Annex 1

DESIGN AND ANALYSIS OF FOUNDATIONS

Final Year Project UG 2012

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Annex 2

This is to certify that the

Final Year Project, titled

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"In the name of Almighty Allah, the Most Beneficent, the Most Merciful"

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ABSTRACT

For any building the safe transfer of the superstructure's load to the foundation is one of the most critical component of the design. At every building site due to varying geotechnical and geophysical conditions a unique design is required therefore advocating the detailed geotechnical analysis.

In this project we have divided the design and analysis into two major parts,

- Shallow Foundations
- Deep Foundations

In both these types of foundations we checked the Shear Criteria and Settlement Criteria for various shapes, conditions and methods of analysis and resulting in proposing the most suitable and economical design parameters for particular sites and conditions.

Moreover, the development of an extensively detailed computer program on Microsoft Excel and Microsoft Visual Studio Community Edition (using C# language) using various methods and techniques for foundation analysis and design which is designed as an end product of this project.

In the end, the user gets a detailed report about the bearing capacities of various types of foundations based on shear and settlement criteria by just putting the basic field parameter as inputs.

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CHAPTER 1

INTRODUCTION

1.1 General

Foundation is a part of a structure which provides support to the structure and the loads coming from it. Thus, foundation means the soil or rock that ultimately supports the load and any part of the structure that serves to transmit the load into the soil.

The design of foundations has evolved tremendously over the past years due to the extensive research on the behavior and properties of soils. Previously, very little consideration was given to the design of foundations. The ability of the structural element to transmit the load is limited by the capability of the soil to support the loads. Therefore, a foundation failure may destroy the superstructure as well while a failure in the superstructure might result only in the localized damage and does not necessarily mean failure of the foundation.

As a result, it is necessary to conduct extensive soil investigations in order to obtain accurate geotechnical properties. These values facilitate in determining the most appropriate foundation applicable to the given strata.

So, our group took this as our Undergraduate Final Year Project because we wanted to obtain a complete experience and understanding of the various engineering aspects related to foundation design and analysis which will inevitably be extremely beneficial in our professional career.

1.2 Objectives

The basic purpose of taking this project is to study in depth the vast set of geotechnical principals and techniques and their application in the real life problems. Our team is trying to achieve our objectives via following practices:

1.2.1 Relate the theoretical knowledge with practical application

Gaining the theoretical knowledge is also as important as gaining the technical and practical knowledge. Through this project, we gained all the required knowledge, conditions, methods, requirements and techniques required for the analysis and design of foundations and then implementing those finding in real life projects.

1.2.2 Develop a simple yet extensive software

Extensive and rigorous calculations are main part of our project. The soil data that we collect from the site is then analyzed using various methods and formulae to achieve an optimum design for the foundation. For this purpose, our team would make various simple and complex spreadsheets which were later converted into a portable computer software which is user friendly and is able to handle all kinds of problems and cases with speed and ease. The development of this project is not only helpful for us to master our concepts but will be useful throughout our professional careers and can have some industrial applications and uses in the near future.

1.2.3 Learning various software

In the entire working of our project we learned and mastered various software which will be useful in our professional lives:

- Microsoft Word
- Microsoft Excel
- Microsoft Visual Studio
- Adobe Illustrator
- Prezi

1.3 Why Foundation Design and Analysis?

As it is already clear from the above mentioned introduction and objective we choose Design and Analysis of Foundations as our Final Year Project because foundation is the basis of any civil structure. Design of a foundation does not belong to a single subject of civil engineering but it’s a combinations of Geotechnical Engineering, Structural Engineering, Construction Economics, Surveying and Hydraulics Engineering etc. These subject have played a major part on our civil engineering degree so our team felt that foundations would be the ideal topic to sum it all up and achieve an end product of all our learnings.

1.4 Academic Project Outcomes

Other than the main objective of developing our geotechnical knowledge and practical skills the scope of this project goes beyond that thus making it a very dynamic project. We have merged various fields of engineering in our one single project ranging from a geotechnical engineering to a software developer.

The following are the fundamental academic outcomes of our project which encompasses these attributes:

- Understanding
- Accuracy
- Coherence
- Ease

All these attributes play a vital role when fresh engineers step into their professional carriers. These elements will become the stepping stones in a geotechnical design of a foundation, by that concluding this thesis.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Foundations can be classified as shallow and deep foundations depending upon the depth of the soil which is affected by the foundation leading and consequently affect the foundation behaviors. These two types can be further subdivided into different types of foundations which are normally seen in the field.

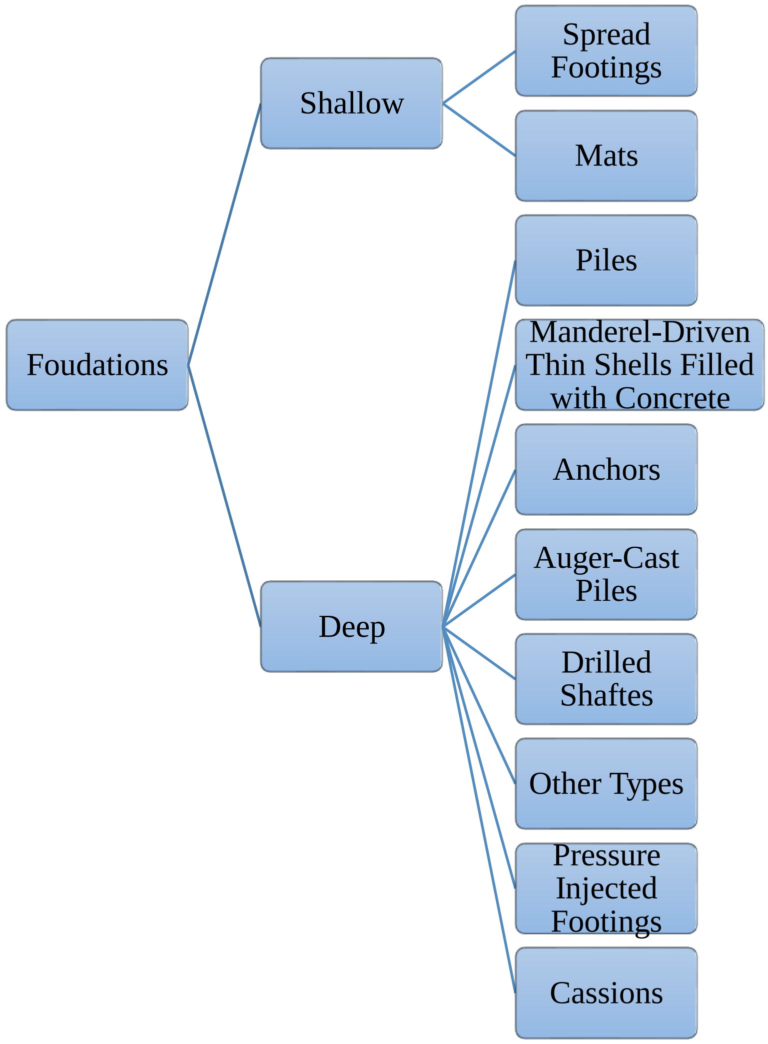


Figure 1 Types of foundations

2.2 Shallow Foundations:

2.2.1 Introduction

Shallow foundations are those which transfer load to the near surface soils. The depth of shallow foundation is less or equal to width of foundation. Depending on the load imposed there are multiple types of a shallow foundation

- Square footing
- Strip footing
- Rectangular footing
- Circular footing
- Mat foundation
- Combined footing

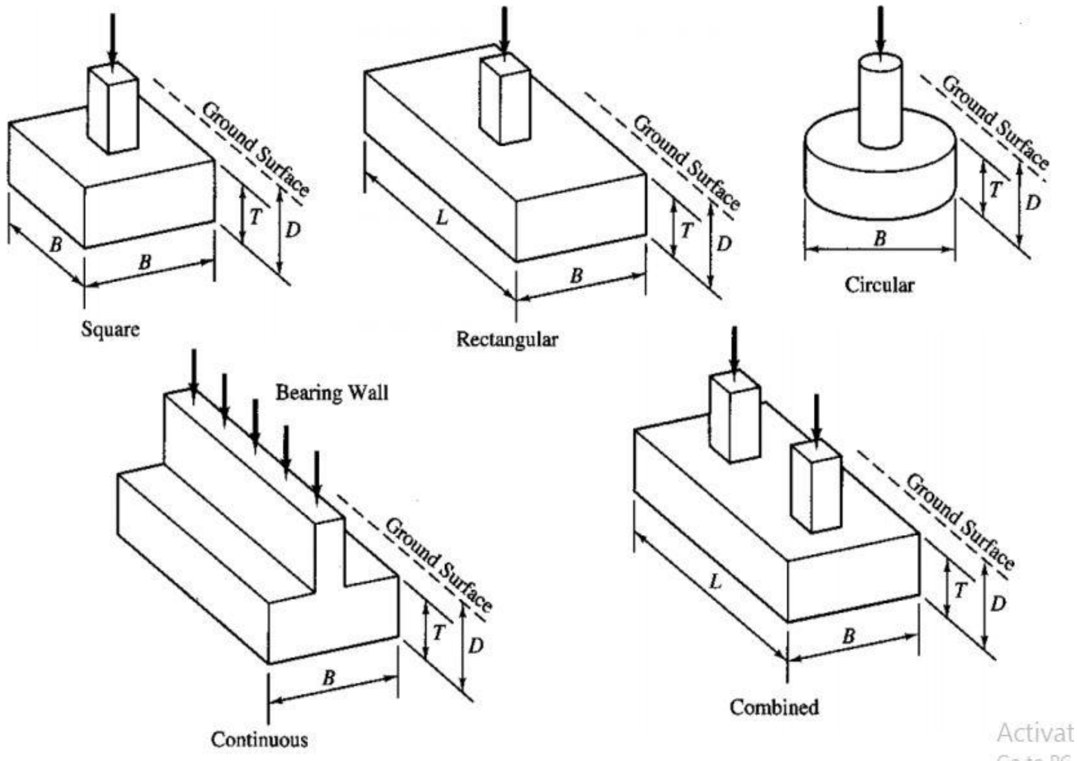


Figure 2 Types of shallow foundations

To perform satisfactorily, shallow foundations must have two main characteristics:

- The foundation must be stable against shear failure of supporting soil.
- The foundation must not settle beyond a tolerable limit to avoid damage to structure.

Other major factors include the depth and location of foundation.

Factors effecting the choice of foundation are:

- Function of structure
- Load the structure has to carry
- The subsurface condition of soil
- The cost of the structure

2.2.2 Steps for foundation selection

- Calculate the loads acting on the footing
- Obtain soil profiles along with pertinent field and laboratory measurement and testing results
- Determine the depth and location of the footing
- Evaluate the bearing capacity of the supporting soil
- Determine the size of the footing
- Compute the footing's contact pressure and check its stability against sliding and overturning
- Estimate the total and differential settlements Design the footing structure

Now calculate cost of each type of footing, which are suitable for such conditions and choose the type which provide perfect balance between cost and performance.

2.2.3 Depth and location of foundation

Foundations must be located properly. The depth and location of foundations are dependent on the following factors:

- Frost action.
- Significant soil volume change.
- Adjacent structures and property lines.

- Groundwater.
- Underground defects.
- Building codes

2.2.4 Types of failure:

2.2.4.1 General Shear Failure (Dense Sand)

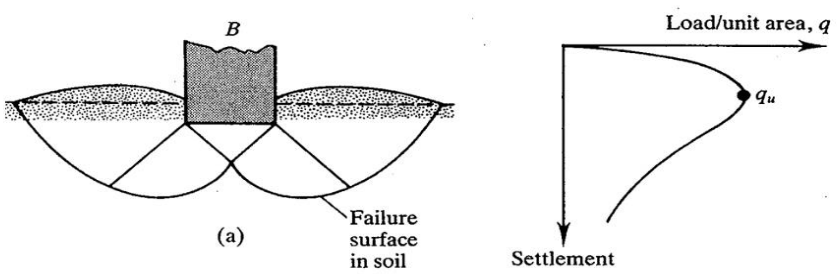


Figure 3 General shear failure

- When relative density >67%
- When settlement reaches 7% of foundation width.
- Considerable bulging

2.2.4.2 Local Shear Failure

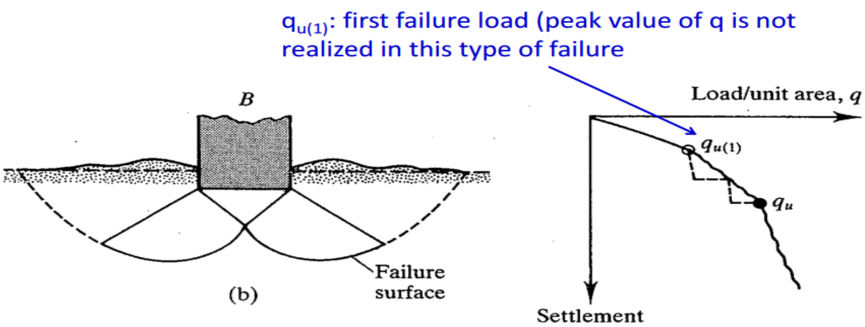


Figure 4 Local shear failure

- When relative density is between 30%-67%
- When settlement exceeds 8% of foundation width

2.2.4.3 Punching Shear Failure (relatively loose soil)

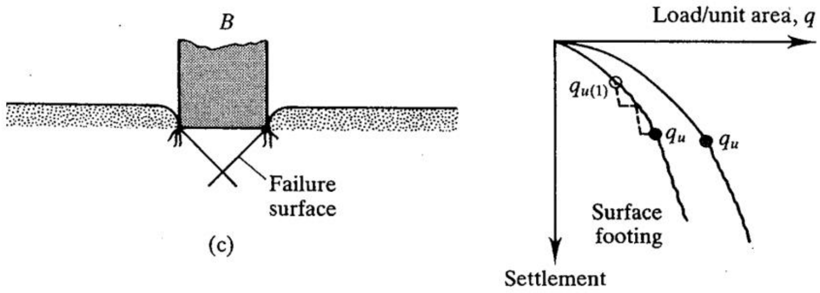


Figure 5 Punching shear failure

- When relative density is <30%, no bulging on sand surface.
- When settlement reaches 6-8% of foundation width

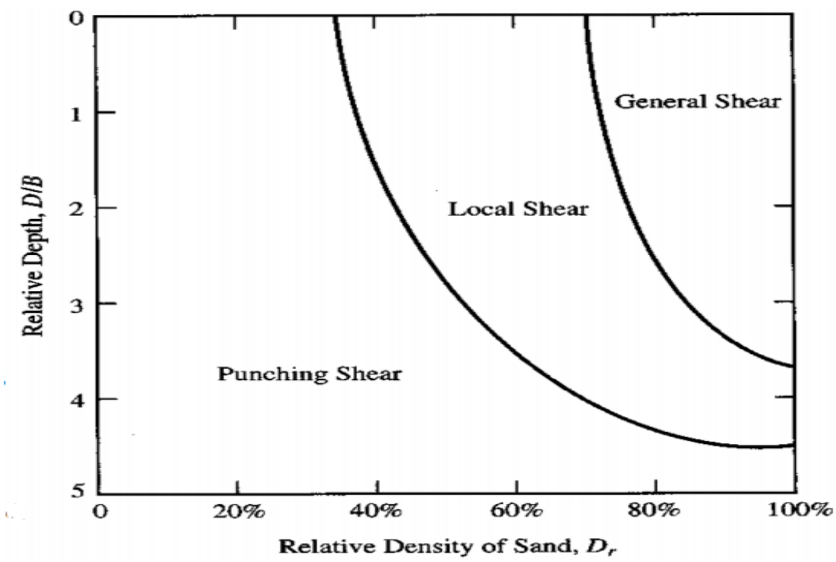


Figure 6 Failure for footings in sand

2.2.5 Bearing Capacity Theories Shear Criteria

Bearing capacity refers to the ability of a soil to support or hold up a foundation and structure. The ultimate bearing capacity of a soil refers to the loading per unit area that will just cause shear failure in the soil. It is given the symbol **qult**. The allowable bearing capacity (symbol **qa**) refers to the loading per unit area that the soil is able to support without unsafe movement. We will discuss all utilized theories in this portion.

2.2.5.1 Terzaghi's Bearing Capacity Theory

Terzaghi (1943) was the first to present a comprehensive theory for the evaluation of the ultimate bearing capacity of rough shallow foundations. In the derivation of the equation, Terzaghi made the following assumptions:

- The soil is homogeneous, isotropic and Columb's law of shear strength is valid.
- The footing is continuous and has a rough base.
- Failure zone does not extend above the base of the foundation.
- Shear resistance of the soil above the base of the foundation is neglected.
- The soil above the base of the foundation is replaced by a uniform surcharge.
- Principal of superposition holds good.

Terzaghi developed bearing capacity equations for different types of footings. For general shear failure

Continuous footings (width B): $\boldsymbol{q_{ult} = c' N_c + \gamma D_f N_q + 0.5 \gamma B N_\gamma}$

Circular footings (radius B): $\boldsymbol{q_{ult} = 1.2 c' N_c + \gamma D_f N_q + 0.3 \gamma B N_\gamma}$

Square footings (width B): $\boldsymbol{q_{ult} = 1.2 c' N_c + \gamma D_f N_q + 0.4 \gamma B N_\gamma}$

For local shear failure, the basic change is c=0.667 c, so revised equations are:

Continuous footings (width B): $\boldsymbol{q_{ult} = 0.667 c' N_c + \gamma D_f N_q + 0.5 \gamma B N_\gamma}$

Circular footings (radius B): $\boldsymbol{q_{ult} = 0.867 c' N_c + \gamma D_f N_q + 0.3 \gamma B N_\gamma}$

Square footings (width B): $\boldsymbol{q_{ult} = 0.867 c' N_c + \gamma D_f N_q + 0.4 \gamma B N_\gamma}$

Where

C' = cohesion of soil

ɣ = unit weight of soil

$\boldsymbol{N_c}$, $\boldsymbol{N_q}$, $\boldsymbol{N_\gamma}$ = Bearing capacity factors

The equation of all bearing capacity factors used in above equations are given as following:

$$N_q = \frac{a_\theta^2}{2cos^2(45 - \frac{\emptyset'}{2})}$$

$$a_\theta^2 = e^{\pi\left(0.75 - \frac{\emptyset'}{360}\right)tan\emptyset\prime}$$

$$N_c = (N_q - 1)/tan\emptyset'$$

$$N_\gamma \cong \frac{2\left(N_q + 1\right)tan\emptyset'}{1 + 0.4\sin(4\emptyset')}$$

For $\emptyset' = 0$, cohesive soil the values of these bearing capacity factors are $N_c = 5.7$, $N_q = 1.0$, $N_\gamma = 0$

For Purely cohesion less soil, c=0 the value of $N_c = 0$. These modifications are applied to equation described before to use them in Excel for calculation of these bearing capacity parameters.

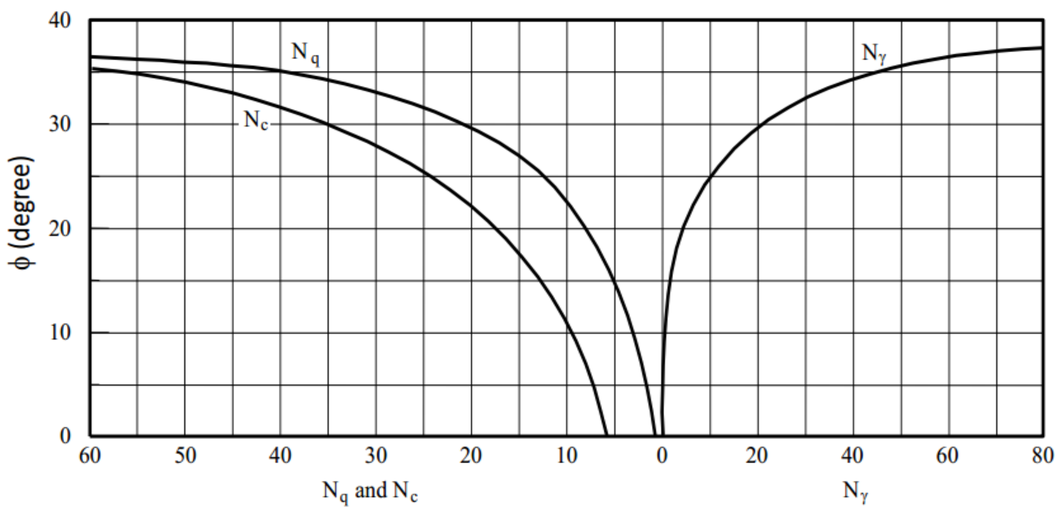


Figure 7 Graph for N factor

2.2.5.2 Effect of water table on bearing capacity

Based on location of water table below ground surface there may be three different cases when water table have effect on bearing capacity.

Case-1: If depth of water is between ground surface and depth of footing, $0 \leq D_w \leq D_f$, then q in bearing capacity equation is calculated as:

$$q = effective\ surcharge = D_1\gamma + D_2(\gamma_{sat} - \gamma_w)$$

Case-2: If depth of water is such that it is below footing but should not more than width of footing, $D_f \leq D_w \leq (D_f + B)$. Then,

$$\bar{\gamma} = \frac{1}{B}\{\gamma d + \gamma'(B - d)\}$$

Case-3: If depth of water is below the footing such that $D_w \geq D_f + B$, then water will have no effect on ultimate bearing capacity.

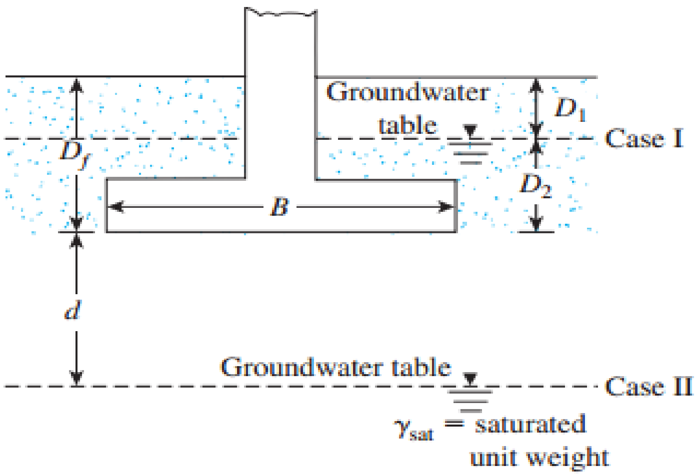


Figure 8 Effect of water table on bearing capacity

2.2.5.3 General bearing capacity equation

Terzaghi developed the general bearing capacity equation by using the equations developed by Prandtl.

Meyerhof modified Terzaghi's bearing capacity theory for strip footings to incorporate shape, inclination, and depth.

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + 0.5\gamma BN_\gamma s_\gamma d_\gamma i_\gamma$$

Later on Hanson modified the Meyerhof's work by introducing the two more factors accounting for base tilt and foundation on slopes.

Vesic used following equation for the computation of bearing capacity factors:

$$N_q = e^{\pi tan\emptyset'} tan^2\left(45 + \frac{\emptyset'}{2}\right)$$

$$N_c = \frac{N_q - 1}{tan\emptyset'}$$

For $\emptyset' = 0$ $\qquad\qquad N_c = 5.14$

Although these expression for N_c and N_q are the same but all of them use different equations for the calculation of N_γ.

Meyerhof $\qquad N_\gamma = (N_q - 1)\tan(1.4\emptyset')$

Hanson $\qquad N_\gamma = 1.5(N_q - 1)tan\emptyset'$

Vesic $\qquad N_\gamma = 2(N_q + 1)tan\emptyset'$

Out of these equation stated above Terzaghi's bearing capacity equations are most popular, Hanson and Meyerhof are used widely, while Vesic is not used extensively.

2.2.5.4 Skempton's method (For Clay):

Skempton gave his method in 1951 to calculate bearing capacity for cohesive soil ($\emptyset = 0$).

When encountering the clays, one may use the following equation for bearing capacity calculation based on shear

$$q_{d(net)} = 5S_u(1 + \frac{0.2D_f}{B})(1 + \frac{0.2B}{L})$$

Here, S_u= undrained shear strength

L= length of footing

The value of undrained shear strength (S_u) can be calculated from spt-N value by using following relations.

Terzaghi and peck (1967) $S_u(KPa) = 6.25N$

Hara et al (1974) $S_u(KPa) = 29N^{0.72}$

Sowers (1979) $S_u(KPa) = 7.5N$

Sivrikaya & Togrol (2002) $S_u(KPa) = 3.35N$

Farzad & Behzad (2011) $S_u(KPa) = 1.5N - 0.1w_n - 0.9LL + 2.4PI + 21.1$

After calculating S_u from these method we take average to use the value of undrained shear strength for bearing capacity calculation by Skempton's Method.

2.2.6 Bearing Capacity Theories Settlement Criteria

The bearing capacity of footing on clay is not affected by the size of footings, it remains constant. However, the settlement is increases with an increase in size of the footing. For footing design, it is essential to consider both settlement criteria and shear (bearing capacity) criteria to decide safe bearing pressure.

When footing is designed on stiff clay, hard clay and other firm soil, it does not need settlement analysis if design provides a minimum factor of safety of 3 on the net ultimate bearing capacity of the soil. But for soft clay, compressible silt and weak soils settlement analysis is necessary, even under moderate pressure.

2.2.6.1 Effects of settlement

A structure may settle in two different days; it may settle uniformly or differential settlement may occur. If a structure settles uniformly, there will be no detrimental effects. The most that can happen is that the structure's utilities will be disrupted. This can only be, if there is uniform load and the soil is homogenous. Although, it is to be noted outside certain permissible limits, even uniform settlement can be devastating.

Differential settlement between parts of structure may not exceed 75% of the absolute settlement. Settlement calculations should be done with great caution and care, keeping in view the cost of project.

Generally, there are two major components of foundation settlement one is elastic settlement and other is consolidation settlement. Consolidation settlement consist of two parts one is primary consolidation settlement and other is secondary consolidation settlement. Elastic settlement is mainly for foundation on granular soils. There is different method to calculated elastic settlement for the foundation on granular soil.

2.2.6.2 Terzaghi and Peck's method

Terzaghi and Peck proposed a relation for the calculation of elastic settlement based on observed settlement in 1948. This relation is between allowable bearing capacity, standard penetration test (SPT) N value and width of footing.

$$S_e = C_w C_D \frac{3q}{N_{60}} \left(\frac{B}{B+0.3}\right)$$

Where,

C_W= ground water table correction

C_D = depth of embedment correction = $1 - (\frac{D_f}{4B})$

D_f = depth of embedment (footing)

If the depth of water table is equal to or greater than 2B below the foundation, the magnitude of C_w is equal to 1, and if the depth of water table is less than or equal to B below the foundation it is equal to 2.

2.2.6.3 Meyerhof's method

Meyerhof proposed relationships for the elastic settlement based on observed settlement in 1956, for foundations on granular soil. But later on he had applied correction for water table location and depth of footing.

For B≤1.22m,

$$S_e = C_w C_D \frac{1.25q}{N_{60}}$$

For B>1.22m,

$$S_e = C_w C_D \frac{2q}{N_{60}} \left(\frac{B}{B + 0.3}\right)^2$$

Where

$C_w = \text{water dept correction} = 1.0$

$C_D = \text{depth of footing correction} = 1 - (\frac{D_f}{4B})$

2.2.6.4 Peck and Bazaraa's method

In 1969, Peck and Bazaraa found that relation provided by Terzaghi and Peck was overly conservative and they give a relation for the elastic settlement.

$$S_e = C_w C_D \frac{2q}{(N_1)_{60}} \left(\frac{B}{B + 0.3}\right)^2$$

Where

$S_e = \text{settlement in mm}$

$C_D = 1 - 0.4(\frac{\gamma D_f}{q})^{0.5}$

$$C_w = \frac{\text{σo at 0.5B below the foundation}}{\text{σo}' \text{ at 0.5B below bottom of foundation}}$$

$\sigma o = \text{total overburden pressure}$

$\sigma o'$ = effective overburden pressure

For $\sigma o' \leq 75KN/m^2$, $(N_1)_{60} = \frac{4N_{60}}{1+0.04\sigma o'}$

For $\sigma o' > 75KN/m^2$, $(N_1)_{60} = \frac{4N_{60}}{3.25+0.01\sigma o'}$

2.2.6.5 Schmertmann's method

$$\delta = C_1 C_2 C_3 q' \Sigma(\frac{I_E \Delta z}{E_s})$$

Where,

C_1 = Depth factor

C_2 = Secondary creep factor

C_3 = Shape factor

q' = Net bearing pressure

I_E = Strain influence factor at the midpoint of soil layer

Δz = Thickness of soil layer

E_s = Equivalent modulus of elasticity in soil layer

$C_1 = 1 - 0.5[\frac{\sigma_{zD}}{q'}]$

C_2 = 1+0.2log[t/0.1]

C_3 = 1.03-1.03 L/B≥ 0.73

σ_{zD} = Effective stress at depth D

t = Time since application of load (years)

L= Foundation length

B= Foundation width

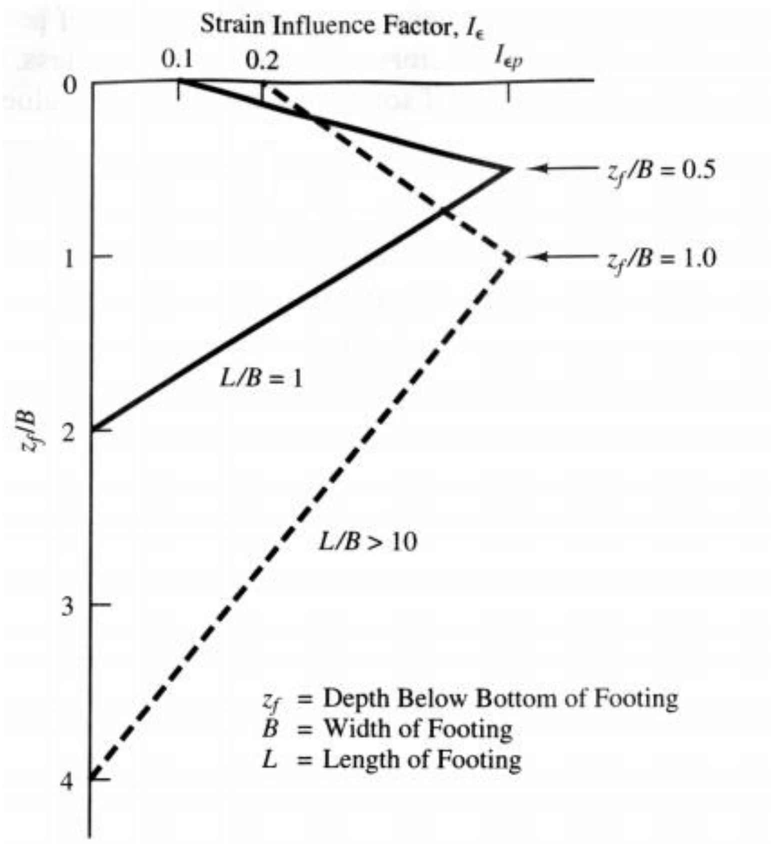


Figure 9 Graph for Schmertmann's Method for Settlement (in sand)

The equivalent modulus of elasticity can be ascertained using the number of blows obtained from SPT.

$$E_s = 766N_{60} \text{ in KPa}$$

$$E_s = 766N_{60} \text{ in Ksf}$$

$$E_s = \beta_o\sqrt{OCR} + \beta_1 N_{60}$$

Where the values of β_o and β_1 can be derived from the table

Soil Type	β_0 (lb/ft²)	β_0 (kPa)	β_1 (lb/ft²)	β_1 (kPa)
Clean sands (SW and SP)	100,000	5,000	24,000	1,200
Silty sands and clayey sands (SM and SC)	50,000	2,500	12,000	600

Figure 10 Table for Values of βo and β1

The methodology that governs the settlement calculation is as follows:

- Perform appropriate testing
- Divide the zone of influence into layers, with thickness of each layer depending upon the variation of E with depth profile
- Compute the peak strain influence factor
- Compute the peak strain influence factor at the mid-point of each layer
- Compute the correction factors
- Finally determine the settlement

2.2.6.6 Burland and Burbidge method

Burland and Burbidge proposed an empirical relation for the settlement calculation of foundation on granular soil in 1985, which uses the SPT-N value to calculate settlement of foundation.

For L/B=1,

$$S_e = B^{0.75}\frac{1.7}{\bar{N}^{1.4}}(q - \frac{2\sigma_o'}{3})$$

For L/B>1,

$$S_e = B^{0.75}\frac{1.7}{\bar{N}^{1.4}}(q - \frac{2\sigma_o'}{3})(\frac{\frac{1.25L}{B}}{\frac{L}{B}+0.25})^2$$

2.2.6.7 Bearing capacity based on SPT (for 25mm settlement)

There is different method available to calculate bearing capacity from SPT-N value. These methods are based on maximum allowable settlement 25mm.

2.2.6.7.1 Modified Meyerhof method

Meyerhof (1965) suggested the following procedure to obtain allowable bearing pressure to give a settlement of 25 mm. The equation proposed by Meyerhof was found to be very conservative and Bowles (1982) modified this equation. The equation proposed is:

For B $\leq 1.2m$ $\quad q_s = 20N_{cor} * F_d * R_w$

For B$> 1.2m$ $\quad q_s = 20N_{cor} * F_d * R_w(\frac{B+0.3}{B})^2$

2.2.6.7.2 Modified Teng's method

Teng (1962) based on the work of Terzaghi and Peck gave a relationship for allowable bearing capacity for a given permissible settlement. The equation proposed by Teng was found to be very conservative and Bowles (1982) modified this equation. The equation proposed is:

$$q_s = 53(N_{cor} - 3) * F_d * R_w(\frac{B+0.3}{2B})^2$$

If the tolerable settlement is greater than about 25mm, the safe bearing pressure calculated by the above equation can be projected as:

$$q_s' = \frac{S}{25}q_s$$

2.2.6.8 Plate Load Test

It is a semi-direct method to estimate the allowable bearing pressure of soil to induce a given amount of settlement. These plates vary in size and thickness.

2.2.6.8.1 Procedure

From the test results load settlement curve should be plotted. The allowable pressure on a prototype foundation for an assumed settlement may be found by making use of following equations suggested by Terzaghi and Peck (1948) for square footing on granular soil.

2.2.6.8.2 Calculation Steps

- S_r and b_p are known
- S_p and B are unknowns. The value of B is assumed in order to calculate the value of S_p from the equation.
- The value of bearing pressure corresponding to the computed value of S_p is found from the settlement curve.

$$S_f = S_p [\frac{B(b_p + 0.3)}{b_p(B + 0.3)}]^2$$

$$S_f = S_p x \frac{B}{b_p}$$

Due to the short duration it can be used to determine the consolidation settlement, only applicable for immediate settlement. In sandy soil the immediate settlement is equal to the total settlement, whereas in clayey soil it is only a fraction. Therefore, plate load test is not effective for clayey soils.

2.3 Deep Foundations

2.3.1 Introduction

Pile foundations are used to transfer loads of the structure to underlying soil strata. They go deep into the soil unlike shallow foundations. Shallow foundation is always cheaper than deep foundations and also easier to build and take less time for construction but under some conditions where shallow foundations can’t provide structural safety it is necessary to construct deep foundations. Some of the situations under which it is necessary to go for deep foundations are:

- Upper soil is weak and can’t provide enough support to the loads of structure
- Presence of lateral forces.
- Presence of expansive or collapsible soils on the site.
- To resist the uplifting force.
- Soil erosion at the ground surface.
- Large values of concentrated loads.

Pile distribute load of superstructure to the ground in one of the following ways

- Skin friction.
- End bearing.
- Combination of both skin friction and end bearing.

Different material which can be used for deep foundations are

- Steel.
- Timber.
- Concrete.

Piles are mostly used in the form of group with pile cap on the top of individual piles, connecting them together to form a pile group and act as single unit to resist the loads.

2.3.2 Pile Load Transfer

To understand that how pile transfers the load to underlying soil, consider a pile is loaded with load Q on its top. Some of this load will be taken pile surface along the length of the pile and the remaining by end resistance. As the load is increased, most of the side frictional portion along the pile length will be developed when the pile moves 5 to 10 mm, and doesn't depend on pile size and its length. On the other hand, the maximum tip resistance will not be developed unless the pile has moved about 10 to 25% of the pile dia. It indicates that as compare to the point resistance, side friction along the pile can be developed at a much smaller pile displacement.

2.3.3 Estimation of Pile Capacity

To find ultimate pile capacity following equation can be used

$$Q_u = Q_p + Q_s$$

Where,

$Q_p = A_p q_p$ = load resistance at pile point

$Q_s = \sum p \Delta L f$ = skin resistance from the soil–pile interface

Different methods are used to estimate values of Q_p and Q_s as described below.

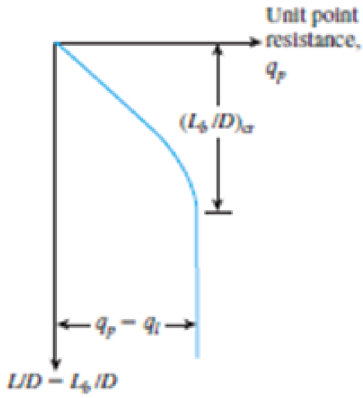


Figure 11 Variation of unit point resistance in sand

2.3.4 Meyerhof's Method to Estimate value of Qp for Sand

The bearing capacity at pile point q_p, in sand increases with the depth of pile and reaches a maximum value at $L_b/D = (L_b/D)_{cr}$.

For homogeneous soil L_b is equal to the total length of the pile. Beyond the value $(L_b/D)_{cr}$, the value of q_p doesn't change.

For piles in sand,

$$Q_p = A_p q_p = A_p q' N_q^*$$

However, value of Q_p should not be greater than $A_p q_t$.

Where,

$$q_t = 0.5\, p_a N_q^* \tan \phi'$$

Value of N_q^* can be taken from the figure below,

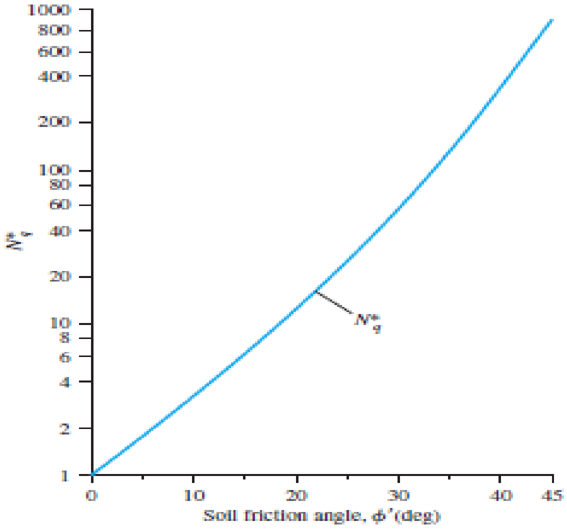


Figure 12 Variation of Nq^* with soil friction angle ϕ'

Clay (ϕ = 0)

For piles in clays the net ultimate load can be calculated as

$$Q_p = N_c^* c_u A_p = 9 c_u A_p$$

Where,

c_u= soil cohesion below pile tip.

2.3.5 Vesic's Method to Estimate value of Q_p

For Sand

According to Vesic (1977) following equation can be used to estimate the point bearing capacity,

$$Q_p = A_p q_p = A_p \sigma_0' N_\sigma^*$$

Where,

σ_0' = effective vertical stress. Where,

$$\sigma_0' = \left[\frac{(1 + 2K_0)}{3}\right] q'$$

K_0 = coefficient of earth pressure, which can be calculated as, 1 - sin ϕ'

$N_\sigma{}^*$ = bearing capacity factor

$$N_\sigma{}^* = f\,(I_{rr})$$

I_{rr} = reduced rigidity index

$$I_{rr} = \frac{I_r}{(1 + I_r\Delta)}$$

Where,

$$I_r = \frac{E_s}{2(1 + \mu_s)q' \tan\phi'} = \frac{G_s}{q' \tan\phi'}$$

G_s = soil's shear modulus

Δ = volumetric strain below the pile point

And,

$$\Delta = 0.005\left[1 - \frac{\phi' - 25}{20}\right]\left(\frac{q'}{p_a}\right)$$

I_{rr}	N_c^*
10	6.97
20	7.90
40	8.82
60	9.36
80	9.75
100	10.04
200	10.97
300	11.51
400	11.89
500	12.19

Figure 13 Variation of $N_c{}^*$ with Irr for ϕ = 0

Clay (ϕ = 0)

For clay, to calculate point bearing capacity of a pile following equation can be used

$$Q_p = N_c{}^* c_u A_p$$

According to Vesic,

$$N_c{}^* = \left(\frac{4}{3}\right)(\ln I_{rr} + 1) + \left(\frac{\pi}{2}\right) + 1$$

For saturated clay with no volume change, $\Delta = 0$.

$$I_{rr} = Ir = \frac{E_s}{3c_u}$$

Figure showing variation of $N_c{}^*$ with I_{rr} for ϕ = 0.

2.3.6 Coyle and Castello’s Method to Estimate value of Qp in Sand

According to Coyle and Castello (1981) performed many pile load test on piles in sand. Depending upon the test results, they proposed that,

$$Q_p = N_q{}^{*} q' A_p$$

Where,

q′ = effective vertical stress value at tip of the pile

Figure below shows the trends of $N_q{}^{*}$ with L/D and ϕ′.

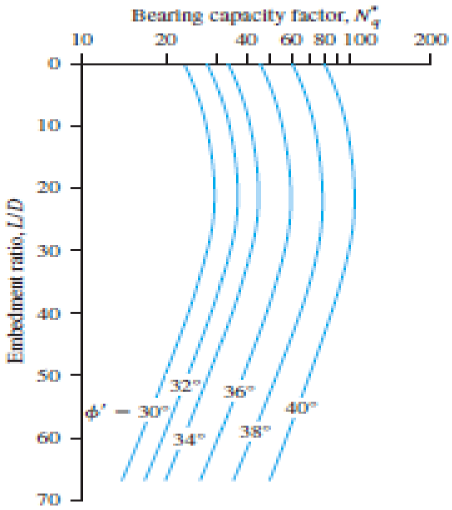


Figure 14 Variation of $N_q{}^{*}$ with L/D and the soil friction angle ϕ'

2.3.7 Calculation of Qp using SPT and CPT values

2.3.7.1 Meyerhof (1976)

To estimate value of ultimate point resistance in a granular soil using standard penetration numbers, Meyerhof suggested following relation

$$q_p = 0.4 p_a N_{60} \left(\frac{L}{D}\right) \leq 4 p_a N_{60}$$

Where,

N_{60} = average N value near tip of the pile

Meyerhof (1956) also suggested that

$$q_p = q_c \quad \text{(in granular soil)}$$

Where,

q_c = cone penetration resistance.

2.3.7.2 Briaud et al. (1985)

According to Briaud et al. (1985) following correlation using N value can be used for q_p calculation in granular soil.

$$q_p = 19.7 p_a {N_{60}}^{0.36}$$

2.3.8 Frictional Resistance (Qs) in Sand

As described above, the frictional resistance

$$Q_s = \sum p \Delta L f$$

Where,

$$f = K(\sigma_0') \tan(\delta')$$

And,

K = earth pressure coefficient

σ_0' = effective overburden pressure

δ' = friction angle of soil

The values of δ' ranges from $0.5\phi'$ to $0.8\phi'$.

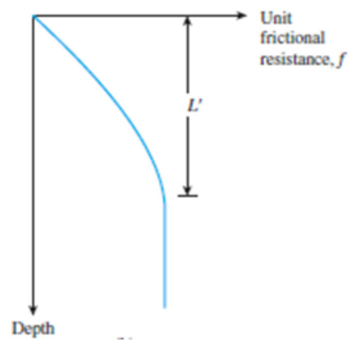


Figure 15 Unit frictional resistance for piles in sand

In the case of sand one thing should be kept in mind that the unit skin friction value increases up to certain value of depth and then its value become constant. Its value ranges from 15 to 20 pile diameters. To use conservative value 15D is to be used.

$$L' = 15D$$

The value of *K* changes with depth; at the top it is equal to the Rankine passive earth pressure coefficient, K_p of the pile and at a greater depth its value is equal to at-rest pressure coefficientK_0. For use following values are recommended.

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

Figure 16 Average values of effective earth pressure coefficient K

2.3.9 Q_s Calculation using Standard Penetration Test Results

Meyerhof (1976) suggested different equations for calculation of unit frictional resistance f_{av}, for high displacement driven piles following equation can be used

$$f_{av} = 0.02p_a N_{60}$$

Where,

N_{60} = average N value

For driven piles causing low displacement

$$f_{av} = 0.01p_a N_{60}$$

Briaud et al. (1985) suggested that,

$$f_{av} = 0.224p_a (N_{60})^{0.29}$$

Thus,

$$Q_s = \sum p\Delta Lf$$

2.3.10 Frictional Resistance in Clay

Many methods are available in literature for calculation of unit frictional resistance. Most commonly used methods are,

2.3.10.1 λ-Method

According to this method average unit skin resistance is

$$f_{av} = \lambda\,(\sigma_0' + 2c_u)$$

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

Figure 17 Variation of λ with pile length, L

Where,

σ_0' = effective overburden pressure

c_u = undrained cohesion of soil

Value of λ can be taken from the figure and changes with the depth of penetration of the pile. Thus,

$Q_s = p\Delta L(f_{av})$

2.3.10.2 α-Method

According to this method, value of f in soils can be calculated by the equation

$$f = \alpha\,(c_u)$$

Where,

α= adhesion factor

The variation of the value of α is shown in Figure

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

Note: p_a = atmospheric pressure ≈ 100 kN/m^2

Figure 18 Variation of α based on Terzaghi, Peck and Mesri, 1996)

Knowing value of *f* determined, the total frictional resistance may be calculated as

$$Q_s = p\Delta L(f)$$

2.3.10.3 β-Method

According to this method, value of f in soils can be calculated by the equation

$$f = \beta\,\sigma_0'$$

Where,

σ_0' = effective overburden stress

$\beta = K \tan \phi_R'$

ϕ_R' = remolded clay drained friction angle

K = coefficient of earth pressure

The value of *K* is defined as

$$K = 1 - \sin \phi_R' \quad \text{(for normally consolidated clays)}$$
$$K = 1 - \sin \phi_R' \, (OCR)^{0.5} \quad \text{(for over consolidated clays)}$$

Knowing value of *f*, the total frictional resistance may be calculated as

$$Q_s = p\Delta Lf$$

2.3.11 Q_s Calculation using Cone Penetration Test Results

According to Nottingham and Schmertmann (1975) and Schmertmann (1978) following correlation can be used to calculate unit skin friction in clay.

$$f = \alpha' f_c$$

Where,

f_c = frictional resistance

The value of α' varying is taken from the Figure below

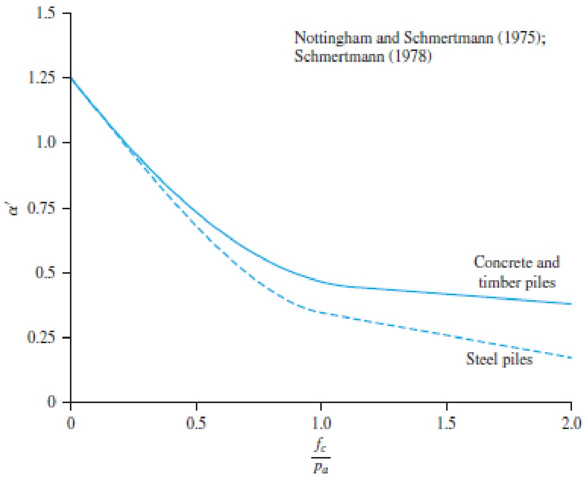


Figure 19 Variation of α′ for the piles in clay

Knowing value of *f* , the total frictional resistance may be calculated as

$$Q_s = p\Delta Lf$$

2.3.12 End Bearing Capacity of Piles Resting on Rock

Bearing capacity of the rock needed to be calculated when piles are driven into the soil layers and reaches rock present under the soil. To calculate ultimate unit point resistance for this condition Goodman, 1980 method can be used, according to which

$$q_p = q_u\,(N_\phi + 1)$$

Where,

$N_\phi = \tan^2(45 + \phi'/2)$

q_u = compression strength of rock

ϕ' = friction angle of rock

And,

$$Qp = \frac{q_p A_p}{FS}$$

2.3.13 Pile Load Tests

For large projects to cater for the unreliability of prediction methods a specific number of load tests must be conducted on piles. The vertical and lateral load bearing capacity of a pile can be tested in the field using these tests. Figure below shows a schematic diagram of the pile load arrangement for testing axial compression in the field. Step loads are applied to the pile, and sufficient time is allowed to elapse after each load so that a small amount of settlement occurs. The amount of load to be applied for each step will vary, depending on local building codes. Most building codes require that each step load be about one-fourth of the proposed working load. The load test should be carried out to at least a total load of two times the proposed working load. After the desired pile load is reached, the pile is gradually unloaded.

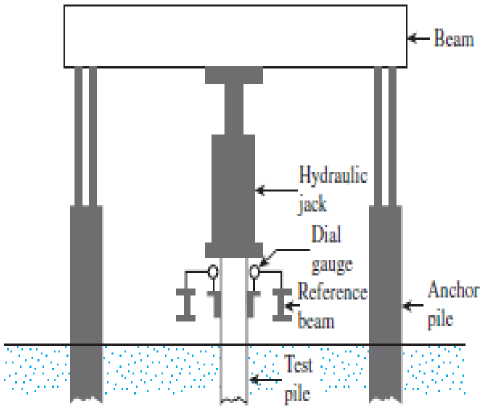


Figure 20 Schematic diagram of pile load

Figure below shows a load–settlement diagram obtained from field loading and unloading. For any load *Q*, the net pile settlement can be calculated as follows:

When $Q=Q_1$

$$s_{net(1)} = s_{t(1)} - s_{e(1)}$$

When $Q=Q_2$

$$s_{net(2)} = s_{t(2)} - s_{e(2)}$$

Where,

s_{net} = net settlement

s_e = elastic settlement of the pile itself

s_t = total settlement

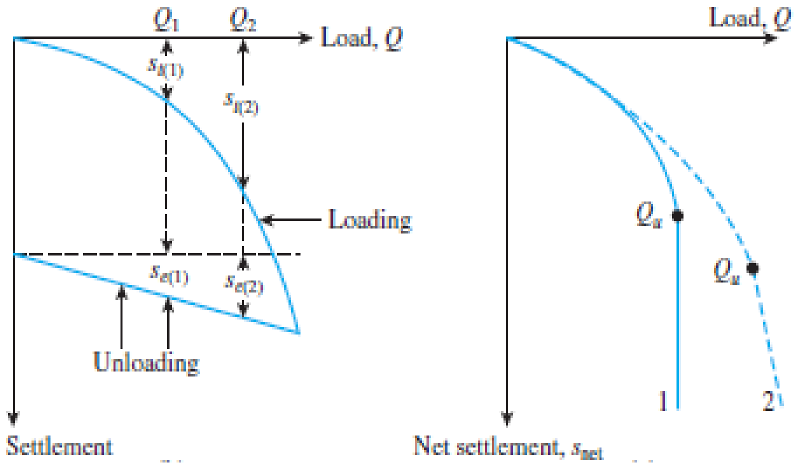


Figure 21 Plot of load against total and net settlement

These values of *Q* can be plotted in a graph against the corresponding net settlement, as shown in Figure above. The ultimate load of the pile can then be determined from this graph.

Pile settlement may increase with load to a certain point, beyond which the load–settlement curve becomes vertical. The load corresponding to the point where the curve of *Q* versus becomes vertical is the ultimate load Q_u, for the pile; it is shown by curve 1 in Figure. In many cases, the latter stage of the load–settlement curve is almost linear, showing a large degree of settlement for a small increment of load; this is shown by curve 2 in the figure. The ultimate load Qu, for such a case is determined from the point of the curve of *Q* versus s_{net} where this steep linear portion starts.

To obtain the ultimate load from the load-settlement plot Davisson's method is used more often in the field. Referring to Figure below, the ultimate load occurs at a settlement level of s_u.

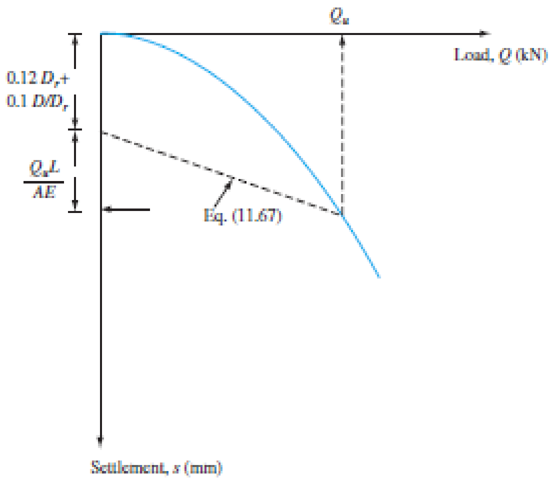


Figure 22 Davisson's method for determination of Qu

$$s_u(\text{mm}) = 0.012D_r + 0.1\left(\frac{D}{D_r}\right) + \frac{Q_u\,L}{A_pE_p}$$

Where,

Q_u = ultimate load in KN

D = pile diameter in mm

D_r = reference pile diameter in mm

L = pile length in mm

A_p = pile cross section area

E_p = Young's modulus of pile material (kN/mm^2)

2.3.14 Elastic Settlement of Piles

To ensure structural safety in addition to the bearing capacity criteria, settlement criteria should also be satisfied according to which the total settlement of structure should not be greater than the total allowable settlement value. To calculate total settlement of a pile following equation can be used

$$s_e = s_{e1} + s_{e2} + s_{e3}$$

Where,

s_{e1} = elastic settlement of pile

s_{e2}= settlement due to tip load

s_{e3}= settlement due to side friction

Formulas to calculate all three settlements are given below,

$$s_{e1} = \frac{\left(Q_{wp} + \varepsilon Q_{ws}\right)L}{A_p E_p}$$

Where,

Q_{wp} = working load at the pile point

Q_{ws} = working load at the pile surface

A_p = area of cross section of pile

L = pile length

E_p = modulus of elasticity of pile

$$s_{e2} = \left(\frac{q_{wp} D}{E_s}\right)(1 - \mu s^2) I_{wp}$$

Where,

D = pile diameter

E_s= modulus of elasticity of soil

μs = Poisson's ratio of soil

I_{wp} = influence factor

$$s_{e3} = \left(\frac{Q_{ws}}{pL}\right)\left(\frac{D}{E_s}\right)(1 - \mu s^2) I_{ws}$$

Where,

P = pile perimeter

I_{ws} = influence factor = $2 + 0.35 \left(\frac{L}{D}\right)^{0.5}$

2.3.15 Group Efficiency

Piles are mostly used in groups. When the piles are placed close to each other the stresses transmitted by the piles to the soil will overlap thus reducing the load-bearing capacity of the piles. Ideally, the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles. In practice, the minimum center to center pile spacing d, is 2.5D.

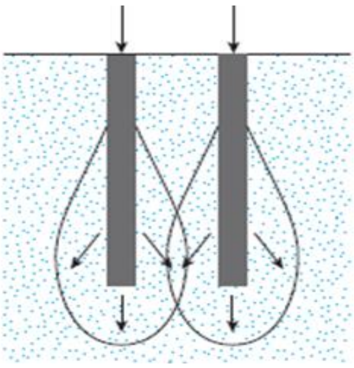


Figure 23 Stress transmission overlap of group piles

The efficiency of the load-bearing capacity of a group pile may be defined as

$$\eta = \frac{Q_{g(u)}}{\sum Q_u}$$

Where,

η = group efficiency

Using Converse–Labarre equation

$$\eta = 1 - \left[\frac{(n_1 - 1)n_2 + (n_2 - 1)n_1}{90} \; n_1 n_2\right]\theta$$

Where,

$$\theta(\text{deg}) = \tan^{-1}\left(\frac{D}{d}\right)$$

2.3.16 Elastic Settlement of Group Piles

Relation for the settlement of group piles was given by Vesic (1969) is

$$s_{g(e)} = \left(\frac{B_g}{D}\right)^{\frac{1}{2}} s_e$$

Where,

$s_{g(e)}$ = elastic settlement of group piles

B_g = width of group pile section

D = diameter of each pile in the group

s_e = elastic settlement of each pile at comparable working load

2.3.17 Consolidation Settlement of Group Piles

The consolidation settlement of a group pile in clay can be estimated by using the

2:1 stress distribution method. The calculation involves the following steps

Let the depth of embedment of the piles be *L*. The group is subjected to a total load of Q_g. If the pile cap is below the original ground surface, Q_g equals the total load of the

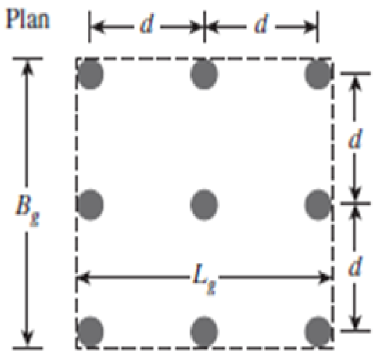


Figure 24 Plan of group piles (3x3)

superstructure on the piles, minus the effective weight of soil above the group piles removed by excavation.

Assume that the load Q_g is transmitted to the soil beginning at a depth of 2L/3 from the top of the pile, as shown in the figure. The load Q_g spreads out along two vertical to one horizontal line from this depth. Lines *aa′* and *bb′* are two 2:1 lines.

Calculate the increase in effective stress caused at the middle of each soil layer by the loadQ_g. The formula is

$$\Delta\sigma_i' = \frac{Q_g}{[(B_g + zi)(L_g + zi)]}$$

Where,

$\Delta\sigma_i'$ = increase in effective stress at the middle of layer *i*

Calculate the consolidation settlement of each layer caused by the increased stress. The formula is

$$\Delta s_{ci}' = \left[\frac{\Delta e_i}{1 + e_{0i}}\right] H_i$$

Where,

$\Delta s_{ci}'$ = consolidation settlement of layer *i*

Δe_i= change of void ratio caused by the increase in stress in layer *i*

e_{0i} = initial void ratio of layer *i*

H_i= thickness of layer *i*

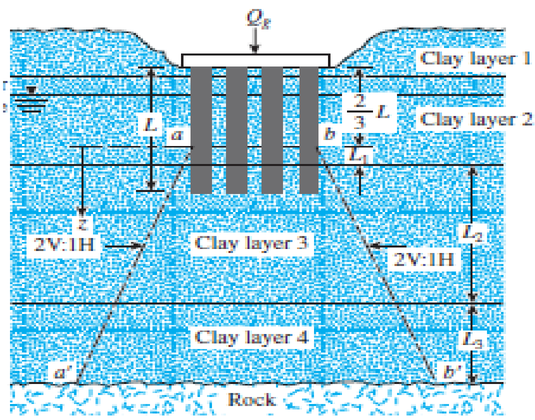


Figure 25 Consolidation settlement of group piles

The total consolidation settlement of the group piles is then

$$\Delta s_{c(g)} = \sum \Delta s_{ci}$$

2.3.18 Drilled-Shaft Foundations

Introduction

Drilled shafts are also named as caisson, pier, drilled pier but they all refer to a cast-in-place pile generally having a diameter of about 750 mm or more.

For construction of drilled shaft, a hole is drilled or excavated to the bottom of a structure's foundation and then filled with concrete. Depending on the soil conditions, casings may be used to prevent the soil around the hole from caving in during construction.

Drilled shafts can be without an enlarged bottom.

The use of drilled-shaft foundations has several advantages over driven piles:

- Constructing drilled shafts in dense sand and gravel is easier than driving piles.
- When piles are driven by a hammer, the ground vibration may cause damage to nearby structures. The use of drilled shafts avoids this problem.
- Piles driven into clay soils may produce ground heaving and cause previously driven piles to move laterally. This does not occur during the construction of drilled shafts.
- There is no hammer noise during the construction.
- Because the base of a drilled shaft can be enlarged, it provides great resistance to the uplifting load.
- The surface over which the base of the drilled shaft is constructed can be visually inspected.
- More economical than methods of constructing driven piles.
- Drilled shafts have high resistance to lateral loads.

There are also some of drawbacks to the use of drilled-shaft construction.

- Concreting operation may be delayed by bad weather.
- Deep excavations for drilled shafts may induce substantial ground loss and damage to nearby structures.

2.3.19 Load-Bearing Capacity in Granular Soil

Reese and O'Neill (1989) proposed a method for calculating the load-bearing capacity of drilled shafts that is based on settlement. According to which

$$Q_{u(net)} = \sum(f_i p \Delta L_i) + A_p q_p$$

Where,

f_i = ultimate unit shearing resistance in layer i

q_p = unit point resistance

A_p = area of the base

And.

$$f_i = \beta_1 \sigma_{zi}' < 192 \frac{kN}{m^2}$$

Where,

$$\beta_1 = 1.5 - 0.224(z_i)^{0.5} \quad (0.25 \leq \beta_1 \leq 1.2)$$

The point bearing capacity is

$$q_p = 57.5 N_{60} \leq 4310 \frac{kN}{m^2} \quad (\text{for } D_b < 1.27 \text{ m})$$

Where,

N_{60} = field standard penetration number within a distance of $2D_b$ below the base of shaft.

If D_b is equal to or greater than 1.27m q_p may be replaced by q_{pr},

$$q_{pr} = \frac{1.27}{D_b} q_p$$

Based on the desired level of settlement, Figures below may now be used to calculate the allowable load $Q_{all(net)}$. To do so we need to follow the following steps

- Select a value of settlement, s.
- Calculate $\sum(f_i p \Delta L_i)$ and $A_p q_p$.
- Using Figures and the calculated values in above Step, determine the side load and the end bearing load.

- The sum of the side load and the end bearing load gives the total allowable load.

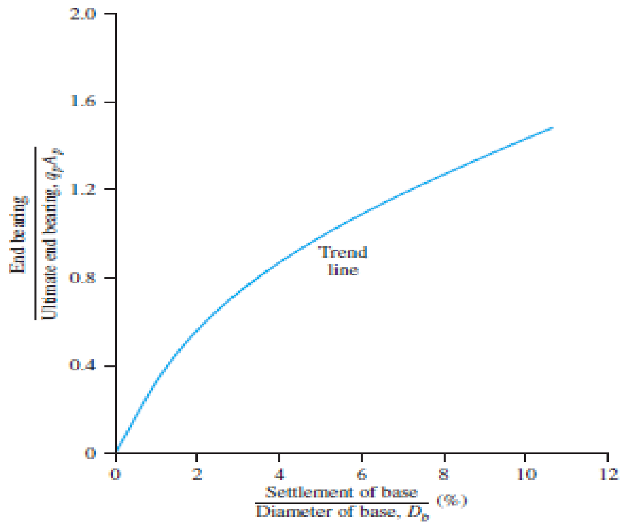


Figure 26 Base-load transfer versus settlement in sand

For calculation of side load separate trend line is used as shown in figure.

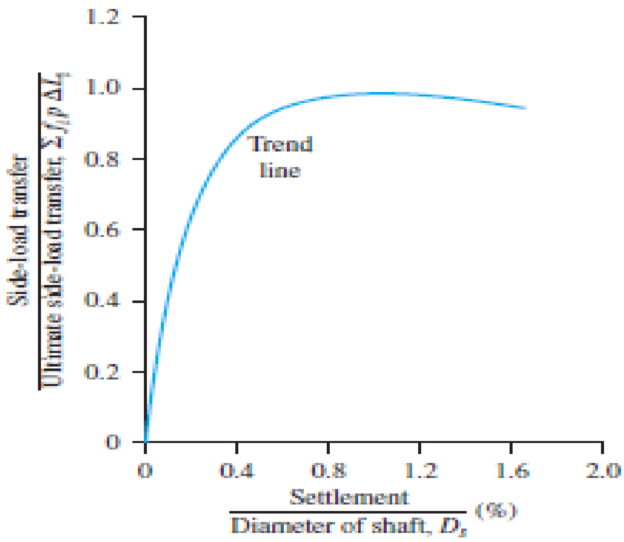


Figure 27 Side-load transfer versus settlement in sand

Rollins et al. (2005) modified the value of β_1 for gravelly sands as follows:

For sand with 25 to 50% gravel,

$$\beta_1 = 2.0 - 0.15(z_i)^{0.75} \quad (0.25 \leq \beta_1 \leq 1.8)$$

For sand with more than 50% gravel,

$$\beta_1 = 3.4(e)^{0.085 z_i} \quad (0.25 \leq \beta_1 \leq 3.0)$$

Figures below provide the normalized side-load transfer trend based on the desired level of settlement for gravelly sand and gravel.

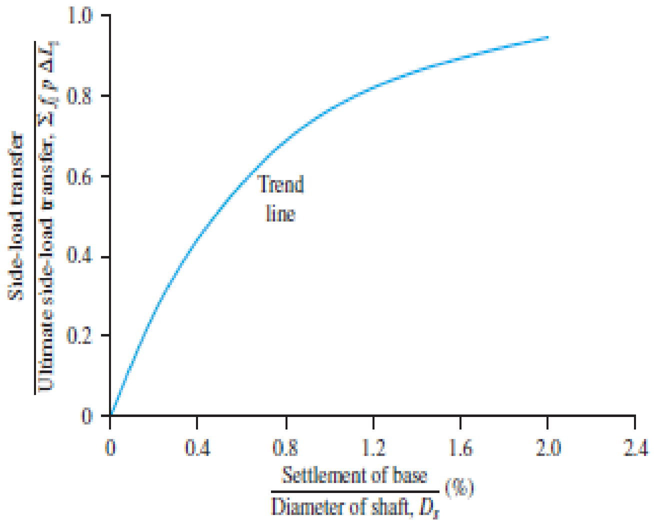


Figure 28 Figure: Side-load transfer versus settlement in sand with 25 to 50%

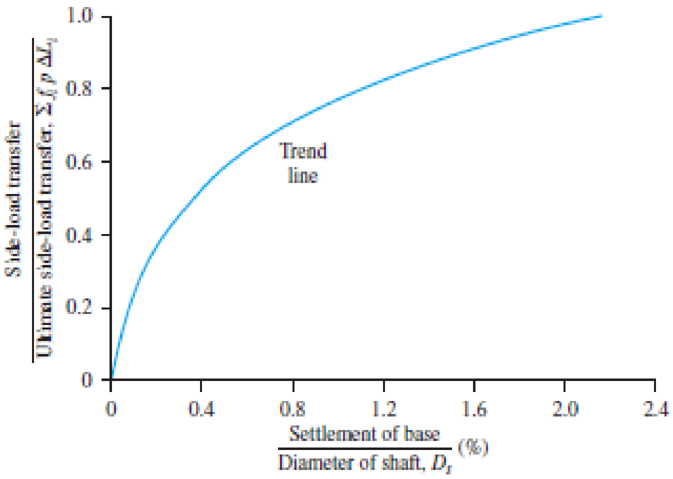


Figure 29 Side-load transfer versus settlement in sand with more than 50% gravel

2.3.20 Load-Bearing Capacity in Clay

Reese and O'Neill (1989) suggested a procedure for estimating the ultimate and allowable bearing capacities for drilled shafts in clay. According to this procedure,

$$Q_{u(net)} = \sum(f_i p \Delta L_i) + A_p q_p$$

Where,

$$f_i = \alpha_i * c_{ui}$$

The expression for q_p can be given as

$$q_p = 6c_{ub}(1 + \frac{0.2L}{D_b}) \leq 9c_{ub} \leq 40p_a$$

If D_b is equal to or greater than 1.91m q_p may be replaced by q_{pr}.

$$q_{pr} = F_r(q_p)$$

Where,

$$F_r = \frac{2.5}{\psi_1 D_b + \psi_2} \leq 1$$

And,

$$\psi_1 = 2.78 \times 10^{-4} + 8.26 \times 10^{-5} \left(\frac{L}{0.001 D_b}\right) \leq 5.9 \times 10^{-4}$$

And,

$$\psi_2 = 0.065\, c_{ub}^{0.5} \qquad (0.5 \leq \psi_2 \leq 1.5)$$

Figures may now be used to evaluate the allowable load-bearing capacity.

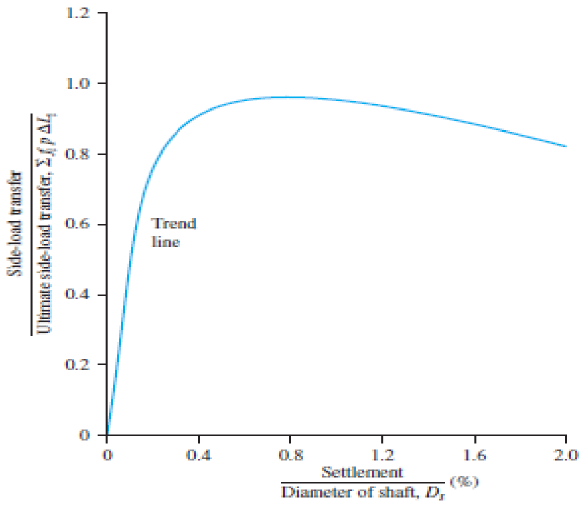


Figure 30 Side-load transfer versus settlement in clays

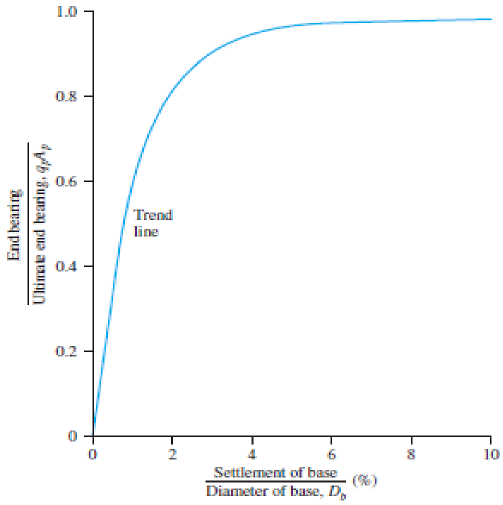


Figure 31 Side-load transfer versus settlement in clays

2.3.21 Drilled Shafts Extending into Rock

Drilled shafts can be extended into rock.

Zhang and Einstein (1998) proposed the relations depending upon the test results of their study. In which,

$$Q_{u(net)} = Q_P + Q_s = \text{fpL} + A_p q_p$$

Where,

$$Q_P(\text{MN}) = A_p q_p = \left[4.83\left(q_u \frac{\text{MN}}{\text{m}^2}\right)^{0.51}\right]\left[A_p(m^2)\right]$$

Figure below shows the plot of q_p (MN/m^2) versus q_u (MN/m^2).

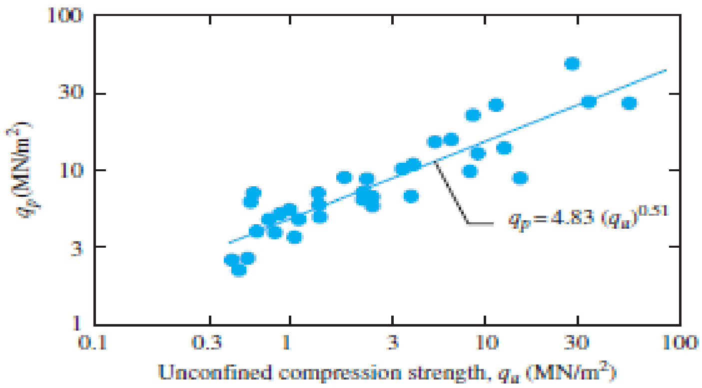


Figure 32 Plot of qp versus qu (Zhang and Einstein, 1998)

And,

For smooth socket,

$$Q_s(\mathrm{MN}) = \mathrm{fpL} = \left[0.4\left(q_u\frac{\mathrm{MN}}{m^2}\right)^{0.5}\right][\pi D_s(\mathrm{m})][L(\mathrm{m})]$$

For rough socket,

$$Q_s(\mathrm{MN}) = \mathrm{fpL} = \left[0.8\left(q_u\frac{\mathrm{MN}}{m^2}\right)^{0.5}\right][\pi D_s(\mathrm{m})][L(\mathrm{m})]$$

CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter explains the methodology of our Final Year Project in which we adopted the following steps to achieve our project objective. Now that we have completed our literature review and established all probable theories applicable we move on to the next step. Please note that the prime objective of this project is the design and analysis of foundations and the end product being the excel sheets and the computer software, constant references from the previous chapter are used here.

Our methodology can be divided into the following major parts:

1. Shallow foundations
    - Granular soil analysis
        - Shear criteria
        - Settlement criteria
    - Cohesive soil analysis
        - Shear criteria
        - Settlement criteria
2. Deep foundations
    - Driven Piles
        - Clays
            - Shear criteria
            - Settlement criteria
        - Sands
            - Shear criteria
            - Settlement criteria

        - Heterogeneous soil
    - Drilled Shafts / Auger Piles
        - Clays
            - Shear criteria
            - Settlement criteria
        - Sands
            - Shear criteria
            - Settlement criteria
        - Heterogeneous soil

3. Development of the excel spreadsheets
4. Conversion of excel spreadsheets into a computer software
5. Verification of the excel sheets and the software with different examples

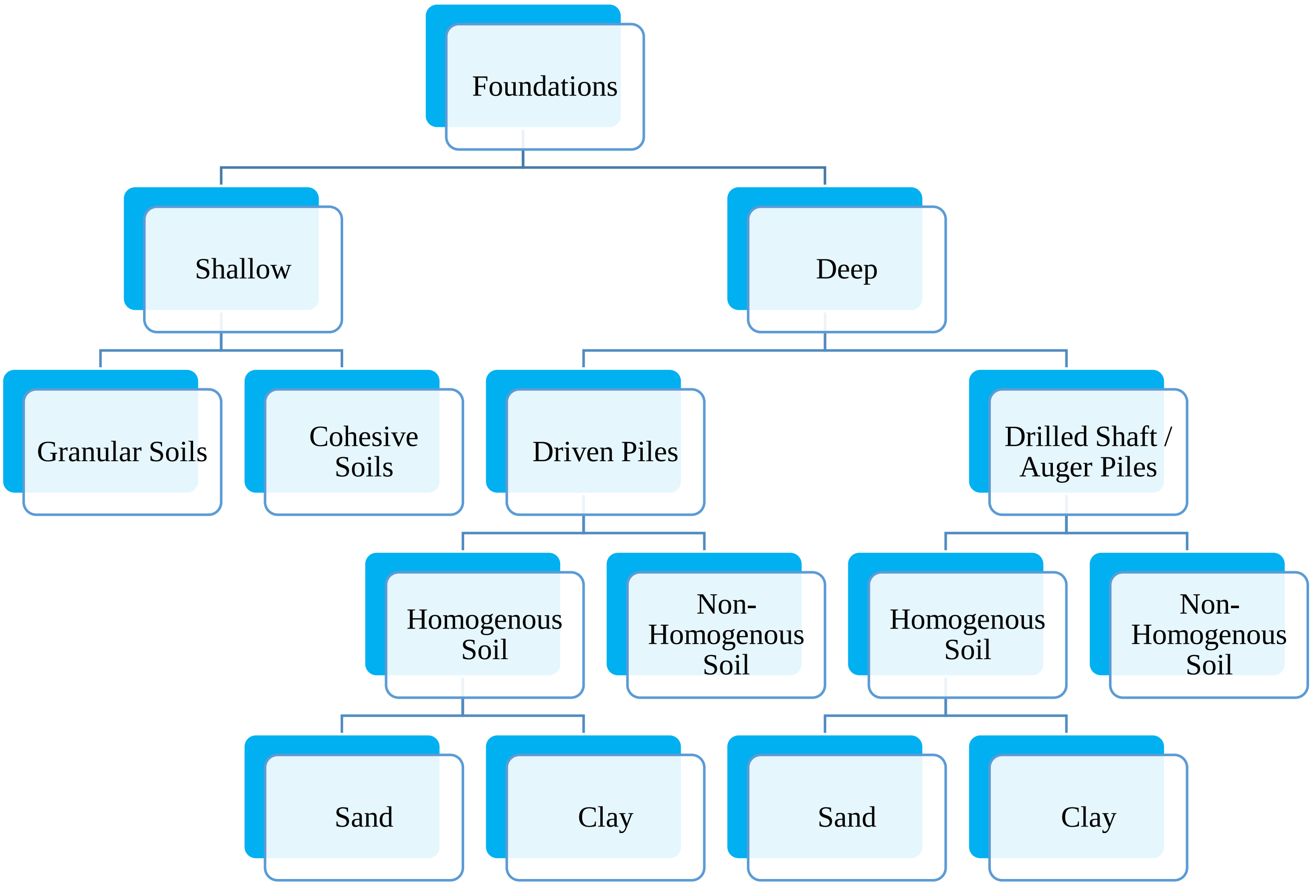


Figure 33 Methodology work flow

3.1 Shallow Foundations

3.1.1 Granular Soils

The work flow for the shallow foundations with granular soils is shown in the flow chart below. The following chart shows the steps in which we proceeded and the various methods we studied for shear and settlement criteria.

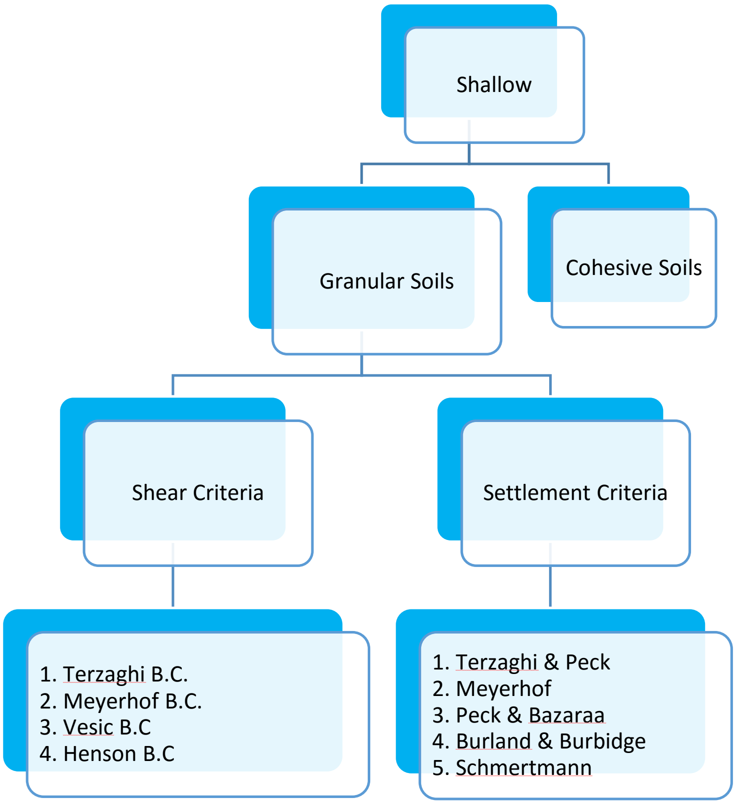


Figure 34 Work flow granular soils

3.1.2 Cohesive Soil

In the second part of the shallow foundations we studied the cohesive soils and the various methods to assess the shear and settlement criteria in these soils. The work flow for this type of soil and the methods involved can be seen in the flow chart below.

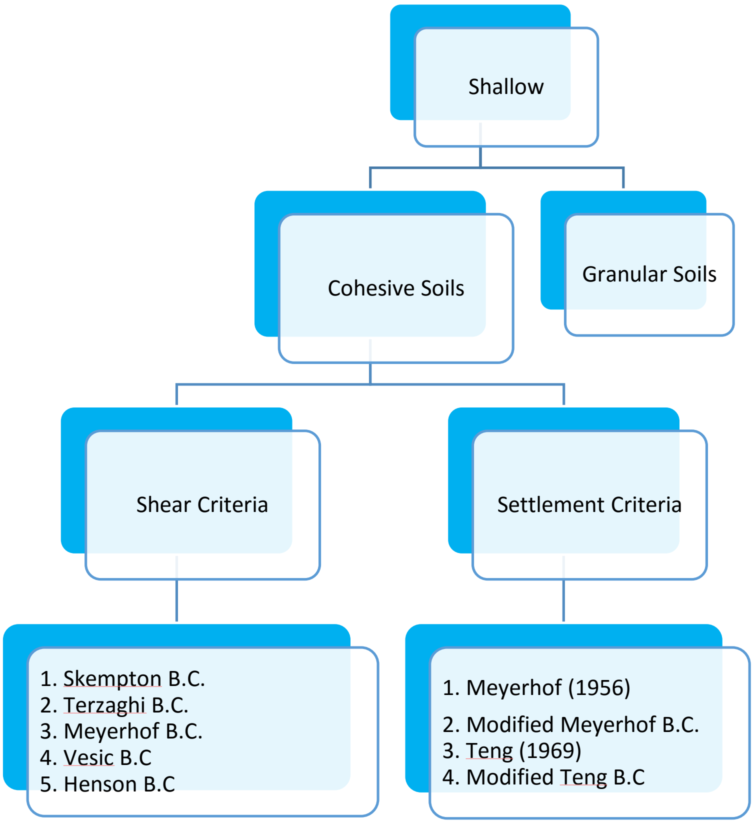


Figure 35 Work flow cohesive soils

3.2 Deep Foundations

3.2.1 Driven Piles

For the study of driven piles system, we firstly divided it into two parts i.e. for homogenous soil strata and heterogeneous soil strata. For homogenous soils we further divided our area of studies in to clayey soils and sandy soils. Again the both types of soils were divided into shear and settlement criteria for their studies. All the methods we considered for our project are mentioned in the flow chart with their details mentioned in the previous chapter.

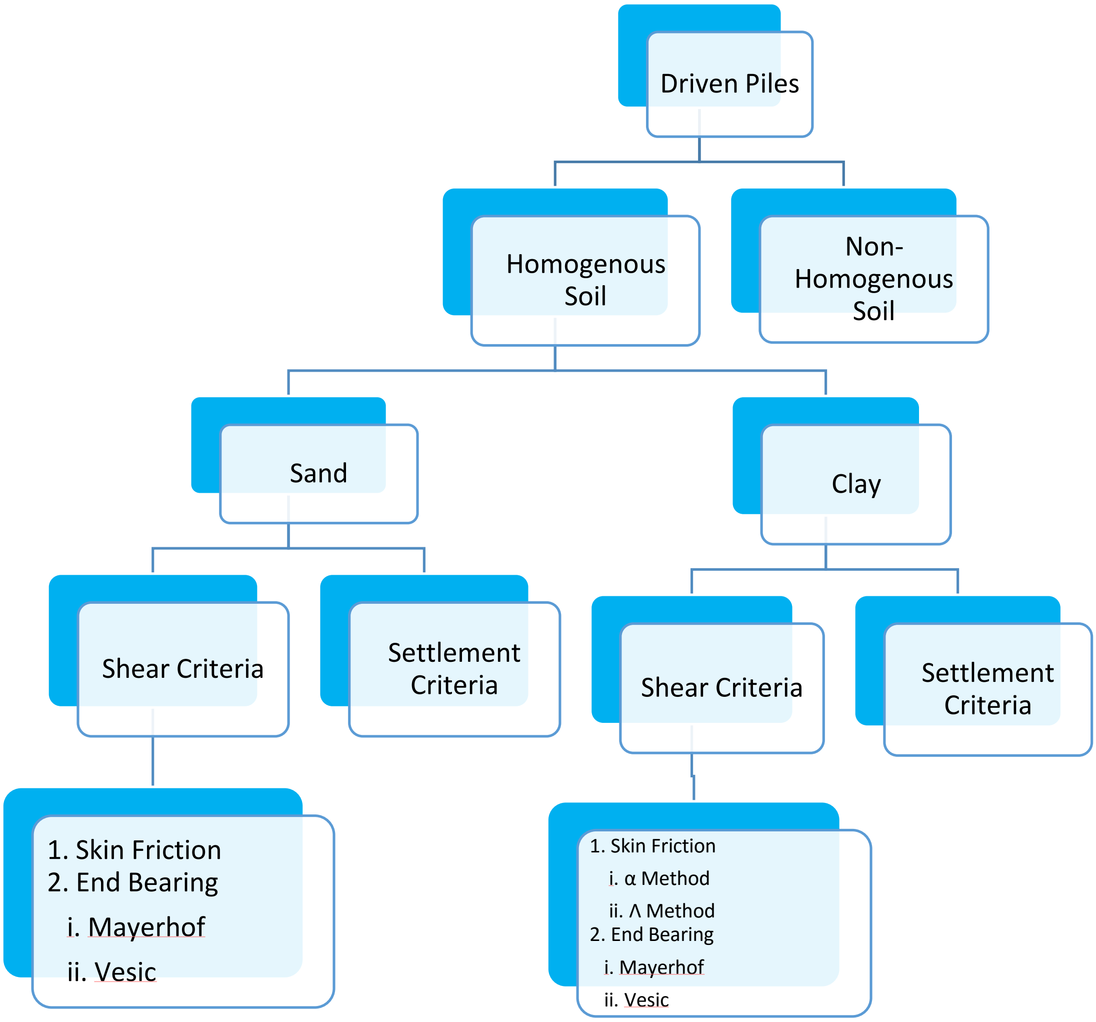


Figure 36 Work flow driven piles

3.2.2 Drilled Shafts/ Auger Piles

Similarly, in drilled shafts / auger piles we followed the same work flow as in case of the driven piles. Here the notable this is that in case of auger piles we studied the Reese O’ Neil method for both the shear and settlement of the piles. The work flow is as illustrated in the following chart.

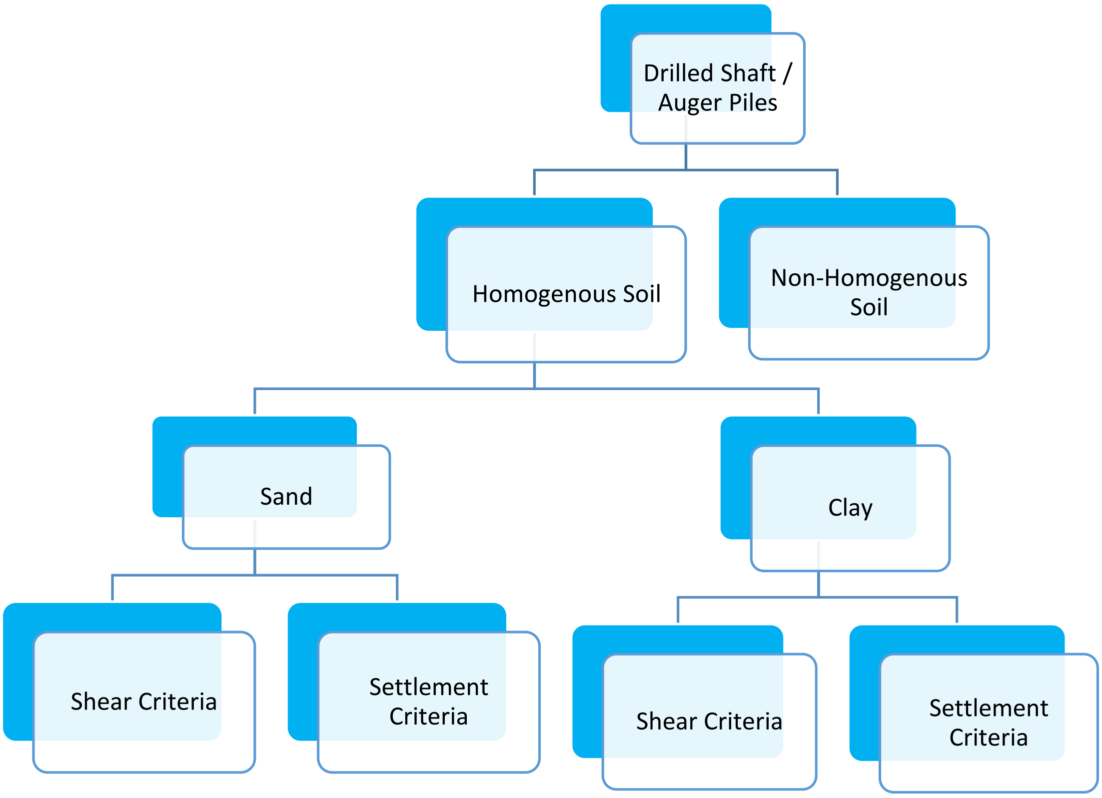


Figure 37 Work flow drilled shaft / auger piles

3.3 Development of Excel Sheets

Next step in our project was to simulate all our studies related to shallow and deep foundations in the form of excel sheets. These excel sheets were developed for the minimum number of inputs from the user so that the maximum automation can be achieved. The inputs were basically field bore hole data and some general soil parameters. These excel sheets were of prime importance because they laid the ground work for the algorithm used for the programming of the software.

3.4 Development of Computer Software

After the formation of all the excel sheets we started off with the programming of these excel sheet into a software code. The language we decided to work on was C# as it the modern form of Visual Basic with much advance control and syntax. We used Microsoft Visual Studio as the development tool for this software. All the algorithm formed in the excel sheets were converted in to this program code. In the following chapter we will discuss more about the capabilities and operation of these excel sheets and the software.

3.5 Verification with examples

The last stage of this project was the verification of the excel sheets and the software code by the help of various examples found in the literature along with some real life examples and cases we found during our study period. The answers of these examples were compared with the hand calculation, excel sheets and the software after which the deviation in these results were checked which came out be negligible. These examples are further discussed in the following chapters.

CHAPTER 4

SOFTWARE

4.1 Introduction

Our project mainly revolves around the analysis and design of foundations using as many as possible geotechnical theories. As manual hand calculation can be too long and cumbersome, therefore, automation was the way to go. Calculations for the problems and various parameters throughout the geotechnical analysis require iterations many times, therefore a program development was the solution.

The preliminary yet extensive research and development was carried out using the Microsoft Excel. Initially the spread sheets were developed and utilized to achieve our objectives, but later on, the use of algorithms developed in excel were used to develop an application which is a stand-alone program.

4.2 Microsoft Excel

We started off with our project's initial stage on Microsoft Excel. Spreadsheet interface of the excel was utilized as the basic interface with is quite iconic. Our main master excel sheet has further 9 work books incorporated so it's a single file for all the foundation design and analysis solution via both shear and settlement criteria.

Our master sheet has following sub sheets:

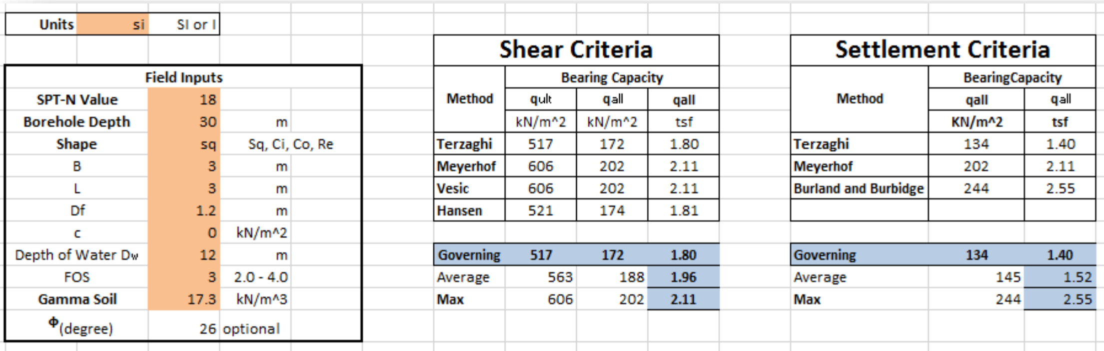
- Shallow Granular
- Shallow Cohesive
- Driven Sand
- Driven Clay
- Driven Sand + Clay
- Driven Heterogeneous
- Auger Clay
- Auger Sand
- Auger Heterogeneous

Excel sheets allowed us to carry out the calculations of bearing capacity both shear and settlement criteria, selection of the governing values, graphical relations to determine the optimum width and depth pf foundations based on both criterions in case of shallow foundations similarly in both pile systems same methodology was adopted.

There were multiple portions that segmented the excel sheet operations. Following are the various spreadsheets that were developed with their interface inputs and outputs.

4.2.1 Shallow Granular

The input panel requires user to input all the variables necessary to perform the calculations.



Units	si	SI or I

Field Inputs		
SPT-N Value	18	
Borehole Depth	30	m
Shape	sq	Sq, Ci, Co, Re
B	3	m
L	3	m
Df	1.2	m
c	0	kN/m^2
Depth of Water Dw	12	m
FOS	3	2.0 - 4.0
Gamma Soil	17.3	kN/m^3
Φ(degree)	26	optional

Shear Criteria			
Method	Bearing Capacity		
	qult	qall	qall
	kN/m^2	kN/m^2	tsf
Terzaghi	517	172	1.80
Meyerhof	606	202	2.11
Vesic	606	202	2.11
Hansen	521	174	1.81
Governing	517	172	1.80
Average	563	188	1.96
Max	606	202	2.11

Settlement Criteria		
Method	BearingCapacity	
	qall	qall
	KN/m^2	tsf
Terzaghi	134	1.40
Meyerhof	202	2.11
Burland and Burbidge	244	2.55
Governing	134	1.40
Average	145	1.52
Max	244	2.55

Figure 38 Excel sheet for shallow granular

B(m)	Qa(shear) in tsf	Qa(settlement) in tsf
1	0.731	1.700
2	0.760	1.481
3	0.828	1.250
4	0.851	1.144
5	0.873	1.084
6	0.902	1.048
7	0.934	1.024
8	0.968	1.008
9	1.003	0.997
10	1.040	0.990
11	1.077	0.986
12	1.114	0.984
13	1.152	0.984
14	1.191	0.984
15	1.229	0.986
16	1.268	0.989
17	1.307	0.992
18	1.346	0.996
19	1.385	1.001
20	1.425	1.006
21	1.464	1.011
22	1.504	1.017
23	1.543	1.023
24	1.583	1.029
25	1.660	1.035

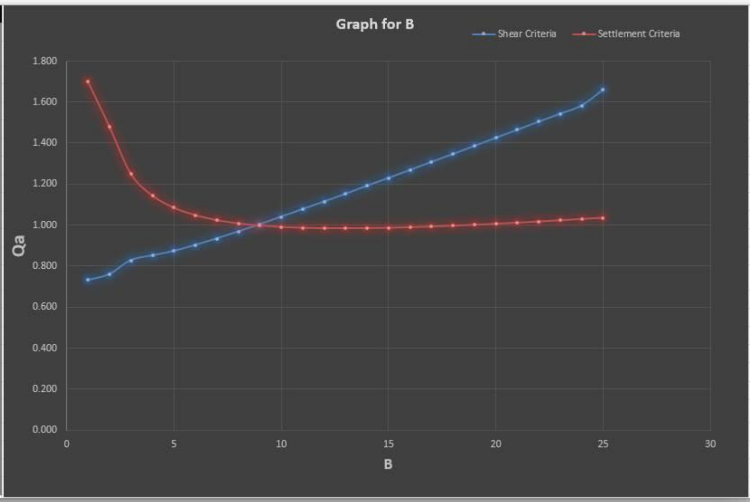


Figure 39 Graph showing optimizations of width based on shear and settlement criteria in granular soils

4.2.2 Shallow Cohesive

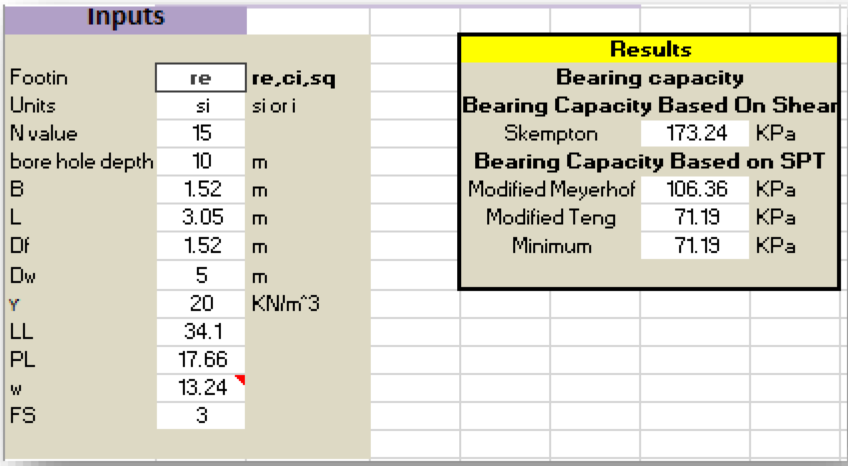


Figure 40 Excel sheet for shallow cohesive

B(m)	Qa(shear) in KPa	Qa(settlement) in Kpa
1	182.41	602.85
2	171.07	240.76
3	173.02	150.91
4	178.31	113.64
5	184.92	93.52
6	192.20	81.00
7	199.86	72.48
8	207.75	66.31
9	215.81	61.65
10	223.97	58.00
11	232.22	55.06
12	240.52	52.66
13	248.88	50.65
14	257.27	48.94
15	265.68	47.48
16	274.13	46.21
17	282.59	45.09
18	291.07	44.11
19	299.56	43.23
20	308.06	42.45
21	316.57	41.75
22	325.10	41.11
23	333.63	40.53
24	342.16	40.00
25	350.70	39.51

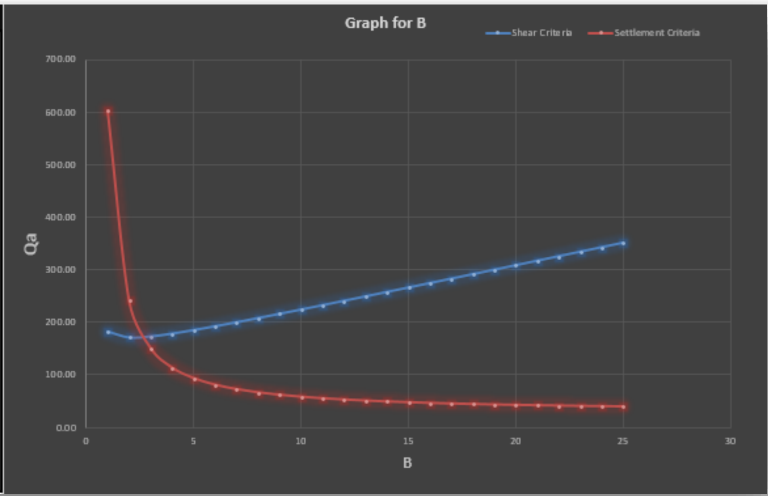


Figure 41 Graph showing optimizations of width based on shear and settlement criteria in cohesive soils

4.2.3 Driven Sand

Driven Piles in SAND

INPUTS		
Shape	C	(sq) or (c)
D	0.365	m
Dw	8	m
Y	12	KN/m^3
FS	2.5	

Group Data				
no of piles in longer Dir (n1)	7			
no of piles in shorter Dir (n2	3	d	1.19	m
Total no of piles	21	Lg	7.48	m
		Bg	2.74	m
		Group Efficiency	0.7104	

BH1	BH2	BH3	BH4	BH5	Avg. N	N60	Cum. N60	σ0'	σ'	(N1)60	Phi(Rad)	Phi(Deg)	Nq*	Qs	Qp	QT	Qallow	Qg(u)	Qg(allow)
7	8	9	88	2	22.8	6.82	6.82	36	36	11.37	0.56	32.10	87.18	45	286.14	330.78	129.17	4934.95	1973.98
5	8	5	5	5	5.6	1.68	4.25	66	72	5.24	0.51	29.30	53.39	160	156.77	316.88	120.48	4727.6	1891.04
6	6	6	6	6	6	1.80	3.43	66	98	4.23	0.50	28.66	47.73	239	136.49	375.29	140.70	5599.02	2239.61
3	3	3	3	3	3	0.90	2.80	66	100	3.45	0.49	28.12	43.42	264	121.40	385.32	143.66	5748.53	2299.41
4	4	4	4	4	4	1.20	2.48	66	105	3.06	0.49	27.83	41.25	316	113.92	429.64	159.30	6409.79	2563.92
6	6	6	6	6	6	1.80	2.36	66	111	2.92	0.48	27.72	40.48	394	111.28	505.47	186.49	7541.06	3016.42
3	3	3	3	3	3	0.90	2.15	66	118	2.66	0.48	27.52	39.05	472	106.44	578.38	212.52	8628.84	3451.54
8	8	8	8	8	8	2.39	2.18	66	124	2.70	0.48	27.55	39.26	551	107.13	657.91	241.19	9815.38	3926.15
8	8	8	8	8	8	2.39	2.21	66	131	2.72	0.48	27.57	39.42	630	107.67	737.29	269.81	10999.71	4399.88
8	8	8	8	8	8	2.39	2.23	66	138	2.75	0.48	27.59	39.54	708	108.10	816.57	298.38	12182.48	4872.99
8	8	8	8	8	8	2.39	2.24	66	144	2.77	0.48	27.60	39.65	787	108.45	895.78	326.92	13364.12	5345.65
8	8	8	8	8	8	2.39	2.25	66	151	2.78	0.48	27.62	39.73	866	108.74	974.92	355.44	14544.91	5817.96
8	8	8	8	8	8	2.39	2.27	66	157	2.79	0.48	27.63	39.81	945	108.99	1054.03	383.94	15725.05	6290.02
8	8	8	8	8	8	2.39	2.27	66	164	2.81	0.48	27.64	39.87	1024	109.21	1133.09	412.43	16904.67	6761.87
8	8	8	8	8	8	2.39	2.28	66	170	2.82	0.48	27.64	39.92	1103	109.47	1212.42	441.02	18088.09	7235.24

Figure 42 Excel sheet for driven sand

Settlement			Sg(e)	56.864	mm
Immediate					
Qpw	152	KN	D	0.365	m
Qsw	350	KN	P	1.14668	m
L	21	m	A	0.10463	m^2
Es	25000	KN/m^2			
			Ep	2.1E+07	KN/m^2
			Iws	4.655	
Se=Se(1)+Se(2)+Se(3)					
Se	20.76	mm			
	0.82	in			
Sg(e)	56.864	mm			

Figure 43 Settlement calculation in driven sand

4.2.4 Driven Clay

INPUTS		
Shape	sq	(sq) or (c)
D	0.45	m
FS	2.5	
Dw	0	m
Y	18	KN/m^3

GROUP DATA				
no of piles in longer Dir (n1)	3	d	1.46	m
no of piles in shorter Dir (n2	3	Lg	3.38	m
Total no of piles	9	Bg	3.38	m
		Group Efficiency	0.7466	

Perimete	1.800	m
Area	0.203	m^2

Dept(m)	BH1	BH2	BH3	BH4	BH5	Avg. N	N60	Cum.N60	σ0'	$\bar{\sigma}'_o$	(N1)60	Cu(kpa)	Meyerof Qp(KN)	Vesic Irr	Nc*	Qp(KN)	Qp(KN)	α Method Qs	λ Method Qs	Qs(KN)	QT(KN)	Qallow	Qg(u)	Qg(all)(KN
3	8	7	9	8	8	8	2.39	2.39	25	12	4.83	14.36	26.18	16.84	7.67	22.31	22.31	77.97	80.06	77.97	100.27	40.11	673.81	269.52
6	9	6	5	9	8	7.4	2.21	2.30	49	25	3.29	13.83	25.20	14.97	7.51	21.03	21.03	152.31	187.15	152.31	173.34	69.34	1164.80	465.92
9	6	6	6	6	6	6	1.80	2.13	74	37	2.49	12.81	23.34	11.44	7.15	18.55	18.55	217.97	307.44	217.97	236.53	94.61	1589.38	635.75
10	7	8	9	4	3	6.2	1.86	2.06	82	41	2.28	12.39	22.58	9.99	6.97	17.49	17.49	237.29	348.78	237.29	254.78	101.91	1712.05	684.82
12	8	9	4	4	4	5.8	1.74	2.00	98	49	2.02	11.99	21.86	8.62	6.78	16.46	16.46	279.13	438.11	279.13	295.59	118.23	1986.23	794.49
15	6	6	6	6	6	6	1.80	1.97	123	61	1.77	11.79	21.49	7.91	6.66	15.91	15.91	345.26	579.64	345.26	361.16	144.47	2426.90	970.76
18	10	5	8	7	3	6.6	1.98	1.97	147	74	1.62	11.80	21.50	7.94	6.67	15.93	15.93	414.49	723.12	414.49	430.42	172.17	2892.30	1156.92
21	8	8	8	8	8	8	2.39	2.02	172	86	1.54	12.12	22.09	9.06	6.84	16.79	16.79	491.62	865.83	491.62	508.41	203.36	3416.32	1366.53
24	8	8	8	8	8	8	2.39	2.06	197	98	1.47	12.37	22.54	9.92	6.96	17.44	17.44	568.93	998.25	568.93	586.37	234.55	3940.21	1576.08
27	8	8	8	8	8	8	2.39	2.09	221	111	1.41	12.57	22.91	10.61	7.05	17.95	17.95	646.38	1117.94	646.38	664.33	265.73	4464.04	1785.62
30	8	8	8	8	8	8	2.39	2.12	246	123	1.35	12.73	23.20	11.18	7.12	18.36	18.36	723.92	1223.76	723.92	742.28	296.91	4987.86	1995.14
33	8	8	8	8	8	8	2.39	2.14	270	135	1.30	12.87	23.45	11.65	7.18	18.70	18.70	801.53	1315.92	801.53	820.23	328.09	5511.67	2204.67
36	8	8	8	8	8	8	2.39	2.16	295	147	1.26	12.98	23.66	12.05	7.22	18.99	18.99	879.20	1395.89	879.20	898.19	359.28	6035.49	2414.20
39	8	8	8	8	8	8	2.39	2.18	319	160	1.22	13.08	23.84	12.39	7.26	19.23	19.23	956.91	1466.46	956.91	976.14	390.46	6559.32	2623.73
42	8	8	8	8	8	8	2.39	2.19	344	172	1.18	13.17	24.00	12.69	7.29	19.44	19.44	1034.66	1531.71	1034.66	1054.10	421.64	7083.15	2833.26

Figure 44 Excel sheet for driven clay

Settlement		ST	62.752	mm		
Immediate				D	0.45	m
Qp	177.63	KN		P	1.800	m
Qs	1208	KN		A	0.203	m^2
L	15	m		Qp(allow)	71.052	KN
Es	25000	KN/m^2		Qs(allow)	483.2	KN
				Ep	2.1E+07	KN/m^2
				Iws	4.021	
Se=Se(1)+Se(2)+Se(3)						
Se	6.59	mm				
	1.43	in				
Sg(e)	18.041	mm				

Consolidation					
Dh	50	m	Z0	10	m
eo	0.82		ZE	40	m
Cc	0.3		Zc	20	m
Dw	0	m	Dc	30	m
Lg	3.3	m	σo'	245.7	
Bg	2.2	m	Lg	3.30	m
Yw	9.81	KN/m^3	Bg	2.20	m
Qg(allow	2000	KN	Δσo'	3.867	
			L	15	m
			Yc	18	KN/m^3
ΔSc	44.7111	mm			

Figure 45 Settlement calculations in driven clay

4.2.5 Driven Sand + Clay

INPUTS		
SHAPE	c	sq / c
D	1.00	m
Dw	8	m
Y(sand)	12	KN/m^3
Y(clay)	20	KN/m^3
Dept. of Sand layer	25	m
FS	2.5	

GROUP DATA				
no of piles in longer Dir (n1)	7	d	3.25	m
no of piles in shorter Dir (n2	3	Lg	20.50	m
Total no of piles	21	Bg	7.50	m
		Group Efficiency	0.7104	

Dept(m)	Soil Type	BH1	BH2	BH3	BH4	BH5	Avg. N	N60	Cum. N60	σ0'(sand)	σ'(T.O.B)	σ'	(N1)60	Phi(Rad)	Phi(Deg)	Cu(kpa)	SKIN FRICTION SAND Qs(KN)	CLAY Qs(KN)	Qs KN	Sand Nq*	Qp	POINT CAPACITY Clay Meyerof Qp(KN)	Vesic Irr	Nc*	Qp(KN)
3	SAND	15	8	9	88	2	24.4	4.49	4.49	36	36	36	7.48	0.53	30.33		121		121	63.97	1469.88				
6	SAND	15	8	5	5	5	7.6	4.49	4.49	72	72	72	5.29	0.51	29.36		481		481	53.94	1191.66				
9	SAND	15	6	6	6	6	7.8	4.49	4.49	111	111	98	4.25	0.50	28.83		1110		1110	49.15	1062.39				
10	SAND	15	8	5	14	3	9	4.49	4.49	111	111	100	4.25	0.50	28.83		1234		1234	49.15	1062.39				
12	SAND	15	4	4	4	4	6.2	4.49	4.49	111	111	105	4.25	0.50	28.83		1481		1481	49.15	1062.39				
15	SAND	15	6	6	6	6	7.8	4.49	4.49	111	111	111	4.25	0.50	28.83		1851		1851	49.15	1062.39				
18	SAND	15	3	3	3	3	5.4	4.49	4.49	111	111	118	4.25	0.50	28.83		2221		2221	49.15	1062.39				
21	SAND	15	8	8	8	8	9.4	4.49	4.49	111	111	124	4.25	0.50	28.83		2591		2591	49.15	1062.39				
24	SAND	15	8	8	8	8	9.4	4.49	4.49	111	111	131	4.25	0.50	28.83		2961		2961	49.15	1062.39				
27	CLAY	8	8	8	8	8	8	2.39	4.28	111	132	138	4.06			25.68	2961	129.67	3091			181.49	56.09	9.27	187.0
30	CLAY	8	8	8	8	8	8	2.39	4.11	111	162	144	3.89			24.65	2961	316.12	3277			174.22	52.53	9.19	177.8
33	CLAY	8	8	8	8	8	8	2.39	3.97	111	193	151	3.76			23.79	2961	494.90	3456			168.16	49.55	9.11	170.1
36	CLAY	8	8	8	8	8	8	2.39	3.84	111	223	157	3.64			23.07	2961	667.65	3629			163.04	47.04	9.04	163.7
39	CLAY	8	8	8	8	8	8	2.39	3.74	111	254	164	3.55			22.44	2961	835.59	3797			158.65	44.88	8.98	158.2
42	CLAY	8	8	8	8	8	8	2.39	3.65	111	285	170	3.46			21.91	2961	999.59	3961			154.84	43.01	8.92	153.4

Figure 46 Excel sheet for driven sand + clay

Settlement		ST	18.050 mm

Immediate (SAND)					
			D	1.00	m
			P	3.142	m
Qs(SAND)	1208	KN	A	0.785	m^2
L	25	m	Qs(allow)	483.2	KN
Es	25000	KN/m^2	Ep	2.1E+07	KN/m^2
			Iws	3.750	
Se=Se(1)+Se(2)+Se(3)					
Se	1.28	mm			
	1.43	in			
Sg(e)	**3.50**	**mm**			

Immediate (CALY)					
			D	1.00	m
Qp	177.63	KN	P	3.142	m
Qs(CLAY)	1208	KN	A	0.785	m^2
L	42	m	Qp(allow)	71.052	KN
Es	25000	KN/m^2	Qs(allow)	483.2	KN
			Ep	2.1E+07	KN/m^2
			Iws	3.443	
Se=Se(1)+Se(2)+Se(3)			L(clay)	17	
Se	3.82	mm			
	1.43	in			
Sg(e)	**10.46**	**mm**			

Consolidation					
Dh	30	m	Z0	28	m
eo	0.82		ZE	2	m
Cc	0.3		Zc	1	m
Qg(allow	2000	KN	Dc	24	m
			σo'	377.79	KN/m^2
			Lg	20.50	m
			Bg	7.50	m
			Δσo'	10.944	
			LCT	5	m
			Dw	8	m
ΔSc	**4.09**	**mm**	L	42	m
			Yc	20	KN/m^3
			Yw	9.81	KN/m^3
			DCT	29	m
			Ys	12	KN?m^3
			L(sand)	25	m

Figure 47 Settlement calculations in driven sand + clay

4.2.6 Driven Heterogeneous

inputs		
Unit	SI	si/i
# of layer	3	
Shape	C	sq,c
D	1	m
Dw	30	m
FS	3	

Group Piles				
no of piles in longer Dir (n1)	6			
no of piles in shorter Dir (n2)	1	d	3.25	m
Total no of piles	6	Lg	17.25	m
		Bg	1.00	m
		Group Efficiency	0.8416	

RESULT		
Qu	7897.773	KN
Qa	2632.591	KN
Group Capacity		
Qu(g)	39882.56	KN
Qa(g)	13294.19	KN

				OPTIONAL												Sand		Clay				
																		Meyerof	Vesic			
layer #	sand/clay	Y(KN/m^3)	Thickness(m)	C	Φ	N value	Y	Thickness	N60	(N1)60	C	Φ(deg)	σ'	fs	Qs	Nq*	Qp	Qp	Irr	Nc*	Qp	**Qp**
1	clay	18	1.5	26		2	18	1.5	0.5985	1.151814	26		27.00	20.80	98.00							
2	sand	19	6		32	14	19	6	4.1895	3.528195		32.00	141.00	50.81	957.76							
3	sand	21	12.5		33	23	21	12.5	6.88275	3.983729		33.00	298.50	107.96	4239.51	102.05	2602.50					2602.501
4							0	0	0	0			298.50									
5							0	0	0	0			298.50									
														Qs	5295.27	KN					Qp	2602.501 KN

Figure 48 Excel sheet for driven heterogeneous

4.2.7 Auger Sand

INPUTS		
# of layers	2	
SHAPE	C	SQ,C
Ds	1	m
Db	1.5	m
Hb	1	m
Dw	15	m
Settlement	12	mm
Using graphs		
Qs(a)/Qs(n)		
Qp(a)/Qp(n)		

layer Data	layer 1	layer 2	layer 3	layer 4	layer 5		
thickness(m)	6	1					
Y(kN/m3)	16	19					
N	15	30					
Effect.L	6	0	0	0	0		
zi	3	6.5	7	7	7		
σci	49.29	106.79	115.00	115.00	115.00		
β	1.658	1.389	1.354	1.354	1.354		
fs	81.72	148.37	155.76	155.76	155.76		
Qs(KN)	1540.37	0.00				1540.37	KN
qp		1725					
Qp(b)						2580.92	KN

Qs(a)	1241.138	
Qp(a)	688.4192	
Qa	1929.56	KN

Figure 49 Excel sheet for auger sand

4.2.8 Auger Clay

INPUTS		
# OF LAYER	3	
Shape	CI	CI,SQ
Ds	0.76	m
Db	1.2	m
Hb	1.5	m
Dw	15	m
SETTLEMENT	12	mm
Using graphs		
Qs(a)/Qs(u)	0.9	
Qp(a)/Qp(u)	0.6	

layer Data	layer 1	layer 2	layer 3	layer 4	layer 5		
thickness(m)	3	3	2.5				
Y1(kN/m3)	16	19	20				
N							
Cu(opt)(Kpa)	40	60	145				
α	0.55	0.55	0.55	0.55	0.55		
Su(kpa)	40.000	60	145	14.84	14.84		
eff.length	1.50	3.000	0.24	0	0		
Qs	78.79	236.37	45.70			360.86	KN
qp			1305				
Qp						1475.92	KN

Qs(a)	324.78	KN
Qp(a)	885.55	KN
Qa	1210.33	KN

Figure 50 Excel sheet for auger clay

4.2.9 Auger Heterogeneous

inputs		
Unit	si	si/i
# of layer	3	
Shape	C	sq,c
Ds	0.76	m
Db	1.2	m
Dw	45	m
Hb	1.5	m
FS	2.5	

Group Piles				
no of piles in longer Dir (n1)	6			
no of piles in shorter Dir (n2)	1	d	2.47	m
Total no of piles	6	Lg	13.11	m
		Bg	0.76	m
		Group Efficiency	0.8416	

RESULT		
Qu	6085.08	KN
Qa	2434.03	KN
Group Capacity		
Qg(u)	30728.7	KN
Qg(a)	12291.5	KN

								Sand			Clay										
layer #	sand/clay	Y(KN/m^3)	Thickness(m)	Cu(Kpa)	N value	Y	Thickness	Zi	σz'	β	α	Cu	fs	Qs	Nc*	qp	Ψ1	Ψ2	Fr	qpr	Qp
1	CLAY	18	10	40	20	18.00	10.00				0.55	40.00	22.00	446.48							
2	SAND	20	10		15	20.00	10.00	15.000	280.000	0.55			154.31	3684.40							
3	CLAY	22	20	60	18	22.00	20.00	30.000	600.000		0.55	60.00	33.00	1397.75	8.2	492	3.10267	0.50349	0.50181	492	556.439
4						0.00	0.00														
5						0.00	0.00														
													Qs	5528.64	KN					Qp	556.439 KN

Figure 51 Excel sheet for auger heterogeneous

4.2.3 Limitations of Microsoft Excel

Although excel provided support for all the calculations, there were some limitations that needed the shift to visual studio:

- Level of accuracy desired could not be ascertained using Microsoft Excel.
- Design and revision of footing parameters could not be conducted in excel.

- The spreadsheets were dependent on the platform of Microsoft Excel, in order to develop a stand-along program, the need of a C# based program was paramount.
- Excel, despite its benefits, did not give a pleasing user interface.
- Real-time checks could not be developed in Excel, as opposed to C#.

4.3 Microsoft Visual Studio

Once the ground work was set on the Microsoft Excel, in order to curb and improve all the limitations of the spreadsheets, the logic was developed and imported to Microsoft Visual Studio. Visual Studio provided a platform to write the entire algorithm on C#, and develop an application based on Windows forms. This platform allowed us the manipulation and creation of the customized user interface and the division of program into multiple modules, thereby simplifying the task yet increasing the overall efficiency.

4.3.1 Technical Specifications

Language: C# (C Sharp)

GUI: Windows Forms

Modules: 4 (~3000+ lines of code)

Development Tool: Microsoft Visual Studio Community Edition 2015

4.3.2 Elements of the Program

The program is the composite of all the geotechnical theories pertaining to design and analysis of shallow and deep foundations. Like all program, this one is also divided into multiple components/modules. Each component has its own set of functions with further sub divisions. We will be detailing the functionalities and how to proceed with program subsequently.

Project Summary

In the project summary screen, the inputs are the project title, analysis, author, company and date. All these inputs are stored here and then displayed in the final results and reports.

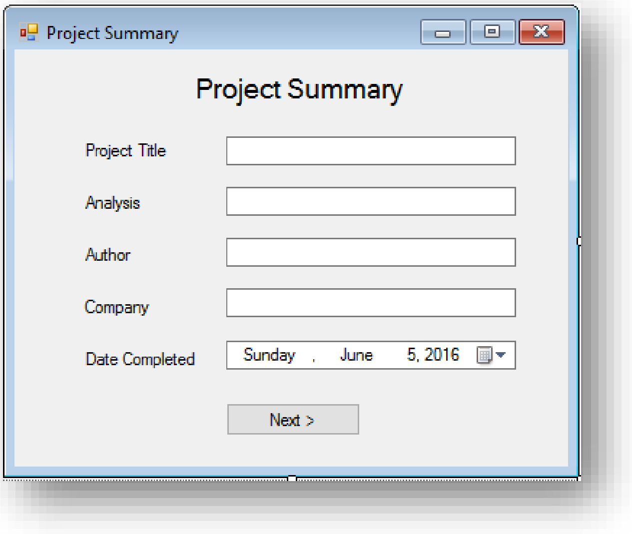


Figure 52 Software - Project summary screen

Foundation Selection

The next screen after pressing the next button on project summary screen is the foundation selection screen. Here you get the option to choose from the 4 modules of this software,

- Shallow Foundation > Granular
- Shallow Foundation > Cohesive
- Deep Foundation > Drilled
- Deep Foundation > Driven

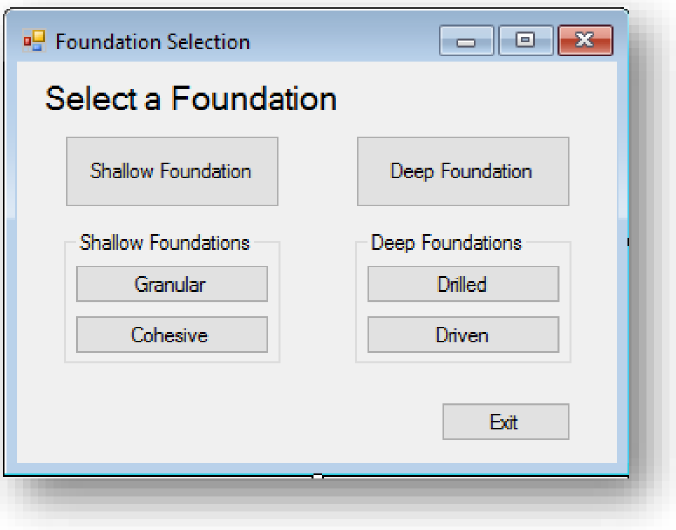


Figure 53 Software - Foundation selection screen

Shallow Foundation – Granular

If you press the shallow foundation the option of granular and cohesive appears on the same screen and when you further press granular button the screen below appears which is the input screen for granular soil and after filling all the inputs, click the calculate button to get the results.

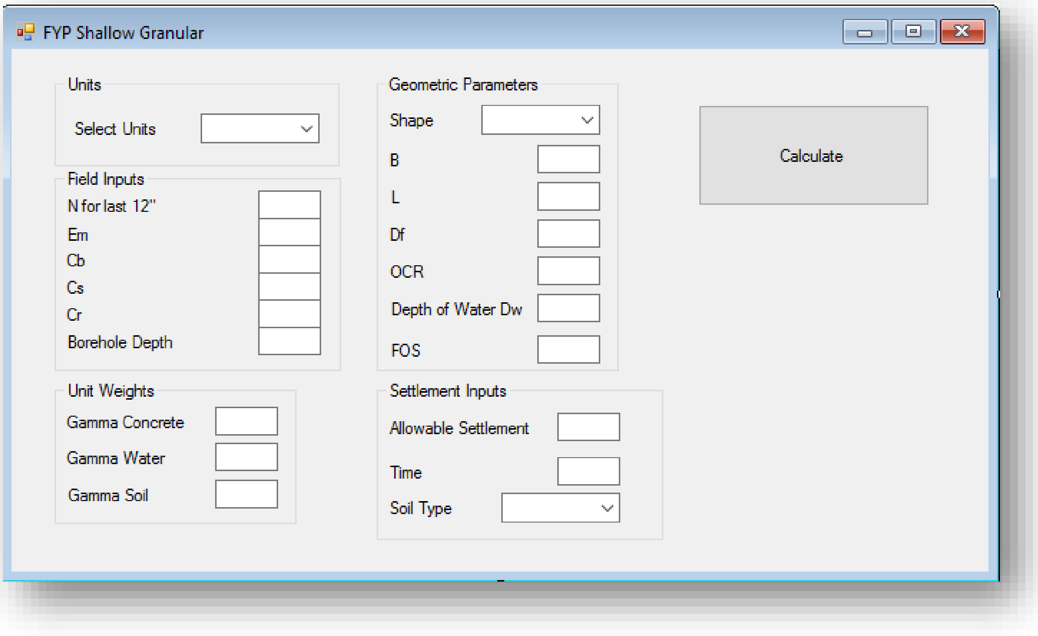


Figure 54 Software - Shallow Granular

Shallow Foundation – Cohesive

If you press the shallow foundation the option of granular and cohesive appears on the same screen and when you further press cohesive button the screen below appears which is the input screen for cohesive soil and after filling all the inputs, click the calculate button to get the results.

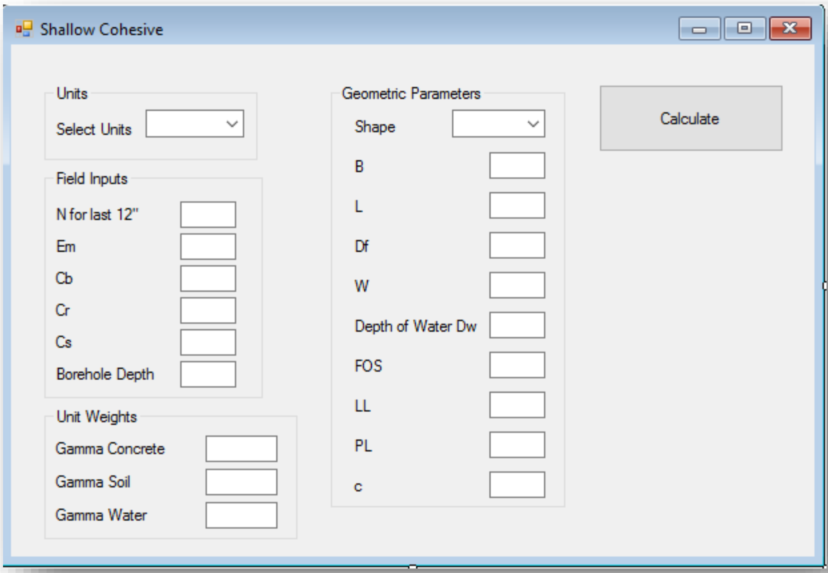


Figure 55 Software - Shallow Cohesive

Deep Foundation – Drilled

Similarly, if you press the deep foundation the option of drilled and driven appears on the same screen and when you further press drilled button the screen below appears which is the input screen for heterogeneous drilled piles and after filling all the inputs, click the calculate button to get the results.

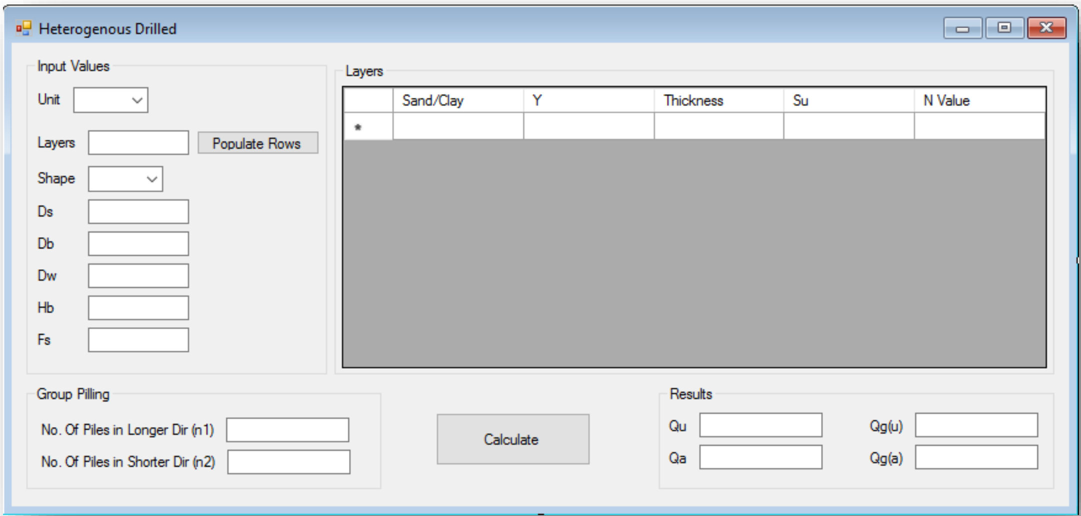


Figure 56 Software - Deep Drilled

Deep Foundation – Driven

Similarly, if you press the deep foundation the option of drilled and driven appears on the same screen and when you further press driven button the screen below appears which is the input screen for heterogeneous driven piles and after filling all the inputs, click the calculate button to get the results.

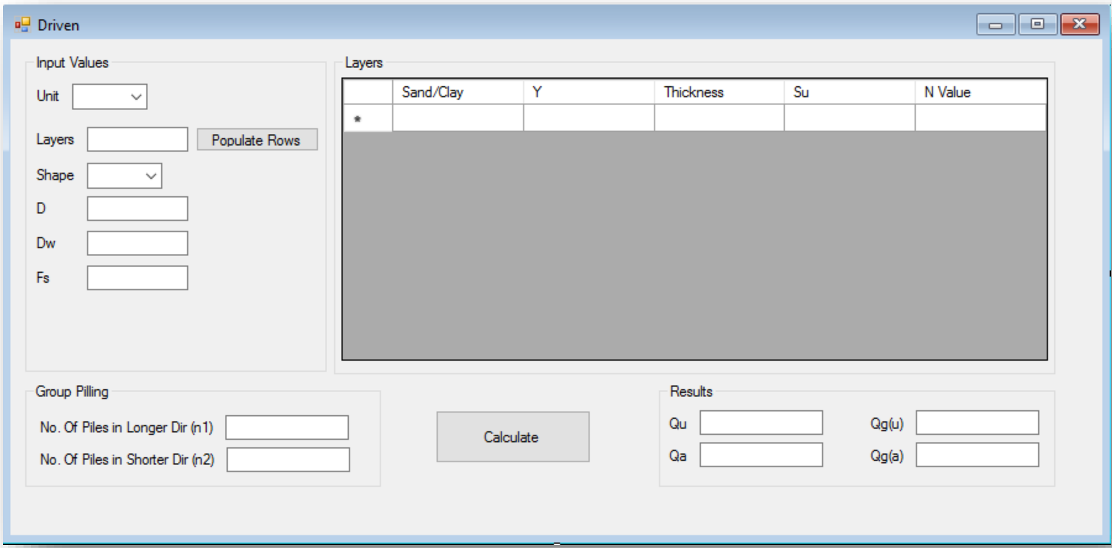


Figure 57 Software - Deep Driven

4.3.3 Incorporated Theories on Shallow Foundations

Shear Criteria

The following theories have been employed in the development of the analysis program.

- Terzaghi's Bearing capacity method
- Meyerhof's Bearing capacity method
- Vesic's Bearing capacity method
- Hanson's Bearing capacity method
- Skempton's Bearing capacity method

Settlement Criteria

The following theories have been used in case of settlement criteria.

- Terzaghi's Method for Settlement
- Meyerhof's Method for Settlement
- Burland and Burbidge's Method for Settlement
- Schmertmann's Method for Settlement
- Modified Meyerhof's Method
- Modified Teng's Method

4.3.4 Incorporated Theories on Deep Foundations

Driven

- Meyerhof
- Vesic
- Based on standard penetration
- λ-Method
- α-Method

Drilled

The drilled shafts have been analyzed by using "Reese and O' Neil Method".

CHAPTER 5

VERIFICATION OF DEVELOPED EXCEL SHEETS AND SOFTWARE

5.1 Introduction

We have taken examples from various books, some real life cases and projects to verify the results of the software and the excel sheets that our group developed.

5.1.1 Example 1

A soil data is given below in table. A drilled shaft with a bell is placed in a layers of soil (sand sandwiched between clay layers). Determine the allowable load the drilled shaft could carry. Use factor of safety of 2.5. The drilled shaft has diameter of shaft 0.76m, diameter of bell 1.2m and height of bell 1.5m. Water was encounter at depth of 45m during performing SPT.

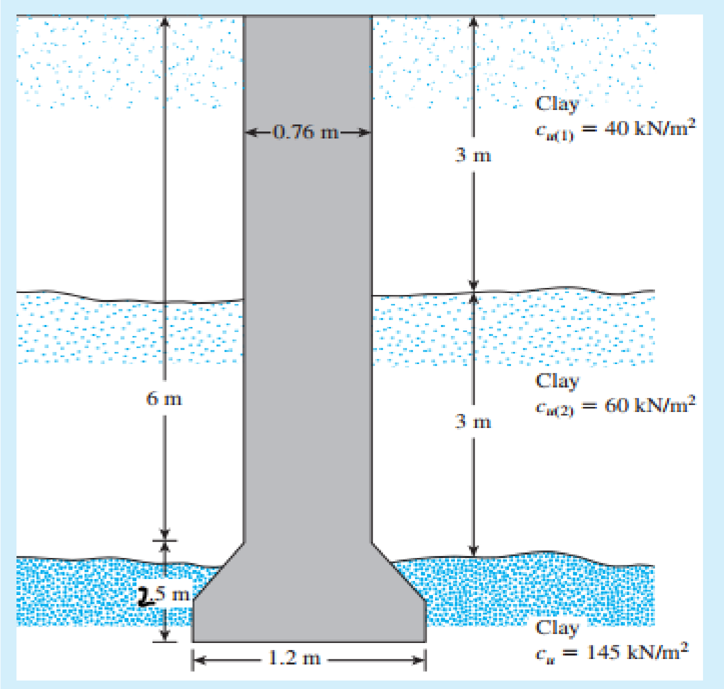
Layer	ϒ(KN/m^3)	Thickness (m)	Cu (KPa)	SPT N-value
Clay	18	10	40	20
Sand	20	10		15
Clay	22	20	60	18

Results

Subject	**Allowable Load**
By hand calculation	2433 KN
Using excel Sheet and program	2434.03 KN
Percentage variation	0.04%

5.1.2 Example 2

This is the example illustrating the method for calculating allowable bearing capacity of pile. It is example 12.5 and page number 666 of book "Principal of Foundation Engineering 7th edition by Baraj M Das". Results are given below.



A drilled shaft in layered clay

Results

Subject	**Allowable Bearing Capacity**
By hand calculation	1211 KN
Using excel Sheet and program	1210.33 KN
Percentage variation	0.06%

5.1.3 Example 3

A soil data is given below in table. A driven pile is inserted in a layers of soil. Determine the allowable load driven pile can carry. Use factor of safety of 3. The pile has diameter of 1 m. Water encountered at depth of 30m performing SPT.

Layer	ϒ(KN/m^3)	Thickness (m)	Cu (kPa)	ϕ (degree)	SPT N-value
Clay	18	1.5	26		2
Loose Sand	19	6		32	14
Dense sand	21	12.5		33	23

Results

Subject	Allowable Bearing Capacity
By hand calculation	2616 KN
Using excel Sheet and program	2632.59 KN
Percentage variation	0.63%

5.1.4 Example 4

Find the bearing capacity of shallow foundation using shear criteria having square footing of width 3m and depth of embedment 1.2m with soil properties ϒ=17.30KN/m^3, ϕ=26 degree, c=0, SPT-N value 18 and borehole depth 30m.

Results

Bearing capacity

Subject	Terzaghi	Meyerhof	Vesic	Hanson
Hand calculation	166.43 KN/m^2	202 KN/m^2	202 KN/m^2	174 KN/m^2
Program	172 KN/m^2	202 KN/m^2	202 KN/m^2	174 KN/m^2
% Variation	3.35%	0.0%	0.0%	0.0%

5.1.5 Example 5

This is the example illustrating the method for calculating settlement of pile. It is example 11.10 and page number 590 of book "Principal of Foundation Engineering 7th edition by Baraj M Das".

Results

Subject	Settlement
Example	19.69 mm
Excel sheet and Software	20.76 mm
Percentage variation	5.4%

5.1.6 Example 6

This is the example illustrating the method for calculating settlement of group piles. It is example 11.20 and page number 627 of book "Principal of Foundation Engineering 7th edition by Baraj M Das".

Results

Subject	Settlement
Example	44.7 mm
Excel sheet and Software	44.71 mm
Percentage variation	0.02%

5.1.7 Example 7

Find the bearing capacity of shallow foundation using settlement criteria having square footing of width 3m and depth of embedment 1.2m with soil properties ϒ=17.30KN/m^3, ϕ=26 degree, c=0, SPT-N value 18 and borehole depth 30m.

Results

Bearing capacity based on 25 mm settlement

Subject	Terzaghi & Peck	Meyerhof
Hand calculation	134.4 KN/m^2	201.67 KN/m^2
Excel sheet	134 KN/m^2	202 KN/m^2
Percentage Variation	0.3%	0.16%

5.1.8 Example 8

This is the example illustrating the method for calculating **consolidation** settlement of shallow foundation. It is example 5.6 and page number 256 of book "Principal of Foundation Engineering 7th edition by Baraj M Das". Results are given below.

Results

Subject	Primary Consolidation	Secondary Consolidation
Hand Calculation	0.36 mm	0.67 mm
Excel Sheet	0.369 mm	0.66 mm
Percentage Variation	2.5%	1.5%

5.1.9 Example 9

This is the example illustrating the method for calculating bearing capacity of pile based on settlement. It is example 12.3 and page number 659 of book "Principal of Foundation Engineering 7th edition by Baraj M Das". Results are given below.

Results

Subject	Bearing Capacity
Example	2018.5 KN
Excel sheet and Software	1929.56 KN
Percentage variation	4%

5.1.10 Example 10

This is the example illustrating the method for calculating elastic settlement of shallow foundation. It is example 5.6 and page number 256 of book "Principal of Foundation Engineering 7th edition by Baraj M Das". Results are given below.

Results

Subject	Settlement
Example	27 mm
Excel sheet and Software	27.5 mm
Percentage variation	1.8%

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