are shown in Table 10 below:

Table 10: Results from Indirect Calculation Methods Using CPT Results

footing width (m)	N_γ	N_q	s_γ	s_q	depth (m)	γ (kN/m^3)	q_u (kPa)	q_{all} (kPa)
1.5	186.54	99.02	0.7	1.68	2.5	16.72	8590	2860
2.0	155.55	85.38	0.7	1.67	3	16.77	9010	3000
2.5	155.55	85.38	0.7	1.67	3.5	16.801	1067	3560
3.0	155.55	85.38	0.7	1.67	4	16.84	1233	4110

A graph of allowable bearing capacity against footing width is shown in Figure 2.4 to display the results more visually. It demonstrates that a greater footing width provides a greater bearing resistance.

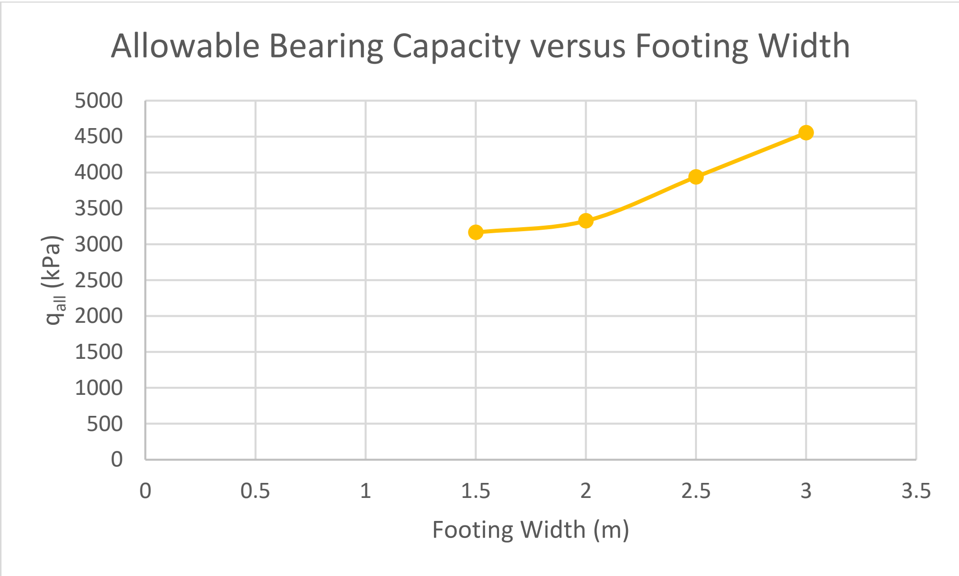


Figure 2.4: Allowable Bearing Capacity versus Footing Width for the Indirect Method

2.2.1 Discussion on Verification with Direct Method Results

From the findings above, it is evident that the ultimate and allowable bearing capacity yielded from Indirect Calculation Methods using CPT results are much greater than those found through Direct Calculation Methods. This is due to the uncertainties within indirect methods, largely present in assumptions. For example, there is a lack of clarity within the indirect method procedure to calculate the soil friction angles. In the equation, q_c is required, but average CPT penetration resistance below the depth of footing $q_{c\ (av)}$ and average effective stress σ' are used instead of the series of values at a specific depth. It is important to consider an average value as soil properties vary across different depths. In addition, the values of soil friction angles are rounded up to integers which reduces the accuracy of the result.

The Direct Method utilises a predetermined depth below the foundation equal to the footing width. This is the zone of soil that is considered to contribute to the bearing capacity, for which the CPT data can be directly used in the calculation of said capacity. This depth is an assumption in itself, but the direct use of measured data eliminates further rounding errors or potential errors in theories, whilst a more direct route to the bearing capacity figure is attainable. As with the indirect method, it is important to consider the average q_c value over this depth due to potential variability in soil properties; it is very unlikely to encounter completely homogeneous soil on site. A potential for the largest error within the direct method is the selection of $k_\emptyset$ due to the lack of precision in the graph. In this case a value of 0.23 was selected as it best represented the data of the soil being investigated, but due to the subjectivity in selection, another person may have selected a slightly different value from the same graph, which introduces human error to the calculations. Despite this being the most prominent error, it is still likely to have a relatively small impact on the results, leading to the direct method for calculating soil bearing capacity to be a robust and relatively easy to implement one which can be used to quickly determine the strength of the soil and the resulting foundations required.

The large discrepancy, in this case a factor of 10, between the results obtained by direct and indirect methods suggests a substantial lack of accuracy in one or both of them. As a result, the only conclusion that can be made from these preliminary calculations is that shallow foundations are unlikely to be suitable for this project, although further confirmation is available in the Settlement calculations of Section 3. To remain on the conservative side of judgement to ensure the safety of this project, it is recommended that the results of the direct method are abided by. Due to the lack of site information available, and the evident uncertainty in calculations, it is with strong recommendation that further site investigations are carried out before any final decisions are made regarding the foundation design. Details of pile foundation calculations can be found later in this report and offer a more effective solution than shallow foundations for this project. However, shallow foundation calculations are completed due to the much lower cost of their use, making them more economical when able to be implemented.

3. Settlement Calculations

The immediate settlement of shallow foundations must be estimated to determine whether the project will be safe to complete. Immediate, or elastic, settlement is caused by the elastic deformation of soil, which in this case is dry, directly after the application of load.

For this project, the client has requested that the immediate settlement be limited to 25mm. The magnitude and envelope of immediate settlement depends on the flexibility of the foundation and the type of soil beneath shallow foundation.

3.1 Determination of Foundation Flexibility Factor

The flexibility of the foundation dictates how the settlement will take place. In this case, the shallow foundations will only be in the presence of sand. A perfectly rigid foundation will cause relatively uniform settlement, as demonstrated in Figure 3.1 (Sedighi, 2022), whilst a perfectly flexible foundation will cause uneven settlement, as in Figure 3.2 (Sedighi, 2022).

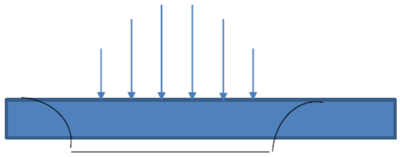


Figure 3.1: Settlement Beneath a Perfectly Rigid Foundation in Sand (Source: Sedighi, 2022)

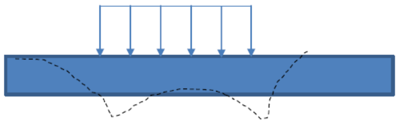


Figure 3.2: Settlement Beneath a Perfectly Flexible Foundation in Sand (Source: Sedighi, 2022)

The Foundation Flexibility Factor can be calculated using equation (5):

$$K_F = \left(\frac{E_F}{E_{s(av)}}\right)\left(\frac{2t}{B_e}\right)^3 \tag{5}$$

Where:

E_F : Elastic Modulus of Foundation Slab
$E_{s(av)}$: Average Elastic Modulus of the Soil
t : Thickness of Foundation Slab
$B_e = \sqrt{\frac{4BL}{\pi}}$ (Where B and L are the width and length of the foundation respectively).

Normally, $E_{s(av)}$ would be estimated through in situ tests or determined in a laboratory using a triaxial test. However, for the purpose of this report an assumption has been made. Based on data provided by StructX, 2022, the average elastic modulus of clean sand is 5-81 MPa and 5-20MPa for silty sand. As a result, with the sand in question being on the border between clean and silty sand, a value of $E_{s(av)}$ = 20 MPa has been assumed.

The concrete grade most typically used for footings and foundations is C25 (EasyMix, 2022). Taking this information and applying it to data to be used in accordance with Eurocode 2, provided by Eurocode applied, 2022, the elastic modulus of C20/25 concrete is 29962 MPa and for C25/30 concrete it is 31475 MPa. Taking an average of these values for use with C25 concrete, we have assumed that the Foundation Slab has an elastic modulus of 30719 MPa.

These assumptions have been combined with an assumed slab thickness of 100mm within Table 11 for use in the calculation of the Foundation Flexibility Factor completed in Table 12.

Table 11: Shallow Foundation Slab Properties

t (m):	0.1
E_f (MPa):	30719
$E_{s(av)}$ (MPa):	20

Table 12: Shallow Foundation Flexibility Factors

B (m)	L (m)	B_e ($m^{1/2}$)	K_F
1.5	1.5	1.693	2.534
2.0	2.0	2.257	1.069
2.5	2.5	2.821	0.547
3.0	3.0	3.385	0.317

K_F was calculated for a range of footing widths between 1.5m and 3.0m in 0.5m increments. It is evident that in all cases, $0.01<K_F<10$, so intermediate flexibility is observed. As a result, the settlement behaviour will be in between those seen within Figures 3.1 and 3.2. The foundation with footing width 3.0m is likely to exhibit the most flexible behaviour, resulting in more varied settlement beneath the foundation. In contrast, the settlement seen below a 1.5m footing is likely to be more uniform due to it being more rigid.

3.2 Calculation of Settlement using Meyerhof Method

Based on the equation suggested by Meyerhof in 1974, immediate settlement can be calculated using equation (6). By setting s to 25mm (or 25 x 10^{-3}m) the required footing width can be determined and thus compared with the proposed footing widths to see if they are sufficient.

$$s = \frac{\Delta PB}{2q_{c(av)}} \tag{6}$$

When dealing with settlement, the allowable bearing capacity q_{all} requires an assumption to be made. It is assumed that $(W_F + W_s)/A$ is equal to the weight of the soil excavated in order to place the shallow foundation, as indicated in Figure 3.3 and equation (7). W_F is the weight of the foundation and W_s is the weight of the soil excavated to lay the foundation.

$$\frac{W_F + W_S}{A} = \gamma_s D \tag{7}$$

Due to the spacing of columns of the proposed structural design, footing widths of 1.5m to 3.0m will be investigated in 0.5m increments.

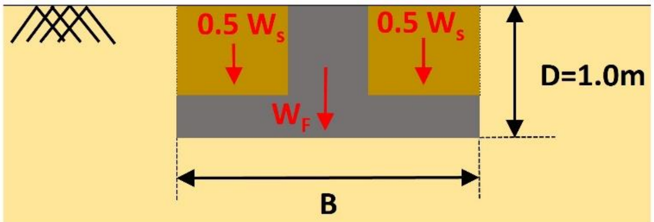


Figure 3.3: Diagram of Shallow Foundation Loads

Assuming that,

$$q_{all} = \frac{W_{D+L}}{A} + \frac{W_F + W_S}{A} \tag{8}$$

Then,

$$q_{all} = \Delta P + \frac{W_F + W_S}{A} \tag{9}$$

Finally, substituting equation (7) into equation (9), the allowable bearing capacity can be calculated by using equation (10) as below.

$$q_{all} = \Delta\mathrm{P} + \gamma_s D \tag{10}$$

ΔP can be calculated by rearranging equation (10) and using s=25mm. These calculations have been completed using excel due to the improved accuracy and reduced chance of human error. The results can be seen below in Tables 13 and 14.

Table 13 shows the estimation of γ_s using data from the CPT results for the site. By multiplying this by the foundation depth of D=1.0m, $\gamma_s D$, the load per unit area caused by the weight of the excavated soil and weight of the foundation, is found and can be used in the calculation of q_{all}.

Table 13: Values of Soil Unit Weight to 4.0m Depth

D (m)	γ_s (kPa)
0.5	16.778
1.0	16.734
1.5	16.692
2.0	15.525
2.5	16.861
3.0	17.023
3.5	17.037
4.0	17.085

From this data, the average value for γ_s is 16.72 kN/m^3.

This results in $\gamma_s D = 16.72\ kPa.$

Table 14 shows the calculation of q_{all} for each footing width B from 1.5m to 3.0m in 0.5m increments. These results are plotted in Figure 3.4.

Table 14: Calculation of q_{all}

B (m)	$q_{c(av)}$ (kPa)	ΔP (kPa)	$\gamma_s D$ (kPa)	q_{all} (kPa)
1.5	3800	127	16.72	143
2.0	4120	103	16.72	120
2.5	4430	89	16.72	105
3.0	4640	77	16.72	94

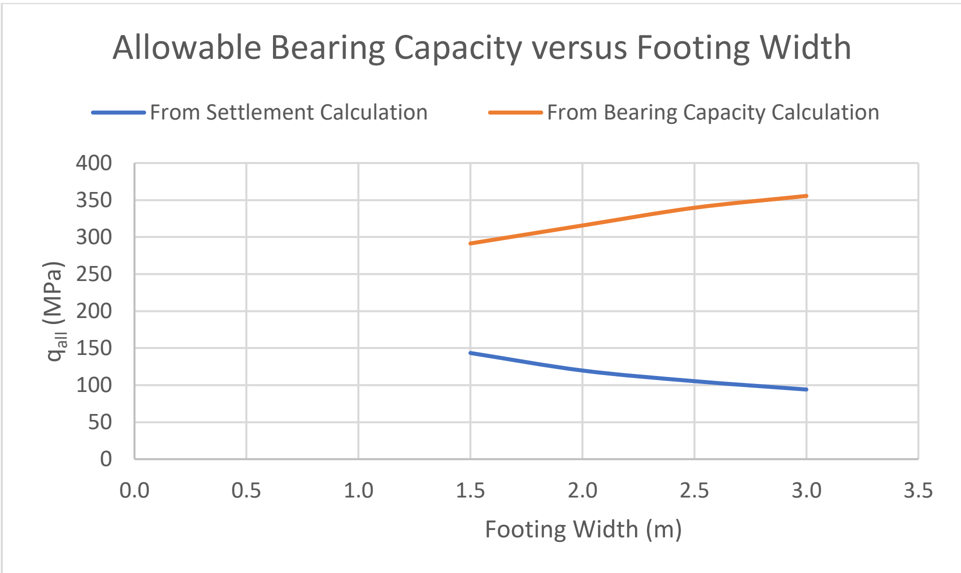


Figure 3.4: Comparison of Calculated Bearing Capacities

From this plot it is clear that the bearing capacity requirement is dominated by the settlement of the footing. Therefore, the results from the soil bearing capacity can be disregarded moving forward and settlement should be focused on as the main controller.

It can be observed from Table 14 that the maximum allowable bearing capacity is 143 kPa which occurs for a footing width of 1.5m, so a base area of 1.5x1.5 m^2.

It was concluded from the structural design that the maximum load exerted by a column on a shallow foundation pad would be 4625.98 kN.

$$\frac{4625.98}{1.5^2} = 2055.91\,kPa \gg 143\,MPa$$

This shows that the structural load is larger than the load that the 1.5m width footing could withstand whilst limiting immediate settlement to 25mm.

The smallest allowable bearing capacity from Table 14 is 94 MPa which occurs for a footing width of 3.0m and resulting base area of 3.0x3.0 m^2.

Using the aforementioned maximum load:

$$\frac{4625.98}{3^2} = 514.00\,kPa \gg 94\,MPa$$

This shows that even with a footing width of 3.0m, the shallow foundation would not be able to limit immediate settlement to 25mm whilst subjected to the maximum structural load exerted by the column.

To find the required footing width:

$$\frac{4625.98}{0.143} < B^2 \;\; => \;\; B > 5.59m$$

Based on the maximum allowable bearing capacity, a footing width of at least 5.59m would be required to limit the settlement to 25mm as required, which would not be possible due to the spacing of columns within the proposed structural design.

As a result, it can be concluded that shallow foundations are not a suitable design scheme for this project based on the provided requirements and subsequent assumptions as stated.

3.3 Calculation of Settlement using Schmertmann Method

Based on the method developed by Schmertmann in 1970, the immediate settlement beneath a shallow foundation can be calculated using equation (11). A slightly different approach is taken here than with the Meyerhof method. Instead of setting the settlement to 25mm and calculating ΔP, and then the resulting q_{all} to find the required footing width, the equation will be used as it is, and the settlement will be calculated for each footing width and compared to the requirement of 25mm to see whether shallow foundations are suitable for this project.

This approach is more complex than the Meyerhof method due to the inclusion of more variables, but provides a figure for comparison and source for verification of both methods, provided they yield similar results.

$$s = C_1 C_2 \Delta P \sum \frac{I_z}{C_3 E'} \Delta z \tag{11}$$

Where:

$$C_1 = 1 - 0.5\left(\frac{\sigma_1'}{\Delta P}\right) = 1 - 0.5\left(\frac{\gamma D}{\Delta P}\right)$$

$$C_2 = 1 + 0.2\log\left(10t_{yr}\right)$$

$$C_3 = 1.2\ for\ square\ footings$$

$$E' = \alpha_E q_c$$

In this case, where the shallow foundation comprises a square footing, the depth investigated is z=2B, with B=footing width. Δz is the thickness of the n increments which depth z is divided into. The settlement will be evaluated for a range of footing widths from 1.5m to 3.0m in 0.5m increments due to the spacing of columns proposed in the structural design of the building.

α_E is a function of the degree of loading, soil density, stress history, cementation, age, grain shape and mineralogy of the soil (Sedighi, 2022), and ranges from 2 to 20 depending on the degree of consolidation of the soil. In this case, a value of α_E=10 has been used which assumed the soil is a normally consolidated sand. This value has been chosen because the soil classification revealed that the soil is Clean Sand to Silty Sand, so the clay content will be low.

C_2 is reliant on cyclic loading, which will not occur in this case. Therefore, a value of C_2=1.0 has been selected for these calculations. I_Z is the rigid footing strain influence factor and can be determined using Figure 3.5. The shallow foundations in this case are square footings.

A safety factor of 3 was applied to produce conservative results given the limited available information. Further site investigation could lead to a lower safety factor being used due to increased certainty, leading to more precise results.

Having split the soil into 0.5m layers, Δq, $q_{c(av)}$, E', I_z and Δs can be found for each increment, and then the Δs can be summed to find the total settlement of the soil to a depth of twice the footing width. These calculations were completed in Excel. The results are summarised in Table 15 for footing widths 1.5 to 3.0m but the more in-depth calculation tables can be found in the Appendix. A visual representation of these results is provided by Figure 3.6.

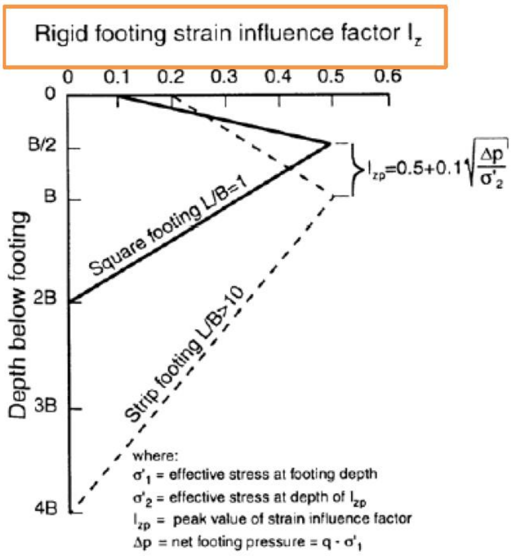


Figure 3.5: Determination of I_z

Table 15: Immediate Settlement for Footing Widths 1.5m to 3m

B (m)	s (mm)
1.5	110.9
2.0	80.7
2.5	59.9
3.0	47.9

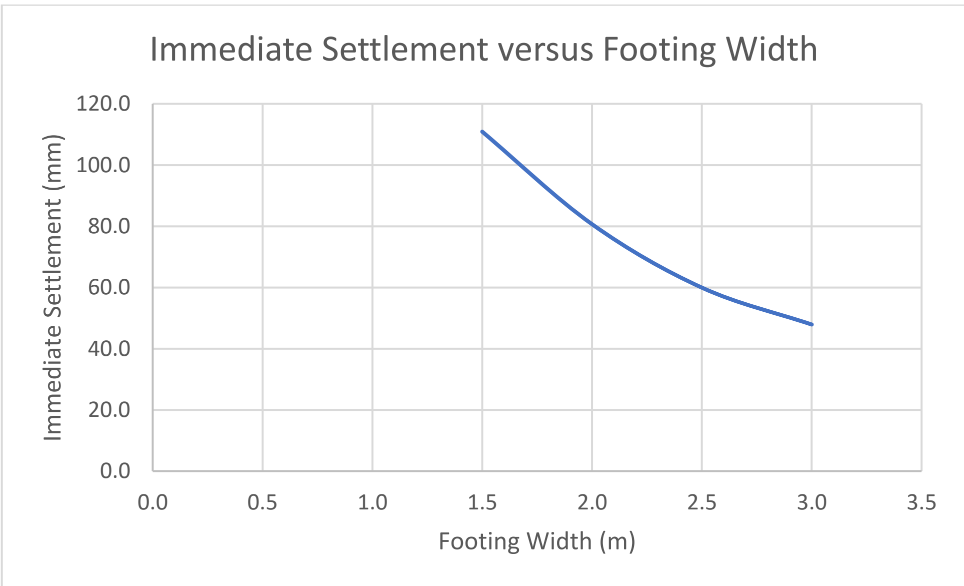


Figure 3.6: Plot of Immediate Settlement versus Footing Width for Schertmann Method

From the immediate settlements in Table 15 it is evident that even a footing width of 3.0m is insufficient to support the structure whilst limiting settlement to 25mm. It is clear that as the footing width increases, the settlement substantially decreases, reinforced by the negative gradient of the curve in Figure 3.6. However, given the constraints of the structural design, the footing width required to limit settlement to 25mm would be too large to be feasible, as even at 3.0m the settlement is nearly twice that allowed, and the reducing gradient implies that each increase in width has a smaller effect on settlement. Calculations have not been completed into the exact required footing width due to the conclusion already being reached and such data would provide little added value, but it can be estimated that based on the Schertmann method, the footing would need to be at least 8 to 9m, exceeding the constraints of the structural column design.

Having evaluated the immediate settlement for the proposed structural design using both the Meyerhof and Schmertmann methods, and reaching the same conclusion, it can be said with some certainty that shallow foundations are not suitable for this application due to the excessive settlement that would ensue. As a result, it is recommended that investment is made into further site investigations to build a comprehensive data set tailored to calculations for pile design, as this is the most suitable foundation scheme. Preliminary design calculations are provided in the following sections of this report.

4. Bearing Capacity and Length of a Single Pile Foundation

4.1 Alpha LCPC Method

The alpha method is used to calculate the bearing capacity of piles, this method is often used in cohesive soils. In addition, this method is orientated around undrained shear strength. The undrained ideology is a conservative approach, as when the soil is loaded, the water does not dissipate instantly, resulting in an increase of pore water pressure, reducing the effective stresses. This scenario increases the chances of failure and considers the worst-case scenario. Designing for the worst-case scenarios is a safe approach. Furthermore, this method is applied to cohesive soils (soils that don't dissipate water instantly), the soil profile in this project is a mix of clay (cohesive) and sand (cohesionless), so applying the alpha method to this soil profile is a conservative approach.

The load which the single pile must withstand is found in the Structures report. The greatest structural load was found to be 4.63MN (viewed in the Figure 4.1 below).

Floor	Slab Area (m^2)	Force per area ULS (kN/m^2)	Slab and applied loads (kN)	Sec beam self weights (kN)	Primary beam self weight (kN)	Total ULS Load (kN)
Roof	87.5	15.2325	1332.84375	39.20076	17.56971	1389.61422
Floor 3	87.5	11.82	1034.25	31.916835	12.62179125	1078.788626
Floor 2	87.5	11.82	1034.25	31.916835	12.62179125	1078.788626
Floor 1	87.5	11.82	1034.25	31.916835	12.62179125	1078.788626
SUM						4625.980099
	Column required to withstand (kN):		4625.980099			
	Column Section (Buckling length 4.6m)		Buckling resistance capactiy (kNm)	Quantity	72	
	356x406x634 UC		4630			

Figure 4.1: Maximum Structural Load of our column - From the Structures Report

Our methodology for calculating the length of a single pile involves using MATLAB programming to perform a data analysis of the soils present. To calculate the depth of a pile by hand, a trial and error is a possible method but it is time-consuming, excel is a method but this will struggle to identify the appropriate k_c, a_{LCPC}, $f_{p,max}$ values (from their tables) for the various depths. 'if' loops are primarily important for condition checking the factors (soil type and q_c value) which determine these parameters (k_c, a_{LCPC}, $f_{p,max}$). The program used computes the allowable bearing capacity for every depth (0-30m in increments of 0.5m). Then the data of the allowable bearing capacity can be checked to find the depth which is suitable to support the load in question: '4.63MN'. A full explanation of the code is discussed in the appendices (section 9). However, the results of the code and identified length is discussed here (section 4).

A verification check of the MATLAB code is produced in section 4.1.1.

The soil data provided only reaches a depth of 30m. No single pile length (from 0.5m-30m) of (diameter 0.4m) was suitable to resist the load acting on the foundation (4.63MN). A copy of the total allowable bearing capacity (factor of safety: 2 was used) results at each 0.5m depth increment are provided below (table 16). From table 16, it is evident that at no depth between 0-30m is a single pile suitable. Therefore, a pile group is now suggested and considered (this is discussed in sections 5 and 6), and the client should not use a single pile foundation, it will result in failure.

Section 4.1.1 checks the MATLAB code at depth 20m depth (random depth chosen to prove verification) to find the allowable bearing capacity (Q_{all}). If the code is correct, the hand calculations (in section 4.1.1) should result in $Q_{all} = 0.5397MN = 539.7kN \cong 540kN$

Table 16: Allowable Bearing Capacity of a Single Pile

Depth (m)	Q allowable (SINGLE PILE) (kN)
0	000.0
0.5	115.3
1	120.8
1.5	114.2
2	131.9
2.5	175.6
3	213.6
3.5	244.3
4	272.7
4.5	276.5
5	280.8
5.5	267.2
6	308.8
6.5	300.9
7	313.5
7.5	205.1
8	228.3
8.5	233.3
9	238.3
9.5	243.4
10	247.5
10.5	252.3
11	257.7
11.5	261.5
12	266.8
12.5	272.0
13	279.3
13.5	284.6
14	266.4
14.5	277.4
15	386.6
15.5	405.7
16	446.1
16.5	437.9
17	463.4
17.5	447.5
18	459.9
18.5	506.7
19	492.2

19.5	532.5
20	539.7
20.5	520.0
21	560.8
21.5	585.9
22	594.1
22.5	607.0
23	622.7
23.5	483.2
24	520.9
24.5	1,484.3
25	1,536.0
25.5	1,563.9
26	1,593.2
26.5	1,629.5
27	1,674.2
27.5	1,702.1
28	1,787.3
28.5	1,844.5
29	1,896.2
29.5	1,922.7
30	1,940.9

4.1.1 Verification of MATLAB result (Alpha LCPC Method)

The use of a MATLAB programme to calculate the necessary pile length vastly improves the efficiency of the procedure and can improve accuracy through reducing the risk of human errors (by making a rounding mistake) throughout. However, the results must be verified through the hand calculation of a given pile length to ensure the results are in line with those expected from the theory. This method is susceptible to mistakes based on what the engineer has determined for the k_c, a_{LCPC}, $f_{p,max}$ values from the tables, this comes from lack of experiences using the tables and this must be considered, that is why the beta method is also completed as a cross check for the Q allowable (beta method is completed and discussed in section 4.2)

In this case, a single pile of length 20m (chosen at random) is investigated to calculate the allowable bearing capacity, for comparison with the results obtained from MATLAB.

At depth 20m, the soil is classified as clay and q_c=5.5 MPa. Using the K_c table, this pile is classed as Group 1. Therefore, K_c = 0.45.

The diameter of the pile is D=0.4m.

$$a = \frac{3}{2} \times 4 = 0.6m$$

Our data is accurate to 0.5m, but 0.6m would require the data to be accurate to 0.1m depth increments. Therefore, 'a' can be rounded down from ±0.6m to ±0.5m for accuracy and to remove the assumptions about what the q_c values are at the 0.1m increments. Assuming q_c values at 0.1m increments would result in assumptions, and if assumptions were made at every 0.1m increments, the final bearing capacity results would be highly inaccurate, therefore only q_c values which are known, are used.

q_{ca} is the equivalent average cone resistance at the base of the pile (Sedighi, 2022).

$$q_{c1} = 5.5\ MPa\ @\ 20m - 0.5m = 19.5m$$

$$q_{c2} = 5.5\ MPa\ @\ 20m$$

$$q_{c3} = 4\ MPa\ @\ 20m + 0.5m = 20.5m$$

$$q_{ca} = \frac{5.5 + 5.5 + 4}{3} = 5\ Mpa$$

Next it must be checked whether the q_c values used to find q_{ca} lie within this range:

$$0.7q_{ca}\ to\ 1.3q_{ca}$$

$$0.7 \times 5\ to\ 1.3 \times 5$$

$$3.5\ to\ 6.5$$

q_{c1}, q_{c2}, and q_{c3} are all values within this range, therefore q_{ca} remains at 5 MPa and does not need to be adjusted.

The unit base resistance is calculated using equation (12).

$$q_b = K_c q_{ca} \tag{12}$$

$$q_b = 0.45 \times 5 = 2.25\ MPa$$

Then, the total base resistance can be calculated using equation (13).

$$Q_b = A_b q_b \tag{13}$$

$$Q_b = (\pi \times 0.2^2) \times 2.25 = 0.2827\ MN$$

The unit shaft resistance can be calculated using the α method which involves the friction coefficient. The necessary equation is (14).

$$f_p = \frac{q_c}{\alpha_{LCPC}} \tag{14}$$

The soil is a clay with q_c=5.5 MPa at depth 20m, so α_{LCPC}=60.

$$f_p = \frac{5.5}{60} = 0.092\ MPa$$

$$f_{p(\max)} = 0.035 < 0.092$$

Therefore,

$$f_p = 0.035\ MPa$$

The average $\bar{f_p}$ for the 20m length becomes 0.0317 MPa.

The next step is to calculate the shaft resistance, Q_s, using equation (15).

$$Q_s = A_s f_p$$

$$Q_s = (\pi \times 0.4 \times 20) \times 0.0317 = 0.797\ MN \tag{15}$$

Therefore, a single pile at a depth of 20m has an ultimate bearing capacity of:

$$Q_{ult} = Q_b + Q_s = 0.2827 + 0.797 = 1.0798\ MN$$

The allowable bearing capacity is yielded by applying a Safety Factor of 2, as below:

$$Q_{all} = \frac{1.0798}{2} = 540\ kN$$

This matches the output of 539.7 kN (0.5397 MN) from the MATLAB code, thus the program results are verified. (View figure 4.2 below for the MATLAB output at depth=20m).

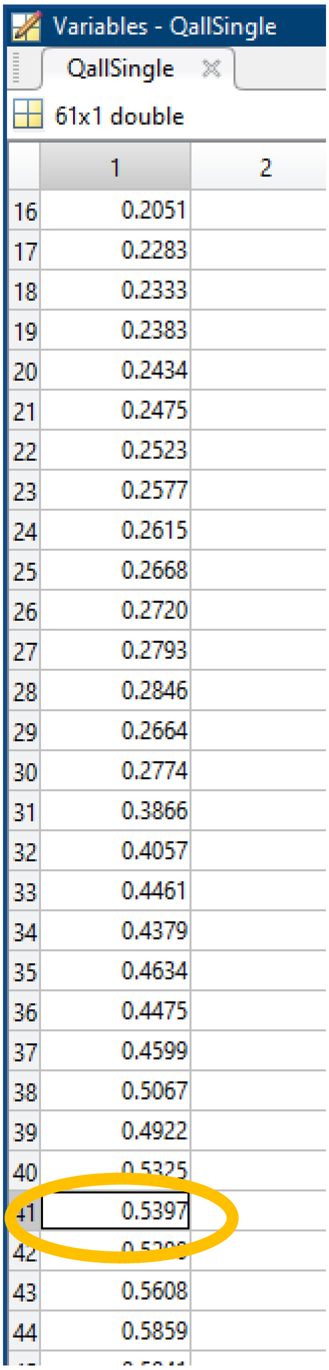


Figure 4.2: Cross check snip of MATLAB Q allowable values for a single pile, highlighting Qall at 20m depth. Units: MN

4.2 Beta Method (to check Alpha LCPC results)

The Beta method is done for comparison reason to ensure the alpha method was not preformed incorrectly, and to examine which method provides more conservative results. The equation used for the beta method is as follows:

$$Q_{ult} = f_p A_s + q_b A_b$$

While A_s is the total periphery area and A_b is the base area of the pile. f_p and q_b is given as follows:

$$f_p = \beta \sigma'_v$$

$$q_b = N_t \sigma'_b$$

And β and N_t can be found from the figure below (figure 4.3):

$$\beta$$

Soil Type	Cast-in-place Piles	Driven Piles
Silt	0.2 - 0.3	0.3 - 0.5
Loose sand	0.2 - 0.4	0.3 - 0.8
Medium sand	0.3 - 0.5	0.6 - 1.0
Dense sand	0.4 - 0.6	0.8 - 1.2
Gravel	0.4 - 0.7	0.8 - 1.5

$$N_t$$

Soil Type	Cast-in-place Piles	Driven Piles
Silt	10 - 30	20 - 40
Loose sand	20 - 30	30 - 80
Medium sand	30 - 60	50 - 120
Dense sand	50 - 100	100 - 120
Gravel	80 - 150	150 - 300

Figure 4.3: Figure Containing the Beta Values and Nt Values

Since a cast-in-place concrete pile is used and the soil profile form CPT data is as follows:

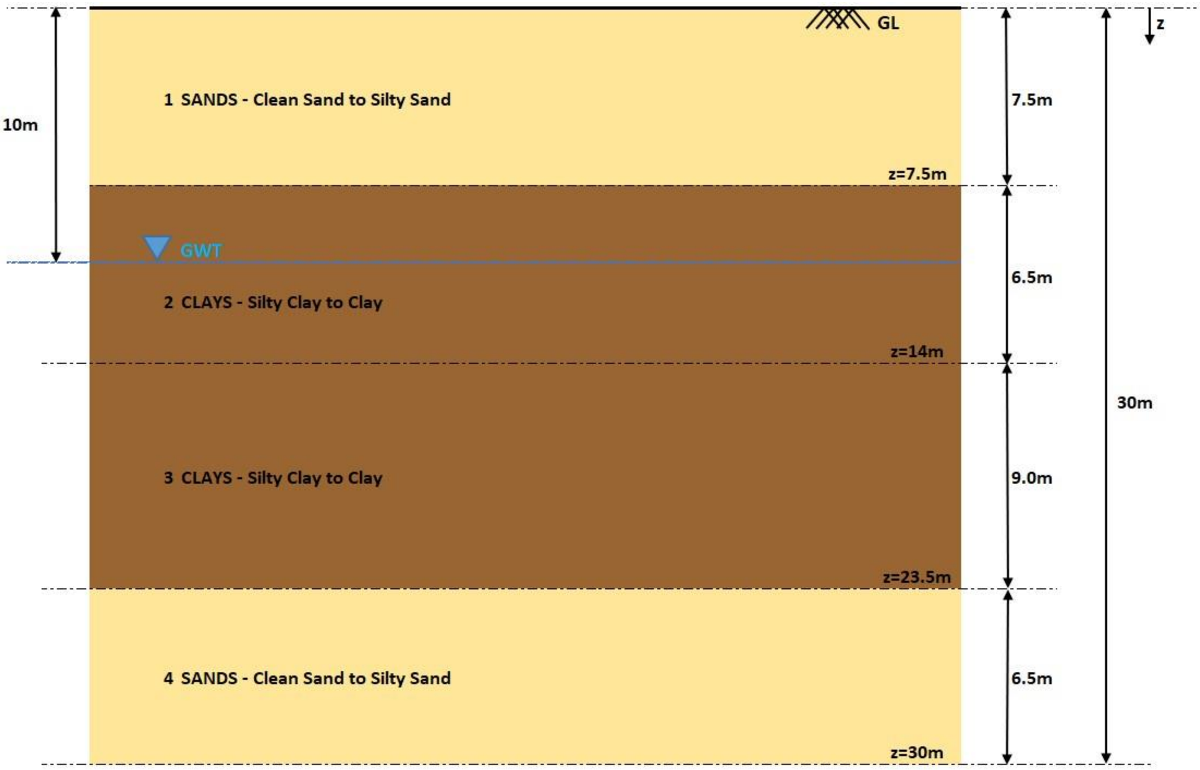


Figure 4.4: Soil profile

According to the soil profile (figure 4.4) the sand and clay has been identified. For the sand to silty sand sections, β and N_t are taken as 0.5 and 40 respectively (subject to human error from lack of experiences with the tables), and for the clay sections, β and N_t are 0.8 and 50 respectively take the number as 0.5 and 30.

Since the unit weight varies vertically, MATLAB code has been used to add up section by section (each section is cylinder with 0.5m height) vertically. The key algorithm is discussed in the appendices section 'Code Methodology' however the results are discussed here.

The results for the lower depth are as follows (FoS 2 has been applied):

Table 17: Comparison of Alpha and Beta method results

Depth (m)	Allowable Bearing Capacity BETA METHOD (kN)	Allowable Bearing Capacity ALPHA METHOD (kN)
25.5	1,379.2	1,484.3
26	1,296.6	1,536.0
26.5	1,334.1	1,563.9
27	1,345.6	1,593.2
27.5	1,433.9	1,629.5
28	1,483.7	1,674.2
28.5	1,569.4	1,702.1
29	1,547.6	1,787.3
29.5	1,589.9	1,844.5
30	1,605.0	1,896.2

The result is compared with alpha LCPC method for a cross check and to note which method is more conservative. The beta method is more conservative. It is noted that the accuracy of the beta method is highly dependent on the accurate estimation of the β and N_t, this could be incorrect due to human error and lack of experience with using the β and N_t tables. However, the result is within tolerance which verifies both the methods (alpha and beta). The alpha/alpha -LCPC method is using the calculated undrained shear stress and strength to find out the total bearing capacity. The beta method results were similar and slightly less than the alpha method results. The beta method calculates the bearing capacity directly from the effective stress calculated from the unit weight profile of the soil investigated. As the beta method uses effective stresses (when pore water pressure has been subtracted) the effective stress is lower than the total stress (undrained, which the alpha method uses), this will result in a lower value as the effective stress is usually less than the total effective stress. Beta is more empirical (although both methods are empirical), which means less accurate and, in this case, underestimate the true bearing capacity (which is safe and conservative, but this could result in overengineered piles which can lead to high costs for the client). In addition, the beta method can be computed faster when compared with alpha/alpha-LCPC method. Overall, alpha method will be processed in the field when the soil profile is new (without existing building) and beta method may be used in circumstances like adding auxiliary or temporary structure near existing building, under supervision of experienced senior engineer. The safest way, of course, is preform both methods to verify each other. Finally, the client requested an alpha method approach to the bearing capacity, in this document a beta method approach has also been provided just to show the client that the beta method values are more conservative than the alpha method values (as expected) and that the alpha method values are similar to the beta method, hence the calculations were preformed correctly.

5. Suitable Foundation Recommendation for the Maximum Structural Load

Analysis has been carried out in this report to find the bearing capacities of different foundation designs (shallow foundations, single piles and group piles which is upcoming) to find the most effective and efficient design for the maximum structural load exerted by a column. The maximum structural load was calculated in the structures part of the project (this value is highlighted in figure 4.1). According to the results in Section 2 and 3, it can be concluded that it is not advised to use shallow foundations as they would not be able to withstand the maximum load of 4,626 kN from the column of Building A whilst limiting settlement to 25mm, which was taken into consideration in this project to consider the worst-case scenario. Four different widths of shallow foundations were considered with a maximum of 3m as per the client's request; however, these options should not be taken further due to its lack of strength to support the column. A width of 5.6m for the square foundation footing would be needed based on the maximum allowable bearing capacity which would exceed the width requirements enforced by the column spacing, using a width of 5.6m between shallow foundations would severely effect bearing capacity and settlement results, as the square shallow foundations global shear failure path will interact with neighboring shallow foundations. In addition, as the site is located near a lake and river (which is prone to flooding as found in the Hydraulics report), there is a possibility of the water table fluctuating at certain times. The increase in water table could impose negative effect to the stability of the structure due to the presence of uplift force. Hence, pile foundation is preferred and highly recommended to the client to be used for this site.

Hand calculation for single pile foundation was previously performed in section 4 by designing for a circular single pile with length of 20m and factor of safety of 2. However, it was found that the maximum resistance of a 20m pile length is only 539.7kN, which is significantly lower than the actual load that the pile needs to resist. Therefore, further calculations regarding suitable length of the single pile were executed using MATLAB to increase efficiency of the calculation process. After executing the program at every depth in increments 0.5m from the surface (0m) to the maximum depth which data was available (30m), it was found that a single pile is not suitable to support the maximum structural load, even at depth of 30m where the allowable bearing capacity is 1,844kN. This result displays that a single pile foundation design is not suitable for the client. Furthermore, it is recommended a group pile design is used, this check was completed in the upcoming section (section 6). (View the final table in the appendices for the results at all depths, table 19).

When calculating the group pile failure, two methods are considered: Block and Individual failure of piles. Both methods can support the maximum structural load from the column of Building A at various depths. However, the efficiency ratio of a pile group was calculated to be greater than 1 (view table 19 at the end of the appendices for the values of eta at all depths, eta represents the efficiency ratio of a pile group), which stated that the bearing capacity of Block Failure is greater than in Individual Failure due to the zone of influence, however the zone of influence topic is discussed further is section 6. In this case, the Individual failure result is chosen as it gives a lower bearing capacity value (more conservative) and thus is the worst-case scenario for safety. Thus, a length of 24m for the group pile is recommended to resist the maximum load (this can be viewed in the final table of the appendices). In general, large-displacement piles are used when the efficiency ratio is greater than one. However, in this project, a bored cast-in-situ pile is recommended to meet the client's request and to provide minimal vibration, options to use a variety of lengths where necessary, and it is the most commonly used in urban areas.

Due to the substantial expenditure (high cost) of a pile group foundation solution, along with a combination of limited data and uncertainties in calculations at this stage, it is suggested that further actions should be considered prior to the detailed design phase. This should include investment into further site investigations to check whether the soil properties vary from one location to another in the construction area or not through in-situ tests. This is vital to ensure all the foundations for each column are strong enough to support that load while also ensuring that it is not overdesigned, as that is uneconomical and unsustainable. As this building will be located in a city centre area, services and underground works must also be considered. Moreover, considering that the area of the project is a brownfield site, there is a potential that the soil might be contaminated; thus, as stated by Design Buildings (2022), a combined geotechnical and geo-environmental investigation should be considered. Where necessary, remediation of the design should be done, as piling through contaminated ground could create a path for pollutants to enter groundwater (NetRegs, n.d.).

6. Further Recommendations and Analysis of Group Pile Foundations if Single Pile Is Not Sufficient

The single-pile foundation is not suitable to support the loading imposed by the column. As a result, a pile group foundation has been proposed along with the results of its bearing capacity. Further recommendations and analysis of group pile are discussed in the upcoming sections.

Based on the client's request, the group pile set up is chosen as a square footing, the piles used in this group have a diameter of 400mm (shown in figure 6.1), with concrete as their construction material. Moreover, the construction methodology is cast-in-place (bored) concrete to be specific.

The number of piles in a pile group is 4. This is due to one pile almost being suitable to withstand the maximum structural load from the column, therefore, there is no need for a pile group with 9 piles or 16 piles, the design would be over-engineered and result in a high cost. As a result, the pile group has been designed for 4 piles.

For the pile's layout within the pile group, the layout pattern is symmetrical, this encourages an even load distribution between the piles. (Layout is provided in figure 6.1).

When calculating the pile group bearing capacity it is important to consider the centre-to-centre (c/c) spacing for the piles within the same group and the c/c spacing between piles in neighbouring pile groups (shown in figures 6.2 and 6.3). This is because the zone of influence of one pile may interact with the zone of influence of another pile (if the c/c spacings is not efficient) within the same group or with a neighbouring group, this will affect the true bearing capacity (reducing it). Ideally, the zone of influence for each pile in the group should not interfere with the other piles in the same group. In this case, the corner column is chosen to consider the spacing because this corner has the shortest distance of 7.5m to the next column (pile group) which is the strictest condition where a zone of influence overlap is likely. As a result, the c/c spacing of piles within the same group is calculated as 3.55m and 3.95m for piles in a different group. The calculations of the c/c spacings are shown below (where c is the c/c spacing of piles within the same group, and s is the distance between the square edge of the pile groups) (highlighted in figures 6.1, 6.2, 6.3):

$$7.5 = s + \big((0.5 + 0.4) \times 2\big) + (c - 0.4)$$

Where,

$$c = s + 1$$

$$7.5 = s + 1.8 + c - 0.4$$

$$7.5 - 1.8 + 0.4 = 6.1 = s + c$$

$$6.1 = c - 1 + c$$

$$7.1 = 2c$$

Therefore,

$$c = 3.55m$$

$$s = 2.55m$$

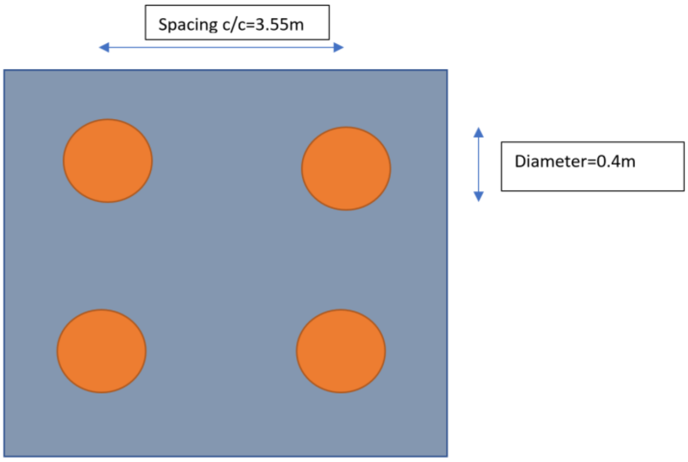


Figure 6.1: Spacing of piles and their diameters.

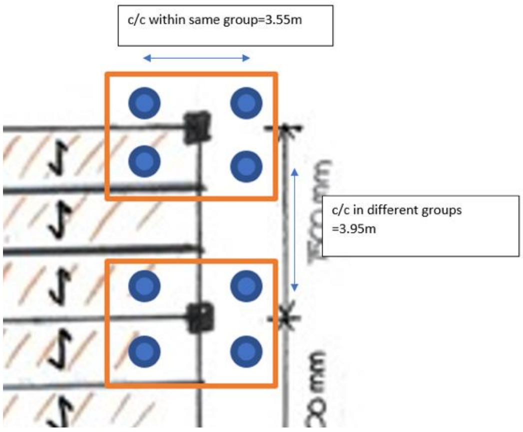


Figure 6.2: Spacing of piles in the same group, and the spacing between neighbouring groups

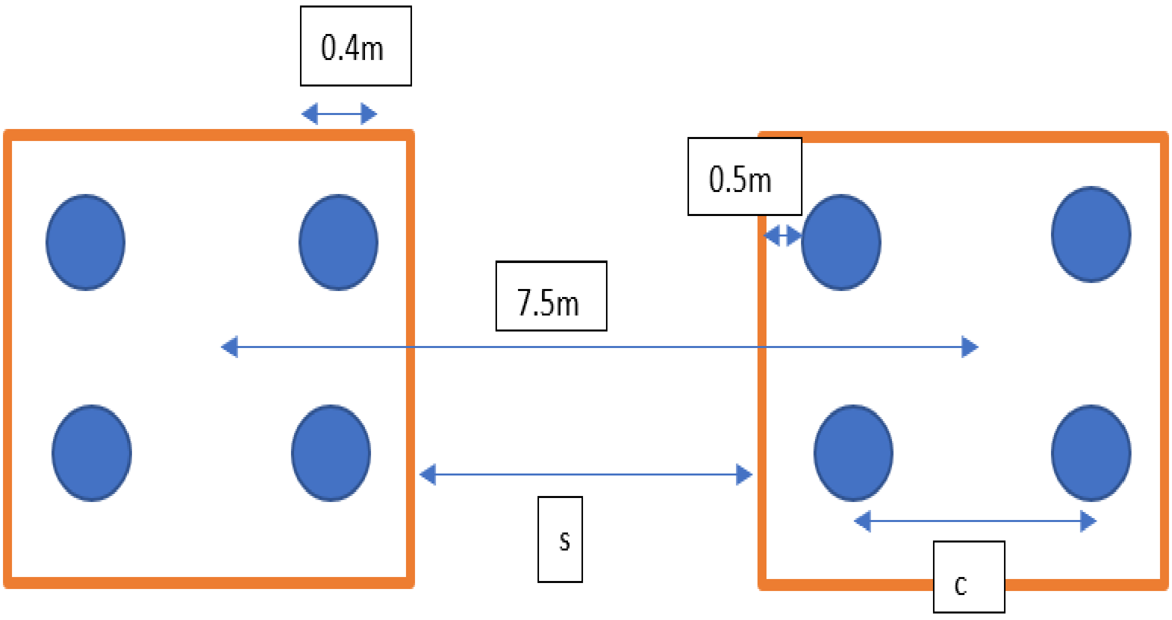


Figure 6.3: Figure highlighting what 's' and 'c' represent

After calculating the pile groups bearing capacity resistance through two methods (completed on MATLAB, verification of both methods is complete in section 6.1 and 6.2). The individual pile group calculation returned lower bearing capacity values than the block bearing capacity calculations. As a result, the individual pile group calculation is used as this method's values are lower, so they provide a more conservative answer. It is better to underestimate the bearing capacity of the soil as opposed to overestimating it and resulting in building failure. It was found that for the pile group layout above, a depth of 24m is suitable to withstand the maximum structural load from the column. The allowable bearing capacity resistance (FoS: 2) at depth 20m is: 4,909.5kN (viewed in table 18 at the end of the appendices) which is greater than the maximum structural load of: 4,630kN. Furthermore, this pile group is utilising 94% of its capability (after FoS has been applied). This number demonstrates efficient material use whilst still having a slight remainder of utilisation for increased safety.

$$Utilisation = \frac{4{,}630.0}{4{,}909.5} \times 100 = 94\%$$

These results have been designed assuming that the zone of influence of the neighbouring piles do not interfere with the nearby piles (of the same group or neighbouring groups), further tests should be completed to find the optimal spacing value. This can be completed using the value eta which helps gauge the efficiency of the pile spacing, however using geotechnical software can help when testing the zones of influence of nearby piles.

Further investigations for the client include a settlement test of the pile group, this is important to consider as the bearing capacity of the pile group may be okay, however the settlement may not be allowable due to standard design codes.

In addition, further investigation is required to ensure that positive skin friction is applied along the pile length. The regions in particular which could result in negative skin friction is the clay sections (ranging from depths 8m-23.5m). Negative skin friction is the result of the soil surrounding the periphery of the pile, settling more than the pile itself. This situation is found in clay soils, as clays experience consolidation settlement (sands only experience instant/elastic settlement). Over a period of time, they have a tendency to drag downwards on the pile periphery. Consolidation of clays is caused by the dissipation of water. This can reduce the pile's bearing capacity as the negative skin friction drags the pile downwards. If, from further tests, it is found that the clay will impose negative skin friction on the periphery of the pile, then mitigation methods are highly recommended. The first mitigation method is to use a sleeve around the pile to stop negative skin friction from occurring, or use a small layer of bitumen on the shaft surface (only apply to regions which are susceptible to negative skin friction) (Szypcio Z, et al. 2006). In addition to clays, negative skin friction could be experienced due to newly back-filled soils.

In addition, the client must be prepared to incur increased costs with this project, as further tests will come at a cost, and that using pile groups can be an expensive method, however the pile group discussed in this section is recommended, provided the settlement tests are acceptable.

6.1 Verification of MATLAB Results – Block Failure Method

This method is a relatively fast way of determining the capacity of a pile group, although it is known to significantly overestimate the true bearing capacity. In this case, we will check for a depth of 24.5m.

As in Section 4.2, the unit base resistance is calculated using equation (3). At 24.5m (chosen at random to check), the soil type is sand with a value of q_c=51.8 MPa. Therefore, K_c=0.3. For this case, due to the accuracy of the provided data being to ±0.5m, a=±0.50m, rather than the 1.5x0.4=0.6m calculation.

$$q_{c1} = 44.4\ MPa\ @\ 24.5m - 0.5m = 24m$$

$$q_{c2} = 51.8\ MPa\ @\ 24.5m$$

$$q_{c3} = 51.1\ MPa\ @\ 24.5m + 0.5m = 25.0m$$

$$q_{ca} = \frac{44.4 + 51.8 + 51.1}{3} = 49.1\ MPa$$

Now, check if the q_c values to calculate q_{ca} are in the required range:

$$0.7q_{ca}\ to\ 1.3q_{ca}$$

$$0.7 \times 49.1\ to\ 1.3 \times 49.1$$

$$34.37\ to\ 63.83\ (MPa)$$

Therefore, all of the q_c values are within this range, so q_{ca} retains its value of 49.1 MPa. The unit base resistance is calculated as follows:

$$q_b = K_c q_{ca} = 0.3 \times 49.1 = 14.733\ MPa$$

$$Q_b = A_{bg} q_b = B_g L_g q_b = 3.55 \times 3.55 \times 14.733 = 185.673\ MN$$

Using the alpha method, the unit shaft resistance is calculated as follows:

The soil is a sand with q_c=51.8 MPa, so α_{LCPC}=150.

$$f_p = \frac{q_c}{\alpha_{LCPC}} = \frac{51.8}{150} = 0.345$$

$$f_{p(\max)} = 0.12\ < 0.345$$

So f_p now becomes 0.12.

The average f_p value along the length of this 24.5m pile is f_p=0.0363 MPa.

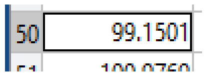
$$Q_s = A_s f_p = 0.0363 \times 3.55 \times 4 \times 24.5 = 12.629\ MN$$

$$Q_{ult} = Q_s + Q_b = 12.629 + 185.673 = 198.302MN$$

Applying a safety factor of 2,

$$Q_{all} = \frac{198.302}{2} = 99.150\ MN$$

This is very close to the 99.1501 MN value provided by MATLAB; thus, the program results are verified by this hand calculation. (View figure 6.4 for the MATLAB output).



50	99.1501

Figure 6.4: MATLAB result (MN) at depth 24.5m

Units MN

6.2 Verification of MATLAB Results – Individual Method

To verify the results produced by the MATLAB program for the pile group, a hand calculation is carried out within this section using the individual method, a more accurate approach than the Block Failure method in Section 6.1.

Firstly, the unit base resistance is calculated at a depth of 15.5m (chosen at random for cross check) using equation (12) as with a single pile in Section 4.1.1. At depth 15.5m, the soil type if clay with q_c=4.2 MPa, therefore K_c=0.35.

$$q_b = K_c q_{ca} \tag{12}$$

q_{ca} is the average unit base resistance. As with a single pile;

$$\alpha = \frac{3}{2} \times 0.64 = 0.6m$$

The provided data isn't accurate to 0.1m increments, so for these calculations, α=±0.5m.

$$q_{c1} = 4.5\ MPa\ @\ 15.5m - 0.5m = 15.0m$$

$$q_{c2} = 4.2\ MPa\ @\ 15.5m$$

$$q_{c3} = 5.8\ MPa\ @\ 15.5m + 0.5m = 16.0m$$

$$q_{ca} = \frac{4.5 + 4.2 + 5.8}{3} = 4.833\ MPa$$

Now, check the q_c values are in the required range:

$$0.7q_{ca}\ to\ 1.3q_{ca}$$

$$3.3831\ to\ 6.2829$$

All q_c values used to calculate q_{ca} are in this range, so q_{ca} retains its value of 4.833 MPa.

Therefore, using equation (3):

$$q_b = 0.35 \times 4.833 = 1.692\ MPa$$

Thus,

$$Q_b = (\pi \times 0.2^2) \times 1.692 = 0.2126\ MN$$

As in section 3.2, the unit shaft resistance is calculated using equation (14):

$$f_p = \frac{q_c}{\alpha_{LCPC}} \tag{14}$$

At depth 15.5m, the soil is classified as clay with q_c=4.2 MPa, so α_{LCPC}=40. Using equation (14);

$$f_p = \frac{4.2}{40} = 0.105\ MPa$$

$$f_{p(\max)} = 0.035\ < 0.105$$

Therefore, f_p = 0.035 MPa.

The average shaft resistance along the 15.5m length of the pile is $f_{p(av)}$ = 0.0307 MPa.

$$Q_s = 0.0307 \times (\pi \times 0.4) \times 15.5 = 0.598\ MN$$

$$Q_{ult} = Q_s + Q_b = 0.598 + 0.2126 = 0.8106\ MN$$

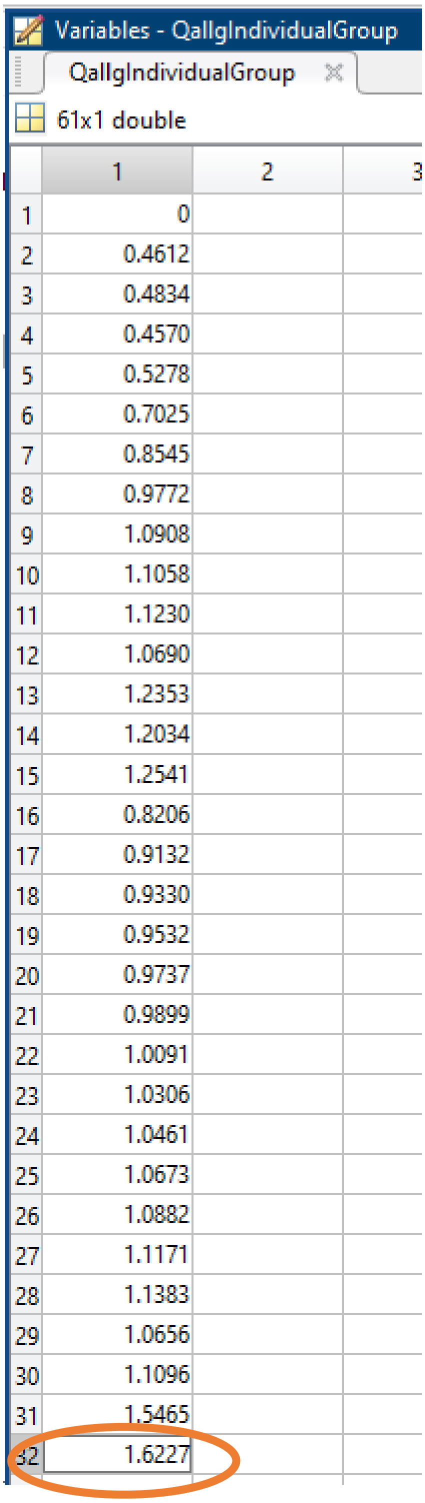
With a group of 4 piles,

$$Q_{ult(group)} = 0.8106 \times 4 = 3.242\ MN$$

Applying a Safety Factor of 2,

$$Q_{all(group)} = 1.621\ MN$$

This is very close to the 1.623 MN obtained by the MATLAB program, with the discrepancy likely due to rounding errors within the hand calculation. Thus, the MATLAB program is verified for the calculation of pile group capacity. (View figure 6.5 to see the MATLAB output at depth).



Variables - QallgIndividualGroup

QallgIndividualGroup

61x1 double

	1	2	3
1	0		
2	0.4612		
3	0.4834		
4	0.4570		
5	0.5278		
6	0.7025		
7	0.8545		
8	0.9772		
9	1.0908		
10	1.1058		
11	1.1230		
12	1.0690		
13	1.2353		
14	1.2034		
15	1.2541		
16	0.8206		
17	0.9132		
18	0.9330		
19	0.9532		
20	0.9737		
21	0.9899		
22	1.0091		
23	1.0306		
24	1.0461		
25	1.0673		
26	1.0882		
27	1.1171		
28	1.1383		
29	1.0656		
30	1.1096		
31	1.5465		
32	1.6227		

Figure 6.5: MATLAB output, highlighting the Qall at depth 15.5m. Units: MN

7. Conclusion and Summary

Summary of findings and recommendations to client.

- Shallow foundations should not be used.
- Single pile foundations should not be used.
- Group pile foundations as design in section 6, can be taken forward for further testing (further tests are recommended at the end of sections 5 and 6).
- For the group pile arrangement, pile lengths of 24m is suitable for withstanding the maximum structural load from the column.

8. References

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9. Appendices

Soil 1 ZONE 6

No	Depth (m)	q_c (MPa)	f_s (kPa)	u0 (kPa)	u(kPa)	qc (kPa)	qt (kPa)	Rf	γ_s (kN/m3)	σ_vo (kPa)	σ'_vo (kPa)	Qt	Fr (%)		u2 (kPa)
1	0.5	4.5	12.2	0.001	0.5	4500	4500	0.270	16.778	8.389	7.889	569.345	0.271		0
2	1	3.8	12.4	0.001	1	3800	3800	0.326	16.734	16.734	15.734	240.451	0.328		0
3	1.5	3.5	12.3	0.001	0.2	3500	3500	0.351	16.692	25.039	24.839	139.902	0.354		0
4	2	2.4	12.1	0.001	0.3	2400	2400	0.504	16.525	33.049	32.749	72.274	0.511		0
5	2.5	4.8	12.8	0.001	0.1	4800	4800	0.266	16.861	42.152	42.052	113.142	0.269		0
6	3	5.7	13.9	0.001	0.3	5700	5700	0.243	17.023	51.069	50.769	111.267	0.245		0
7	3.5	6.3	13.6	0.001	1.2	6300	6300	0.215	17.037	59.628	58.428	106.805	0.217		0
8	4	6.1	14.3	0.001	1	6100	6100	0.234	17.085	68.338	67.338	89.573	0.237		0
9	4.5	6.8	13.3	0.001	0.5	6800	6800	0.195	17.040	76.681	76.181	88.255	0.197		0
10	5	4.2	11.4	0.001	2	4200	4200	0.272	16.679	83.393	81.393	50.577	0.278		0
11	5.5	5.3	11.6	0.001	2	5300	5300	0.218	16.781	92.295	90.295	57.675	0.222		0
12	6	3.2	12.5	0.001	5	3200	3200	0.391	16.676	100.058	95.058	32.611	0.403		0
13	6.5	5.6	13.8	0.001	0.1	5600	5600	0.246	17.011	110.572	110.472	49.691	0.251		0
14	7	4.5	11.5	0.001	4	4500	4500	0.256	16.712	116.982	112.982	38.794	0.262		0
15	7.5	0.8	12.5	0.001	3.2	770	770	1.623	16.119	120.896	117.696	5.515	1.926		0
Mean												117.725	0.398		

Soil 1 has been identified to be in Zone 6 of the Normalised SBT Chart; Sand to Silty Sand.

Soil
2 ZONE 3

No	Depth (m)	q_c (MPa)	f_s (kPa)	u0(kPa)	u(kPa)	qc (kPa)	qt (kPa)	Rf	γ_s (kN/m3)	σ_{vo} (kPa)	σ'_{vo} (kPa)	Qt	Fr (%)		u2 (kPa)
1	8	0.7	39.6	0.001	11	700	700	5.657	17.434	139.475	128.475	4.363	7.065		0
2	8.5	0.7	39.12	0.001	2.2	700	700	5.589	17.420	148.071	145.871	3.784	7.088		0
3	9	0.83	35.78	0.001	0.1	830	830	4.311	17.382	156.438	156.338	4.308	5.312		0
4	9.5	0.74	38.22	0.001	1.5	740	740	5.165	17.414	165.438	163.938	3.505	6.652		0
5	10	0.75	39.02	0.001	0.5	750	750	5.203	17.444	174.440	173.940	3.309	6.779		0
6	10.5	0.75	38.96	4.905	5.2	750	751.125	5.187	17.443	183.150	177.950	3.192	6.859		5
7	11	0.75	39.81	9.81	10.3	750	752.250	5.292	17.469	192.156	181.856	3.080	7.108		10
8	11.5	0.83	39.84	14.715	13.1	830	833.375	4.781	17.510	201.361	188.261	3.357	6.304		15
9	12	0.65	45	19.62	21.6	650	654.500	6.875	17.558	210.696	189.096	2.347	10.140		20
10	12.5	0.82	52.1	24.525	30.1	820	825.625	6.310	17.821	222.757	192.657	3.129	8.642		25
11	13	0.89	35.8	29.43	29.8	890	896.750	3.992	17.413	226.368	196.568	3.410	5.340		30
12	13.5	0.95	37.2	34.335	37.3	950	957.875	3.884	17.484	236.029	198.729	3.632	5.153		35
13	14	0.89	41	39.24	44.6	890	899	4.561	17.573	246.021	201.421	3.242	6.279		40
Mean												3.435	6.825		

Soil 2 has been identified to be in Zone 3 of the Normalised SBT Chart; Clay to Silty Clay.

Soil 3

No	Depth (m)	q_c (MPa)	f_s (kPa)	u0 (kPa)	U (kPa)	qc (kPa)	qt (kPa)	Rf	γ_s (kN/m3)	σ_vo (kPa)	σ'_vo (kPa)	Qt	Fr (%)		u2
1	14.5	4.7	102	44.145	45.1	4700	4710.125	2.166	19.289	279.690	234.590	18.886	2.302		45
2	15	4.5	121	49.05	50.6	4500	4511.250	2.682	19.472	292.086	241.486	17.472	2.868		50
3	15.5	4.2	130.2	53.955	58.9	4200	4212.375	3.091	19.532	302.739	243.839	16.034	3.330		55
4	16	5.8	168.2	58.86	65.1	5800	5813.500	2.893	19.958	319.324	254.224	21.612	3.061		60
5	16.5	4.4	145.2	63.765	76.2	4400	4414.625	3.289	19.678	324.682	248.482	16.460	3.550		65
6	17	5.7	165.3	68.67	77	5700	5715.750	2.892	19.931	338.822	261.822	20.537	3.074		70
7	17.5	3.8	95	73.575	78	3800	3816.875	2.489	19.123	334.659	256.659	13.567	2.728		75
8	18	4.7	126.9	78.48	88	4700	4718.000	2.690	19.546	351.823	263.823	16.550	2.906		80
9	18.5	5.9	182.9	83.385	88.4	5900	5919.125	3.090	20.063	371.166	282.766	19.620	3.297		85
10	19	4.4	118.8	88.29	98	4400	4420.250	2.688	19.443	369.415	271.415	14.925	2.933		90
11	19.5	5.5	192.5	93.195	100.4	5500	5521.375	3.486	20.096	391.868	291.468	17.599	3.753		95
12	20	5.5	170.5	98.1	110	5500	5522.500	3.087	19.954	399.072	289.072	17.724	3.328		100
13	20.5	4	120	103.005	118	4000	4023.625	2.982	19.418	398.068	280.068	12.945	3.310		105
14	21	5.6	179.2	107.91	108.5	5600	5624.750	3.186	20.019	420.401	311.901	16.686	3.443		110
15	21.5	5.3	143.1	112.815	110	5300	5325.875	2.687	19.734	424.281	314.281	15.596	2.919		115
16	22	5.5	159.5	117.72	130	5500	5526.775	2.886	19.876	437.265	307.265	16.564	3.134		119
17	22.5	5.3	153.7	122.625	129	5300	5327.675	2.885	19.818	445.903	316.903	15.405	3.148		123
18	23	5.5	187	127.53	137	5500	5529.025	3.382	20.062	461.434	324.434	15.620	3.690		129
19	23.5	6	150	132.435	134	6000	6030.375	2.487	19.838	466.187	332.187	16.750	2.696		135
						Mean						16.871	3.130		

SoiL 3 has been identified to be in zone 3 of the Normalised SBT chart; Clay to Silty Clay.

Soil 4

No	Depth (m)	q_c (MPa)	f_s (kPa)	u0 (kPa)	U (kPa)	qc (kPa)	qt (kPa)	Rf	γ_s (kN/m3)	□_vo (kPa)	□'_vo (kPa)	Qt	Fr (%)		u2
1	24	44.4	316	137.34	153	44444	44475.719	0.711	21.492	515.819	362.819	121.162	0.719		139
2	24.5	51.8	333.19	142.245	164	51778	51809.728	0.643	21.614	529.549	365.549	140.283	0.650		142
3	25	51.1	248.4	147.15	188	51111	51144.636	0.486	21.265	531.621	343.621	147.293	0.491		149
4	25.5	46.7	338.1	152.055	197	46667	46700.867	0.724	21.591	550.566	353.566	130.528	0.733		152
5	26	50.2	264.42	156.96	198	50222	50257.547	0.526	21.331	554.613	356.613	139.375	0.532		157
6	26.5	49.8	374.08	161.865	196	49778	49814.453	0.751	21.735	575.968	379.968	129.586	0.760		163
7	27	46.4	219.45	166.77	176	46444	46482.019	0.472	21.082	569.218	393.218	116.762	0.478		167
8	27.5	51.3	291.06	171.675	174	51333	51372.033	0.567	21.452	589.941	415.941	122.090	0.573		172
9	28	48.2	301.63	176.58	191	48222	48262.047	0.625	21.470	601.155	410.155	116.202	0.633		177
10	28.5	54.0	374.22	181.485	199	54000	54040.950	0.692	21.767	620.356	421.356	126.782	0.701		182
11	29	54.4	279.3	186.39	196	54444	54486.294	0.513	21.427	621.384	425.384	126.626	0.519		186
12	29.5	50.4	240.62	191.295	196	50444	50487.419	0.477	21.222	626.063	430.063	115.940	0.483		191
13	30	52.2	235	196.2	198	52222	52266.322	0.450	21.208	636.248	438.248	117.810	0.455		196
Mean												126.957	0.594		

Soil 4 has been identified to be in zone 6 of the Normalised SBT chart; Sands to Silty sands.

B (m)	Z (m)	Depth below footing (m)	$P_{present}$ (kPa)	γD (kPa)	ΔP (kPa)	$q_{c(av)}$ (kPa)	q_{all} (kPa)	C_1	C_2	C_3	α_E	E' (kPa)	I_z	Δz (m)	Δs (m)
1.5	0.0	0.0	2057.8	16.7	2041.0	3800.0	1266.7	0.996	1	1.2	10	12667	0.100	0.5	0.00669
1.5	0.5	0.5	2057.8	16.7	2041.1	3650.0	1216.7	0.996	1	1.2	10	12167	0.367	0.5	0.02552
1.5	1.0	1.0	2057.8	16.5	2041.3	3233.3	1077.8	0.996	1	1.2	10	10778	0.445	0.5	0.03494
1.5	1.5	1.5	2057.8	16.9	2040.9	3625.0	1208.3	0.996	1	1.2	10	12083	0.333	0.5	0.02337
1.5	2.0	2.0	2057.8	17.0	2040.8	4040.0	1346.7	0.996	1	1.2	10	13467	0.222	0.5	0.01398
1.5	2.5	2.5	2057.8	17.0	2040.7	4416.7	1472.2	0.996	1	1.2	10	14722	0.111	0.5	0.00640
1.5	3.0	3.0	2057.8	17.1	2040.7	4657.1	1552.4	0.996	1	1.2	10	15524	0.000	0.5	0.00001

Total Settlement (m):	**0.1109**

Figure 9.1: Settlement Calculation for a 1.5m Square Footing using Schertmann Method

B (m)	Z (m)	Depth below footing (m)	$P_{present}$ (kPa)	γD (kPa)	ΔP (kPa)	$q_{c(av)}$ (kPa)	q_{all} (kPa)	C_1	C_2	C_3	α_E	E' (kPa)	I_z	Δz (m)	Δs (m)
2.0	4.0	0.0	1157.5	16.7	1140.8	3800.0	1266.7	0.993	1	1.2	10	12667	0.100	0.5	0.00373
2.0	4.0	0.5	1157.5	16.7	1140.8	3650.0	1216.7	0.993	1	1.2	10	12167	0.300	0.5	0.01164
2.0	4.0	1.0	1157.5	16.5	1141.0	3233.3	1077.8	0.993	1	1.2	10	10778	0.500	0.5	0.02190
2.0	4.0	1.5	1157.5	16.9	1140.6	3625.0	1208.3	0.993	1	1.2	10	12083	0.417	0.5	0.01627
2.0	4.0	2.0	1157.5	17.0	1140.5	4040.0	1346.7	0.993	1	1.2	10	13467	0.333	0.5	0.01167
2.0	4.0	2.5	1157.5	17.0	1140.5	4416.7	1472.2	0.993	1	1.2	10	14722	0.250	0.5	0.00801
2.0	4.0	3.0	1157.5	17.1	1140.4	4657.1	1552.4	0.993	1	1.2	10	15524	0.167	0.5	0.00506
2.0	4.0	3.5	1157.5	17.0	1140.5	4925.0	1641.7	0.993	1	1.2	10	16417	0.083	0.5	0.00239
2.0	4.0	4.0	1157.5	16.7	1140.8	4844.4	1614.8	0.993	1	1.2	10	16148	0.000	0.5	0.00000

Total Settlement (m):	**0.0807**

Figure 9.2: Settlement Calculation for a 2.0m Square Footing using Schertmann Method

B (m)	Z (m)	Depth below footing (m)	$P_{present}$ (kPa)	γD (kPa)	ΔP (kPa)	$q_{c(av)}$ (kPa)	q_{all} (kPa)	C_1	C_2	C_3	α_E	E' (kPa)	I_z	Δz (m)	Δs (m)
2.5	5.0	0.0	740.8	16.7	724.1	3800.0	1266.7	0.989	1	1.2	10	12667	0.100	0.5	0.00235
2.5	5.0	0.5	740.8	16.7	724.1	3650.0	1216.7	0.989	1	1.2	10	12167	0.260	0.5	0.00637
2.5	5.0	1.0	740.8	16.5	724.3	3233.3	1077.8	0.989	1	1.2	10	10778	0.420	0.5	0.01163
2.5	5.0	1.5	740.8	16.9	723.9	3625.0	1208.3	0.989	1	1.2	10	12083	0.467	0.5	0.01152
2.5	5.0	2.0	740.8	17.0	723.8	4040.0	1346.7	0.989	1	1.2	10	13467	0.400	0.5	0.00886
2.5	5.0	2.5	740.8	17.0	723.8	4416.7	1472.2	0.989	1	1.2	10	14722	0.333	0.5	0.00675
2.5	5.0	3.0	740.8	17.1	723.7	4657.1	1552.4	0.988	1	1.2	10	15524	0.267	0.5	0.00512
2.5	5.0	3.5	740.8	17.0	723.8	4925.0	1641.7	0.988	1	1.2	10	16417	0.200	0.5	0.00363
2.5	5.0	4.0	740.8	16.7	724.1	4844.4	1614.8	0.989	1	1.2	10	16148	0.134	0.5	0.00247
2.5	5.0	4.5	740.8	16.8	724.0	4890.0	1630.0	0.989	1	1.2	10	16300	0.067	0.5	0.00122
2.5	5.0	5.0	740.8	16.7	724.1	4736.4	1578.8	0.989	1	1.2	10	15788	0.000	0.5	0.00000

Total Settlement (m):	**0.0599**

Figure 9.3: Settlement Calculation for a 2.5m Square Footing using Schertmann Method

B (m)	Z (m)	Depth below footing (m)	$P_{present}$ (kPa)	γD (kPa)	ΔP (kPa)	$q_{c(av)}$ (kPa)	q_{all} (kPa)	C_1	C_2	C_3	α_E	E' (kPa)	I_z	Δz (m)	Δs (m)
3.0	6.0	0.0	514.4	16.7	497.7	3800.0	1266.7	0.984	1	1.2	10	12667	0.100	0.5	0.00161
3.0	6.0	0.5	514.4	16.7	497.8	3650.0	1216.7	0.984	1	1.2	10	12167	0.233	0.5	0.00391
3.0	6.0	1.0	514.4	16.5	497.9	3233.3	1077.8	0.984	1	1.2	10	10778	0.367	0.5	0.00695
3.0	6.0	1.5	514.4	16.9	497.6	3625.0	1208.3	0.984	1	1.2	10	12083	0.500	0.5	0.00844
3.0	6.0	2.0	514.4	17.0	497.4	4040.0	1346.7	0.983	1	1.2	10	13467	0.445	0.5	0.00673
3.0	6.0	2.5	514.4	17.0	497.4	4416.7	1472.2	0.983	1	1.2	10	14722	0.389	0.5	0.00538
3.0	6.0	3.0	514.4	17.1	497.4	4657.1	1552.4	0.983	1	1.2	10	15524	0.333	0.5	0.00438
3.0	6.0	3.5	514.4	17.0	497.4	4925.0	1641.7	0.983	1	1.2	10	16417	0.278	0.5	0.00345
3.0	6.0	4.0	514.4	16.7	497.8	4844.4	1614.8	0.984	1	1.2	10	16148	0.222	0.5	0.00281
3.0	6.0	4.5	514.4	16.8	497.7	4890.0	1630.0	0.984	1	1.2	10	16300	0.167	0.5	0.00209
3.0	6.0	5.0	514.4	16.7	497.8	4736.4	1578.8	0.984	1	1.2	10	15788	0.111	0.5	0.00144
3.0	6.0	5.5	514.4	17.0	497.4	4808.3	1602.8	0.983	1	1.2	10	16028	0.056	0.5	0.00071
3.0	6.0	6.0	514.4	16.7	497.7	4784.6	1594.9	0.984	1	1.2	10	15949	0.000	0.5	0.00000

Total Settlement (m):	**0.0479**

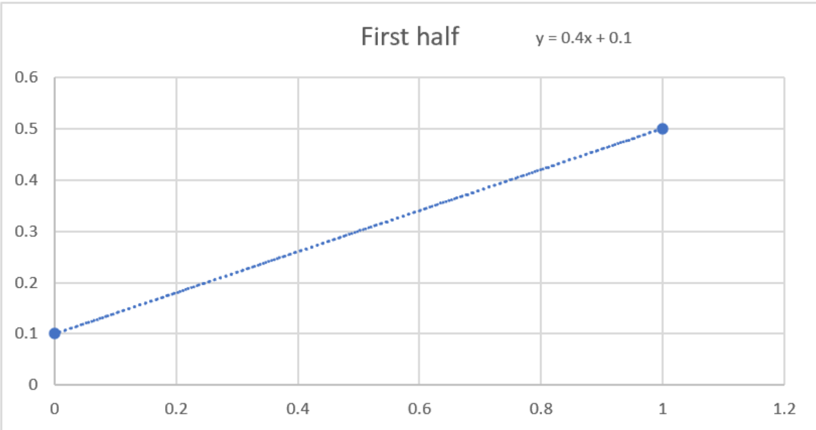
Figure 9.4: Settlement Calculation for a 3.0m Square Footing using Schertmann Method

x	y
0	0.1
1	0.5
4	0

Z	Iz for x<=B/2
0	0.1
0.5	0.3
1	0.5

Iz VALUES FOR B=2M

Z	Iz for x=>B/2
1	0.5
1.5	0.41665
2	0.3333
2.5	0.24995
3	0.1666
3.5	0.08325
4	-1E-04



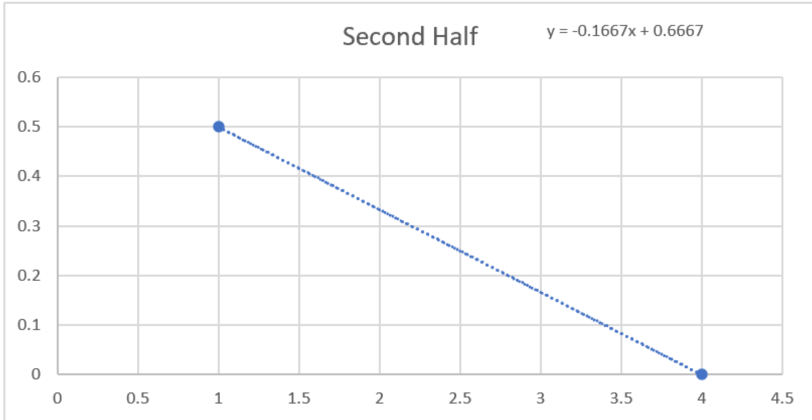


Figure 9.5: Example of how I_z can be interpolated for each soil layer

Code Methodology

The choice of programming language is MATLAB 2020. This program is suitable and appropriate as it is very useful for data analysis, due to its range of functions and storage capabilities.

Firstly, the soil data is stored in excel. The first step is to make a script that extracts the full data set from excel and stores it in a MATLAB matrix.

The data is now arranged in a MATLAB matrix. Key values which need to be extracted include; depth and the q_c values for each depth. This can be completed by implementing either a loop (time inefficient) or simple direct extraction (the used method in this program and quicker). The extraction code for the depths is as follows: `depth=Design3SoilProfileDataS2{1:61,2};`
It is important to use braces as the brackets, so the data is extracted and stored in a matrix and not as a table.

Next, extract the q_c data for each depth, similarly, direct extraction is done instead of a loop (time efficient). `qcMPA=Design3SoilProfileDataS2{1:61,3};`

A range of vectors are sized, just incase the reader/user wants to manually check the values of k_c (stored in vector kc), q_{ca} (stored in vector qcavalues), q_b (stored in vector qbvalues), f_p (stored in vector fpvalues) (fpbarvalues is the average of all the fpvalues for all the depths above a given depth). This is implemented by the following code:

```
kc=zeros(leng,1);
qcavalues=zeros(leng,1);
qbvalues=zeros(leng,1);
fpvalues=zeros(leng,1);
```

A Q_{ult} zeros vector is made where the Q_{ult} (ultimate bearing capacity) values for each depth is stored.

```
Qult=zeros(leng,1);
```

Where leng is simply the length of the depth (number of rows). This is so the code knows how many various depths is being programmed for.

The Q_{ult} is then considered at each depth by initiating a for loop, which will loop through all of the rows in each vector of data. `for c=2:leng`
The loop starts at row 2, because row 1 in this code represents the ground surface, where Q_{ult} is 0.

Extracting the specific depth and qc (MPa) by using the for loop with the variable c. This is programmed as follows:

```
depthi=depth(c);
qcMPAi=qcMPA(c);
```

The next step is to extract the k_c (bearing capacity factor). This depends on the soil type at that specific point in-depth, and the value q_c (MPa) at that specific depth. The first soil is loose and silty sands (view section 1 for the soil profile). The k_c values which apply to this soil are then extracted based on the q_c (MPa) value, which has been extracted and stored as qcMPAi. For the first soil, with a range of q_c (MPa) criteria available the code is as follows:

```
if c <= 15         %Applying kc value for Soil 1= Loose and Silty Sands
        if qcMPAi <=5
            kci=0.4;
        elseif qcMPAi >5 && qcMPAi <=12
            kci=0.4;
        else qcMPAi>12
            kci=0.3;
        end
```

Next, will be analysing the q_c (MPa) values for the next soil in the soil profile. Silty clays/clays are present from depth 8m – 23.5m (inclusive) therefore the upcoming codes knows that within this depth range the k_c values which are relevant to silty clays/clays must be extracted, the 'elseif' code allows the condition of soil type to switch.

```
elseif c >=16 && c<= 47  % Applying kc values for Soil 2 and 3 = Silty
Clays to Clay
       if qcMPAi<1
            kci=0.4;
       elseif qcMPAi >=1 && qcMPAi <=5
            kci=0.35
       elseif qcMPAi >5 && qcMPAi <=12
            kci=0.45;
       else qcMPAi>12
            kci=0.45;
       end
```

Finally, the soil profile switches back to sands for the final depth in the range 24m-30m (inclusive). This is implemented via the same methodology as above, with different k_c values of sand being used as a result of different q_c (MPa) values.

```
else c >= 48  %Applying kc values for loose/ silty sands
    if qcMPAi<1
            kci=0.4;
    elseif qcMPAi >=1 && qcMPAi <=5
            kci=0.4;
    elseif qcMPAi >5 && qcMPAi <=12
            kci=0.4;
    else qcMPAi>12
            kci=0.3;
    end
end
```

The next step is calculating the value of a, which is used to calculate the range of q_c values used to calculate q_{ca}. With the q_c values in the range: depth$\pm a$. `ai=1.5*0.4; %a=3/2*diameter`

The lower and upper depths are coded and stored as follows at each depth by the following code:

```
DL=depthi-ai; %Calculating lower depth value based on a
DU=depthi+ai; %Calculating upper depth value based on a
```

In the case of our data, we have q_c values for each 0.5m. With a=0.6m, this is close to 0.5m and we don't have the accuracy of data to be specific to know what q_c is at 0.1m intervals, the data is in terms of 0.5m, and therefore a is now rounded to 0.5m (this is an important depth interval, as the values of 0.6m q_c is not known, 0.5m interval q_c depths are known), as the values of q_c are known, and no assumptions are made, this will results in an accurate answer. 'a' is rounded by the following formula: `ai=round(ai/0.5)*0.5;`

The q_{ca} value is calculated by taking the average of the individual q_c values within the range $\pm a$ of the depth at this specific point in the loop (variable= depthi). Each value of 'c' represents 0.5m depth. The range of values are stored into qcprime, the q_{ca} is then calculated from this.

```
qcprime=zeros(1,3);
```

```
qcprime(1)=qcMPA(c-1);
qcprime(2)=qcMPA(c);
qcprime(3)=qcMPA(c+1);
```

The q_{ca} value can now be calculated by taking the average of these 3 q_c values:

```
qca=sum(qcprime,2) ./ sum(qcprime~=0,2);
```

The next step is to check if the q_c values used to calculate the above variable qca, are within the range $0.7*q_{ca}$-$1.3*q_{ca}$. If the q_c value for a given depth is out of this range it will be set to the next appropriate value e.g. $0.7*q_{ca}$ or $1.3*q_{ca}$ (an example of this is shown in figure 9.6 below). This is implemented by the following code:

```
    for i=1:3 %Iterating through our 3 values of qc used for qca
        if qcprime(i)<0.7*qca %Below 0.7*qca then edit the value
            qcprime(i)=0.7*qca;

        end
        if qcprime(i)>1.3*qca %Greater then 1.3*qca then edit the value
            qcprime(i)=1.3*qca;
        end
    end
```

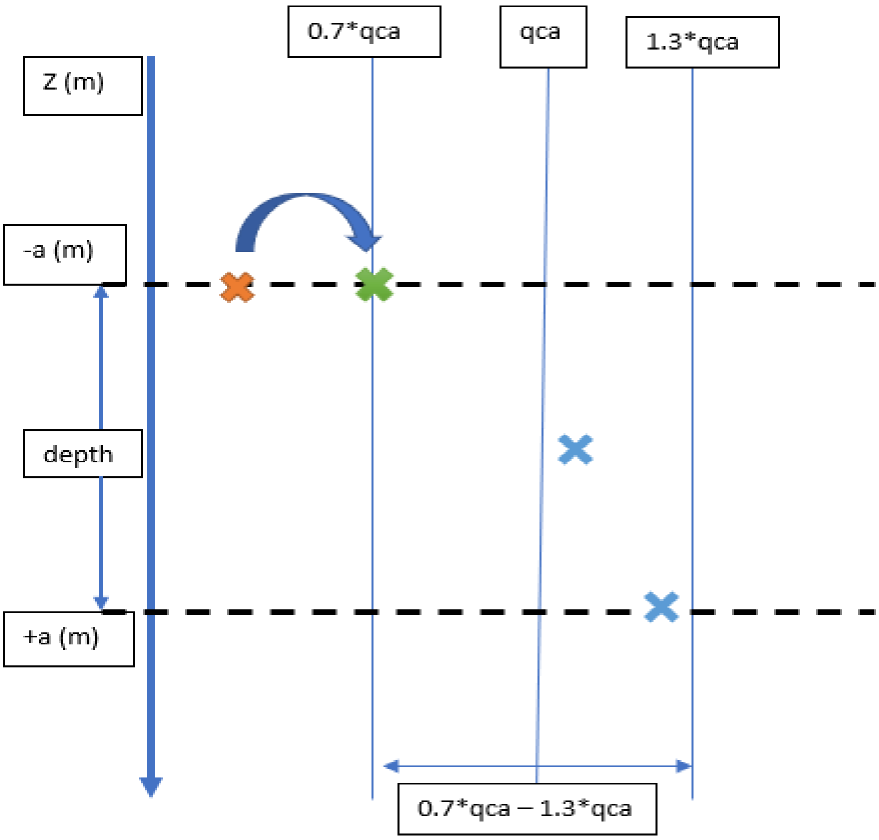


Figure 9.6: Displaying how the qc values are updated if they are out of the specific range

Then the q_{ca} can be re-calculated, excluding any q_c values which were outside this stated range $0.7*q_{ca}$-$1.3*q_{ca}$.

```
qca=sum(qcprime,2) ./ sum(qcprime~=0,2);

    if isnan(qca)
        qca=0;
    end
    qcavalues(c)=qca;
```

Finally, the value Q_b can be calculated by using q_b and the area of the base (this occurs for every depth, implemented by the for loop at the start loop through each 0.5m interval of depth):

```
qb=kci*qca;
Ab=pi*0.2^2;
Qb=qb*Ab;
```

Next is to calculate f_p. For this, a value of $alpha_{LCPC}$ is needed. This methodology is similar to the extraction of the value k_c based on soil type and various values of q_c (MPa) at different depths.

A main 'if', 'elseif' and 'else' loop is in place to separate the various soil types. Within each of these loops is another series of 'if', 'elseif' and 'else' loops. The first series of loops classifies the various soil types, the next series will use the q_c values for the specific soil to extract and store the following values; $alpha_{LCPC}$ and the $f_{p,max}$ (which will be used later as a checker). This is implemented in code form as follows:

```
%FINDING ALPHA FOR EACH SOIL TYPE UNDER DIFFERENT LOADS
    if c <= 15 %Loose sand and silty sand
        if qcMPAi<=5
            aplhai=60;
            fpmax=0.035;
        elseif qcMPAi >5 && qcMPAi <=12
            aplhai=100;
            fpmax=0.08;
        else qcMPAi>12
            aplhai=150;
            fpmax=0.12;
        end
    elseif c >=16 && c<= 47 %CLAYS
        if qcMPAi<1
            aplhai=30;
            fpmax=0.015;
        elseif qcMPAi >=1 && qcMPAi <5
            aplhai=0.40;
            fpmax=0.035;
        elseif qcMPAi >=5 && qcMPAi <=12
            aplhai=60;
            fpmax=0.035;
        else qcMPAi>12
            aplhai=60;
            fpmax=0.035;
        end
    else c >= 48 % Loose Sands
        if qcMPAi<=5
            aplhai=60;
            fpmax=0.035;
        elseif qcMPAi >5 && qcMPAi <=12
```

```
            aplhai=100;
            fpmax=0.08;
        else qcMPAi>12
            aplhai=150;
            fpmax=0.12;
        end
    end
```

At the specific point in depth, the f_p is calculated as follows:

```
fp=qcMPAi/aplhai;
```

Now the checker using $f_{p,max}$ which was mentioned above and stored as variable fpmax is used.

This is checked by an 'if' condition loop. The condition is as follows (using variable names), 'if' fp is less than fpmax, then fp keeps its value. However, 'if' fp is greater than fpmax, then fp is set to take the fpmax value. (The fpmax was found for the specific soil type and q_c value from the above code). This if condition checker is implemented by the following code:

```
if fp>fpmax
     fp=fpmax;
end
```

(Check table 18 to visualise if f_p that was calculated using a_{LCPC} or if fp has been set to $f_{p,max}$ has been used instead at all the depths). Green represents the a_{LCPC} f_p was used, Yellow mean $f_{p,max}$ has been used). All the f_p values for each depth are stored, this allows the next stage of the programme to take the average of all these unit shaft resistances along the length of the pile. The storage procedure is completed by the following code:

```
fpvalues(c)=fp; %Store the unit shaft resistance at each depth.
```

Now that fp has been identified and checked with fpmax there is another step before calculating the Q_s value. This step is taking the average of the unit shaft resistances at all the points along the length of the pile for that given depth/length. This is computed by the following code:

```
fpbar=sum(fpvalues(2:c,1))/(c-1); %Average, represented by fp bar
fpbarvalues(c)=fpbar; %Storing this fp average
```

The value Q_s can be calculated. This is computed by the following code:

```
As=pi*0.4*depth(c);
Qs=fpbar*As;
```

For each depth, the Q_{ult} of a single pile is calculated by simply summing Q_s and Q_b (the base resistance and shaft resistance). This is all within a 'for' loop which ranges over each depth, so it must have a final end present.

```
% calculating Qult at each depth
QultSingle(c,1)=Qb+Qs;

end
```

Finally, the last step is to apply a Factor of Safety of 2, to calculate the Q allowable. This is computed in the program as follows:

```
%Considering factor of safety 2; calculating Q allowable
QallSingle=QultSingle/2;
```

From analysing the Q allowable of a single pile, it was determined that a single pile is not sufficient (discussed in section 4). The upcoming code calculates the Q allowable for the group pile suggestion in sections 5 and 6). Note: there are two methods to calculate the group bearing capacity; Block failure and individual failure multiplied by the number of piles in used (4 in the suggested design).

The unit base resistance and unit shaft resistance has already been calculated and stored in vectors using the above code. The upcoming code will make use of these values but the shaft area and base areas will be increased (as it is now a group pile).

The block group bearing capacity is calculated using the upcoming code, where the base area and shaft area is idealised as a square and a cuboid respectively. The width of this idealised square is B_g=L_g which was determined based on the centre-to-centre spacing and diameters of the piles. A factor of safety of 2 is also used to change the ultimate bearing capacity of the block to the allowable bearing capacity. The code is as follows:

```
%BLOCK FAILURE GROUP PILES
QugBlock=zeros(leng,1);

for c=2:leng
    depthi=depth(c);
    QugBlock(c)=qbvalues(c)*Bg*Lg+fpvalues(c)*(Bg*4*depthi);
end
QallgBlock=QugBlock/2;
```

Next, is calculating the Group failure by considering individual piles capacity and the number of piles within the group. A factor of safety of 2 is also applied. The code for this is as follows:

```
%INDIVIDUAL PILE GORUP MEHTOD
QugIndividualGroup=zeros(leng,1);

for c=2:leng
    depthi=depth(c);
    QugIndividualGroup(c)=QultSingle(c)*4;
end
QallgIndividualGroup=QugIndividualGroup/2;
```

Finally, to check the efficiency of the pile group, eta is determined. This will also show if the centre-to-centre spacing of the pile group is poor, causing an overlap in the zone of influence of adjacent piles. This is coded as follows:

```
eta=QallgBlock./QallgIndividualGroup;
```

Finally, a quick check using the beta method is completed for a single pile, this code uses the beta method formula as described in section 4.2. Note human error may be present when extracting the values of beta and N_t. This code is designed for various soil types (which is present in the soil profile. The different soil types are analysed by using multiple ‘for’ loops.

```
QultB=zeros(leng,1); %Sizing the Q ult bearing capacity which will store
Beta method results

sigma_v_prime=Design3SoilProfileDataS2(1:61,12);

sigma_v_prime(1)=0;

for i=1:15 %Iterating for sands


QultB(i)=0.8*sum(sigma_v_prime(1:i))*0.5*0.4/1000+60*sigma_v_prime(i)*pi*0.
04;

end

for i=16:47 % Iterating for clays

    QultB(i)=QultB(15)-
60*sigma_v_prime(15)*pi*0.04+0.5*sum(sigma_v_prime(16:i))*0.5*0.4/1000+30*s
igma_v_prime(i)*pi*0.04;

end

for i=48:leng %Iterating for sands

    QultB(i)=QultB(47)-
30*sigma_v_prime(47)*pi*0.04+0.8*sum(sigma_v_prime(16:i))*0.5*0.4/1000+60*s
igma_v_prime(i)*pi*0.04;

end

Qall_Beta_MN=QultB/2000; % Dividing by FoS 2 and 1000 to convert from kPa
to MPa.
Qall_Beta_kN=Qall_Beta_MN*1000;
```

Note for reader: The code file has been attached in the document submissions along with this document, please feel free to download it, test it and read the results of the variables, matrices, and vectors.

Green represents the calculated f_p value is used. Yellow represents the $f_{p,max}$ value has been used.

Table 18: fp (green) or fpmax (yellow) was used

Depth (m)	fp values
0	0
0.5	0.035
1	0.035
1.5	0.035
2	0.035
2.5	0.035
3	0.057
3.5	0.063
4	0.061
4.5	0.068
5	0.035
5.5	0.053
6	0.035
6.5	0.056
7	0.035
7.5	0.015
8	0.015
8.5	0.015
9	0.015
9.5	0.015
10	0.015
10.5	0.015
11	0.015
11.5	0.015
12	0.015
12.5	0.015
13	0.015
13.5	0.015
14	0.015
14.5	0.035
15	0.035
15.5	0.035
16	0.035
16.5	0.035
17	0.035
17.5	0.035
18	0.035
18.5	0.035
19	0.035
19.5	0.035

20	0.035
20.5	0.035
21	0.035
21.5	0.035
22	0.035
22.5	0.035
23	0.035
23.5	0.06
24	0.12
24.5	0.12
25	0.12
25.5	0.12
26	0.12
26.5	0.12
27	0.12
27.5	0.12
28	0.12
28.5	0.12
29	0.12
29.5	0.12
30	0.12

ϕ' (deg)	N_c	N_q	N_γ	ϕ' (deg)	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

Figure 9.7: Bearing Capacity Analysis (Sedighi, 2022)

Table 19: FINAL RESULTS of: Single plie, group pile individual method, group pile block method, eta, beta method for all depths

DEPTH (M)	Single Pile (kPa)	Group Bearing Cap - Block Method (kPa)	Group Bearing Cap - Individual Method (kPa)	Eta	BETA Method (kPa)
0	0.0	0.0	0.0	0	0.0
0.5	104.9	9538.3	419.5	23	24.8
1	120.8	10162.5	483.4	21	49.4
1.5	114.2	8522.4	457.0	19	78.0
2	133.1	9430.8	532.3	18	102.9
2.5	167.2	11879.5	668.9	18	132.1
3	213.6	14938.4	854.5	17	159.5
3.5	244.3	16254.3	977.2	17	183.6
4	272.7	17395.0	1090.8	16	211.6
4.5	276.5	15872.1	1105.8	14	239.4
5	280.8	15324.2	1123.0	14	255.7
5.5	267.2	12487.7	1069.0	12	283.7
6	290.7	13863.8	1162.9	12	298.7
6.5	300.9	13314.9	1203.4	11	347.1
7	299.0	12150.9	1196.0	10	355.0
7.5	250.4	6855.1	1001.5	7	148.0
8	228.3	4219.8	913.2	5	161.5
8.5	233.3	4298.2	933.0	5	183.4
9	238.3	4385.1	953.2	5	196.5
9.5	243.4	4480.3	973.7	5	206.1
10	247.5	4466.4	989.9	5	218.7
10.5	252.3	4528.0	1009.1	4	223.7
11	257.7	4648.5	1030.6	5	228.6
11.5	261.5	4617.7	1046.1	4	236.7

12	266.8	4729.8	1067.3	4	237.7
12.5	272.0	4833.4	1088.2	4	242.2
13	279.3	5138.7	1117.1	5	247.1
13.5	284.6	5250.8	1138.3	5	249.8
14	315.7	7955.6	1262.9	6	253.2
14.5	358.8	11294.0	1435.1	8	294.9
15	386.6	13109.9	1546.5	8	303.6
15.5	405.7	14042.8	1622.7	9	306.6
16	446.1	17118.1	1784.4	10	319.6
16.5	437.9	15320.5	1751.8	9	312.4
17	463.4	16894.0	1853.5	9	329.2
17.5	447.5	14319.2	1789.9	8	322.7
18	459.9	14590.5	1839.7	8	331.7
18.5	506.7	18306.5	2027.0	9	355.5
19	492.2	15868.2	1968.7	8	341.3
19.5	532.5	18933.0	2130.0	9	366.5
20	539.7	18679.2	2158.9	9	363.5
20.5	520.0	15726.4	2080.2	8	352.2
21	560.8	18833.2	2243.1	8	392.2
21.5	585.9	20375.2	2343.6	9	395.2
22	594.1	20215.9	2376.3	9	386.4
22.5	607.0	20529.2	2427.8	8	398.5
23	622.7	21126.1	2490.7	8	408.0
23.5	905.0	47762.0	3619.9	13	432.8
24	1227.4	76739.8	4909.5	16	529.1
24.5	1484.3	99150.1	5937.2	17	537.7
25	1536.0	100976.9	6143.8	16	468.8

25.5	1563.9	100422.4	6255.5	16	500.1
26	1593.2	100008.4	6372.8	16	509.6
26.5	1629.5	100293.9	6518.0	15	583.0
27	1674.2	101420.0	6696.7	15	624.7
27.5	1702.1	100865.5	6808.4	15	696.1
28	1787.3	106052.7	7149.1	15	678.0
28.5	1844.5	108439.1	7378.1	15	713.2
29	1896.2	110265.2	7584.8	15	725.8
29.5	1922.7	109570.8	7690.9	14	740.6
30	1844.1	98332.7	7376.4	13	766.3

Name (Block Capitals)	**Weighting to be applied to Group mark** (indicate + or - %)	**Discussed and agreed by all Group members** (Yes or No)
JOSEPH JOLLEY	0	YES
JAMIE SHUTTLEWORTH	0	YES
JINHONG LI	0	YES
KARLA KANGHARA	0	YES
ALIMA ELFRIDA	0	YES
WENG TANG	0	YES