



ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
ADDIS ABABA INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE
TIPS ON HARD STRATA USING FINITE ELEMENT METHOD: Case of Commercial
Bank of Ethiopia Headquarters Building

By

FISSEHA KIROS

A THESIS SUBMITTED TO ADDIS ABABA UNIVERSITY AS PARTIAL
FULFILLMENT OF MASTER OF SCIENCE IN CIVIL ENGINEERING

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ABSTRACT

Piled raft foundation is a geotechnical composite construction consisting of three elements: piles, raft and soil. In piled raft foundation, the superstructure load is shared between the piles and the raft. Therefore, the piled raft foundation allows an increase in the load capacity and reduction of settlements in a very economical way as compared with traditional foundation concepts.

In this study, the contributions of raft on bearing behaviour of piled raft foundation and on load sharing mechanisms between raft and piles in case of pile tips resting on hard stratum has been conducted. The full interaction between the components of piled raft foundation was accounted by a three-dimensional finite-element method. Commercial Bank of Ethiopia (CBE) head quarter building has been taken as case study. This foundation is primarily designed as conventional pile foundation. The raft; pile and soil have been discretized as eight-noded brick elements. The raft and piles were modelled with a linear elastic constitutive model whereas; the soil continuum was modelled with modified Drucker – Prager cap plasticity constitutive model.

The foundation settlements and the load sharing between raft and pile have been investigated to identify the contribution of raft to the total capacity of piled raft foundations. The results of the analyses show that although the foundation is primarily designed by conventional piled foundation, the raft alone is adequate to carry the total superstructure load with acceptable settlement. Based on the result adding 46 piles below the raft has no contribution for settlement reduction. However, almost the entire superstructure load is carried by the piles; the contribution of the raft became negligible only 7% of the total load is carried by the raft. When this result is compared with simplified method; the hand calculations give marginally higher value. It gives 14.6% of the total load is transfer directly to soil by the raft. A parametric study has been conducted to see the effect of weak layer located under hard stratum below pile tips on piled raft behaviour. Based on the results, it can be concluded that the effect of this weak layer is negligible for relative stiffness between the weak and hard strata is greater than 0.5. In addition; the piled raft behaviour is not affected if the weak layer is located below 9 m for all stiffness ratios (0.05 to 0.8) considered in this study.

Key words: pile; raft; raft - soil interaction; finite element modelling and analysis; weak layer; hard strata; pile tips

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LIST OF SYMBOLS

E	Young's modulus
G	Shear modulus
γ	Unit weight of the soil
k_0	Coefficient of earth pressure at rest
p	Mean stress
q	Deviator stress
ϕ'	Effective friction angle of the soil
c	Cohesion of the soil
ν	Poisson's ratio of the soil
σ	Normal stress
σ_z	Vertical stress
τ	Shear stress
d	Intersection of yield surface F_s with the t-axis
b	Slope of the yield surface F_s in the p-t plane
C_c	Compression index
C_s	Swelling index
μ	Stiffness coefficient
w	Stiffness exponent
p_a	Atmospheric pressure
ϵ_v^p	Volumetric plastic strain
ϵ_{vo}^p	Initial cap position
R	Cap eccentricity
K	Flow stress ratio
P_b	Mean effective yield stress
W	Natural moisture content
β	Slope of the yield surface F_s in the p-t plane
L	Length of the pile
X	Load shared by the raft
α_{pr}	Load shared by the piles
FE	Finite element

CHAPTER ONE

INTRODUCTION

1.1 Background

The function of any foundation is to transmit the load to the soil in order to provide safety, reliability and serviceability to the structure. Designing the foundation system carefully and properly, will surely lead to a safe, efficient and economic project overall. The conventional pile design philosophy is based on that piles carry all the load and they are accepted as a group, no contribution is made by the raft to the ultimate load capacity. However, a combination of the shallow and deep foundation can be a cost effective design approach. The piled raft foundation is such a combination of a deep pile group and a shallow raft foundation, which has gained increasing recognition in very recent years.

In a piled raft foundation, the total load coming from the superstructure is partly carried by piles and the remaining load is taken the by raft through contact with the soil. The design of piled raft is based on the soil-structure interaction between the constituting elements, however, the piled raft subsoil interaction problem is highly complicated as it depends on large number of parameters like pile-raft geometry, pile spacing, subsoil type etc. Hence it needs 3D analysis to account for these factors.

In this study an investigation is made to assess raft contributions on the bearing behaviour of piled raft foundation and on load sharing mechanisms between raft and piles in the case of pile tips resting on a hard stratum. In addition, a parametric study has been conducted to see the effect of weak layer located the under hard stratum below the pile tips on piled raft behaviour. In this study the Commercial Bank of Ethiopia (CBE) head quarter building has been taken as a case study. The complex geometry and interaction of piled-raft foundation can be accounted for using Finite Element Method modelling. So, this method will be used for modelling and analysing the piled raft foundation system. Prior to detail modelling a preliminary assessment of the foundation has been carried out using simplified method. The results of the analyses show that the raft alone is adequate to carry the total superstructure load with an acceptable settlement of 5.1 cm.

Based on the results of the parametric study, the effect of weak layer located under hard stratum below pile tips on the behaviour of piled raft is negligible for relative stiffness

between the weak and hard stratum greater than 0.5. In addition; the piled raft behaviour is not affected if the weak stratum is located below 9 m for all stiffness ratios (0.05 to 0.8) considered in this study.

1.2 Problem statement

Though foundation engineers have long recognized the use of piles along with raft, the contribution of the raft towards the load sharing and settlement is ignored in the conventional design approach. Piles are assumed to carry the total structural load and this leads to a conservative estimation of foundation performance. The Commercial Bank of Ethiopia (CBE) head quarter building is now under construction (2010 E.C). This building is taken as a case study to assess the contribution of the raft on the load sharing and bearing behaviour of the actual piled raft foundation.

1.3 Objectives of the study

The main objective of this study is to investigate the contribution of raft to bearing behaviour of piled raft foundation and to assess load sharing mechanisms between raft and piles in case of pile tips resting on a hard stratum for the case of CBE Headquarter building under construction in Addis Ababa.

Specific objectives

- to assess the load transfer mechanisms which take place within the piled raft foundation system.
- to evaluate the load sharing behaviour between the raft and the piles.
- to investigate the contribution of the raft on the maximum and differential settlements of the foundation.
- to assess the effect of the weak stratum on the failure mechanism of the piled raft when this stratum is located under the hard stratum below the pile tip.

1.4 Methodology

In this thesis, finite element software is used for modelling the piled raft foundation.

To meet the above objectives the following procedure was followed.

- ✓ Data collection

All the necessary secondary data that helps for the modelling and analysis of the piled raft foundation were collected from investigation reports conducted in the lab and field tests.

Different correlation techniques were also used to determine soil parameters that are not included in the soil investigation report.

- ✓ Software modelling and analysis the data

The finite element software **ABAQUS 2013** was selected for simulating the raft, pile and piled rafts foundation. The behaviour of piled raft foundation was analysed based on different constitutive material behaviours. The material behaviour of soil was modelled with non-linear material constitutive model, whereas, the raft and piles were modelled with a linear elastic constitutive models.

- ✓ Finally the results are interpreted, conclusions are drawn and recommendations made based on the obtained results.

1.5 Scope of the study

This study is limited to investigate the contribution of raft, on settlement, load sharing behaviour and bearing capacity of piled raft foundations when the pile tips rest on a hard stratum. The analysis is limited to the case of the Commercial bank of Ethiopia (CBE) head quarter building under gravity loading only. It is also restricted to assess the failure behaviour of piled raft in case where, a weak stratum is located under the hard stratum below the pile tips.

1.6 Thesis Organization

Chapter 2 presents the general background of the use of piled raft foundations to support different types of structures and the advantages of using such foundations. It also highlights the current available means for analysing and designing piled raft foundations. A review of previous studies on piled raft foundations is also presented in this chapter. Chapter 3 presents information about the case study building and its surroundings. It also contains information about geotechnical conditions at the site. The determination of soil parameters used for modelling the foundation is also included in this chapter.

A preliminary assessment of the foundation a using simplified method has been presented in chapter 4. This is to investigate the adequacy of the raft alone, with regard to ultimate bearing capacity and settlement requirements. Chapter 5 presents details of the numerical model. This model was used to analyse the piled raft foundation. In addition, the validity of the developed model with respect to other models available in the literature is also included in this chapter.

Chapter 6 presents the analysis results and discussions of the piled raft foundation. It has been discussed in terms of load-settlement and load sharing-settlement curves. Conclusions and recommendations of the whole work have been summarized in Chapter 7 and References also added at the end of this chapter.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Piled Raft Foundation is a composite foundation system consisting of three bearing elements; piles, raft and sub soil, that combines the bearing effect of both foundation elements raft and piles. Due to combining different element of structures (raft and piles) in one system, piled-raft foundations are regarded as very complex systems. The complexity of this type of foundations is caused by the presence of four interaction factors involved in the system these are pile-to-soil, pile-to-pile, raft - to-soil and pile-to-raft interactions Fig 2.1 (Jean-Louis n.d.).

In conventional foundation design, it has to be shown that either the raft or the piles will support the building load with adequate safety against bearing capacity failure and against loss of overall stability. In piled raft foundation, the contributions of the raft and piles are taken into consideration to verify the ultimate bearing capacity and the serviceability of the overall system. In other words, the role of the raft is to provide the required bearing capacity and the piles are used mainly as settlement reducers but can also contribute to the bearing capacity.

Piled raft foundation provides an economical foundation option for circumstances where the performance of the raft alone does not satisfy the design requirements. Under these situations, the addition of a limited number of piles may improve the ultimate load capacity, the settlement and differential settlement performance, and the required thickness of the raft (H. Poulos 2001). The most effective application of piled rafts occurs when the raft can provide adequate load capacity, but the settlement and/or differential settlements of the raft alone exceed the allowable values. This generally occurs when the near-surface soil profile contains relatively stiff clays or relatively dense sands (H. Poulos 2002).

The characteristic value of the total resistance $R_{total}(s)$ of the piled raft foundation depends on the settlement S of the foundation and consists of the sum of the characteristic pile resistance $\sum_{j=1}^m R_{pile,k,j}(s)$ and the characteristic raft resistance $R_{raft,k}(s)$. The characteristic raft resistance results from the integration of the settlement dependent contact pressure $\sigma(s, x, y)$ in the ground plan area of the raft. i.e

$$R_{total,k} = R_{raft,k} + \sum_{j=1}^m R_{pile,k,j} \geq S_{total} \quad (2.1)$$

Where: R_{total} = total characteristic resistance of the piled raft, R_{raft} = characteristic resistance of raft, $\sum_{j=1}^m R_{pile,k,j}$ = total characteristic resistance of piles and S_{total} = total applied load to piled raft

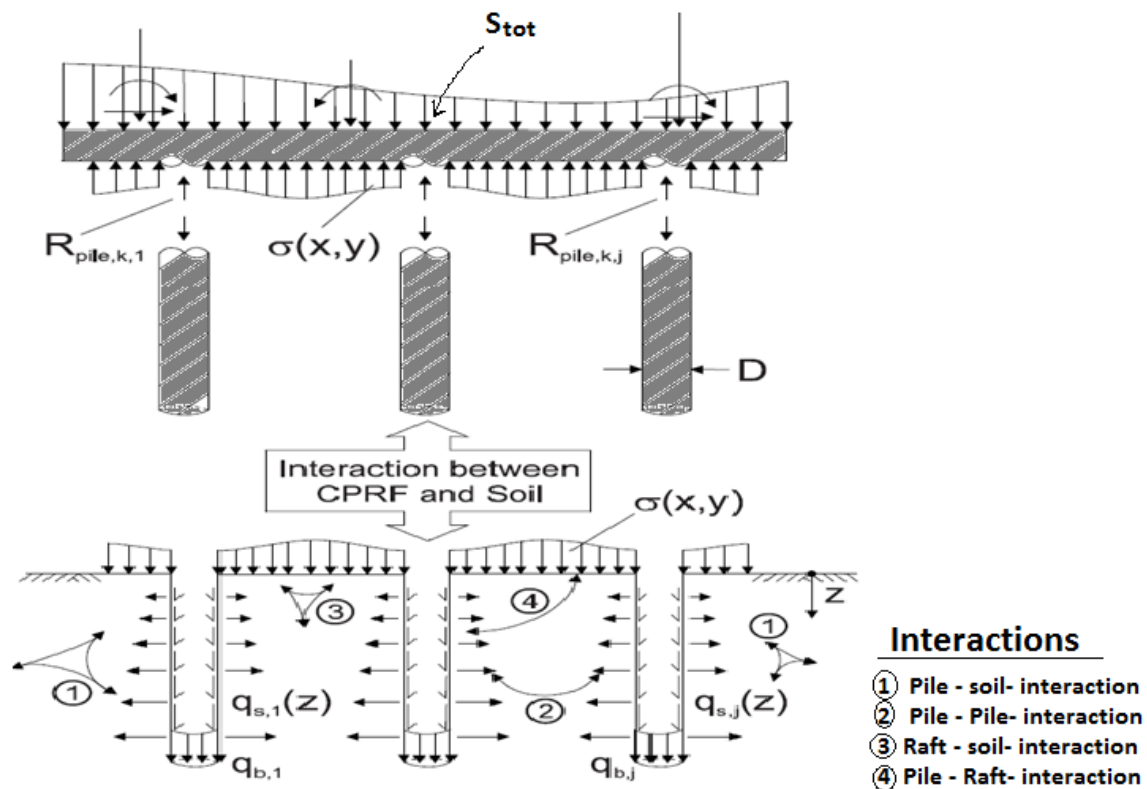


Figure: 2.1 Piled-Raft Foundation and the interactions among the bearing elements (Jean-Louis n.d)

Bearing behaviour of the piled raft foundation is described by pile-raft coefficient α_{pr} which is defined by ratio between the sums of the characteristic pile resistance to the characteristic value of the total resistance

$$\alpha_{pr} = \frac{\sum_{j=1}^m R_{pile,k,j}}{R_{total,k}} \quad (2.2)$$

A piled raft coefficient of $\alpha_{pr} = 0$ represents the case of shallow foundation, and a coefficient of unity ($\alpha_{pr} = 1$) represents the case of a fully piled foundation without contact pressure beneath the raft. Piled raft foundation cover the range $0 < \alpha_{pr} < 1$, whereby conventional shallow and piled foundations are the limiting case of piled raft.

The pile-raft coefficient α_{pr} depends on the stress level and on the settlement of the piled raft.

Figure 2.2 shows a qualitative example of the dependence between the pile-raft coefficient α_{pr} and the settlement of a piled raft S_{pr} related to the settlement of a spread foundation S_{sf} with equal ground plan and equal loading.

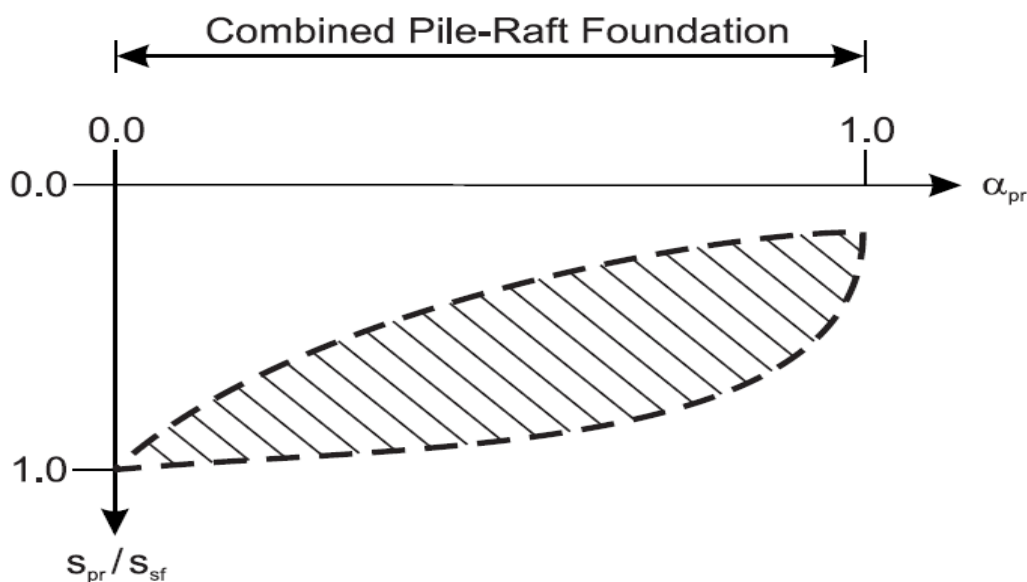


Figure 2.2 Qualitative example of a possible settlement reduction of a CPRF in function of the pile-raft coefficient α_{pr} (Jean-Louis n.d)

2.1.1 Soil – Structure interactions of piled raft foundations

The load bearing behaviour of piled raft foundation is characterised by complex soil – structure interaction between the element of foundation and sub soil. The load bearing behaviour of piles as part of piled draft differs with free standing pile. The ultimate bearing capacity of pile beneath piled raft is greater than as compared with single isolated pile

whereas its stiffness smaller than the stiffness of a single isolated pile. This is due to the effect of pile - pile and pile - raft interactions (Giretti 2009).

Although significant load may be carried by the raft (depending on the group size and pile spacing) which increases shaft resistance of piles due to increasing vertical effective stress, there is a corresponding decrease in load transfer in the upper region of each piles. This softening response of pile group in the upper part is due to over lapping of displacement field of piles and raft. But the two effects compensate, giving only marginally greater stiffness of the overall foundation compared with a pile group with a free-standing pile cap (Basile 1999).

The four interacting mechanism among foundation elements and their surrounding soil that mentioned before, could be classified into two categories (Vakili 2015).

- pile-soil-pile and
- Pile-soil-raft.

The pile-soil-pile interaction is the same as the free standing pile group and is a function of pile spacing and pile installation method. The interaction effect in the pile group design is considered by defining the group efficiency as follows:

$$Q_{gu} = EnQ_u \quad (2.3)$$

Where: Q_{gu} = bearing capacity of pile group, Q_u = bearing capacity of the single piles,
E = group efficiency and n = number of piles.

Pile-soil-raft interaction could have favourable and unfavourable effects on bearing capacity and the settlement of piled rafts, respectively. The pressure between the raft and the soil favourably increases the horizontal stress on the pile shaft and consequently increases the shaft resistance. Whereas, this pressure may induces a negative skin friction on the piles which increases the settlement. The pile-soil-raft interaction mainly controls the load sharing mechanism of piled raft foundations when pile spacing (S) is more than 3.5d (d = pile diameter) (Vakili 2015).

2.1.2 Alternative Design Philosophies

Different design philosophies are developed by different authors for designing of piled raft foundation. Some of the design philosophies are discussed here.

- There are three different design philosophies of piled raft foundation as clearly defined by (H. Poulos 2001).

- The “conventional approach”, in which the piles are designed as a group to carry the major part of the load, while making some allowance for the contribution of the raft, primarily to ultimate load capacity.
- “Creep Piling” in which the piles are designed to operate at a working load at which significant creep starts to occur, typically 70-80% of the ultimate load capacity. Sufficient piles are included to reduce the net contact pressure between the raft and the soil to below the pre consolidation pressure of the soil.
- Differential settlement control, in which the piles are located strategically in order to reduce the differential settlements, rather than to substantially reduce the overall average settlement.

Figure: 2.3 illustrate, conceptually, the load-settlement behaviour of piled rafts designed according to the first two strategies. Curve 0 shows the behaviour of the raft alone. Curve 1 represents the conventional design philosophy, for which the behaviour of the pile-raft system is governed by the pile group behaviour. Curve 2 represents the case of creep piling where the piles operate at a lower factor of safety, but because there are fewer piles, the raft carries more load than for Curve 1. Curve 3 illustrates the strategy of using the piles as settlement reducers, and utilizing the full capacity of the piles at the design load. Therefore, the design depicted by Curve 3 is acceptable and is likely to be considerably more economical than the designs depicted by Curves 1 and 2.

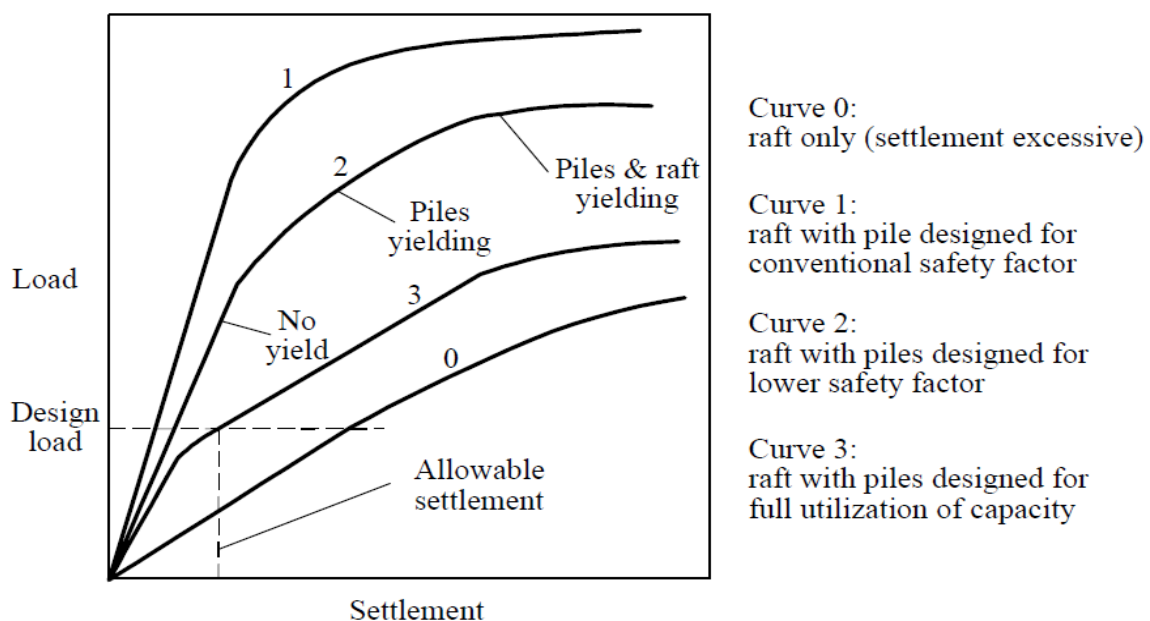


Figure: 2.3 Load settlement curves for piled rafts according to various design philosophies (H. Poulos 2001)

➤ Burland's approach (piles as settlement reducer)

In many cases, the raft alone can provide adequate bearing resistance; however, it may experience excessive settlement. Therefore, the concept of employing piles as settlement reducers was proposed by (Burland 1977). The author proposed a new design philosophy to consider the raft as the main bearing element and to apply the pile group as the settlement reducer. Following this idea, the implemented piles below the raft are designed to operate typically at 70-80% of their ultimate load or even at their full load capacity.

In piled raft foundation system, including piles beneath raft may have two purposes. In one hand to control raft settlement within the allowable value, On the other hand, both to reduce settlement and to improve its bearing capacity. In the case of bearing capacity of raft is sufficient to resist the applied load and the ultimate capacity of the piles may utilize since the main purpose of including them beneath the raft is to reduce settlement. In the case of bearing capacity of raft is not adequate to resist the applied load piles are required not only to reduce settlement the raft but also to improve its bearing capacity, In this case, the ultimate capacity of piles are not used some factor of safety is taken for their ultimate capacities.

➤ On the basis of design requirements, a piled raft foundation can be subdivided in two broad groups: "small" and "large" piled rafts (Viggiani 2012).

1. "Small" piled rafts, where the primary reason for adding the piles is to increase the factor of safety (this typically involves rafts with widths between 5 and 15 m);
2. "Large" piled rafts, whose bearing capacity is sufficient to carry the applied load with a reasonable safety margin, but piles are required to reduce settlement or differential settlement. In such cases, the width of the raft is large in comparison with the length of the piles (typically, the width of the piles exceeds the length of the piles).

These two categories broadly mirror the conventional and creep piling philosophies considered by Randolph.

2.1.3 Design issue

The performance of piled raft foundation depends on different factors since it is composed of piles, raft and soil, its behaviour is mainly depends on these factors. A sound prediction of piled raft foundation behaviour implies a full interaction between piles, raft and soil. Hence, the following factors must be considered, the raft characteristics, piles characteristics, and soil characteristics (Sinha 2013).

As with any foundation system, the design of a piled raft foundation requires the consideration of a number of issues. (H. Poulos 2001) suggested that in order to provide an optimum piled raft foundation design, the following aspects shall be considered and checked:-

- Ultimate load capacity for vertical, lateral and moment loadings
- Maximum settlement
- Differential settlement
- Load shared between piles and raft
- Raft moments and shears for the structural design of the raft
- Pile loads and moments, for the structural design of the piles

2.1.4 Design process

The design of a piled raft foundation has three main stages. Preliminary stage, Second stage and detailed design stage (H. Poulos 2001). These three design processes are discussed as follows:-

a) Preliminary stage to assess the feasibility of a piled raft foundation, and to assess the required number of piles to satisfy design requirements; in this stage the performance of raft without piles is assessed under uniformly distributed load over the raft to estimate vertical and lateral bearing capacity, settlement and differential settlement may be made via conventional techniques. If the raft alone provides only a small proportion of the required load capacity, then it is likely that the foundation will need to be designed with the conventional philosophy in which the piles are designed to carry most of the load. If however the raft alone has adequate or nearly adequate load capacity, but does not satisfy the settlement or differential settlement criteria, then it may be feasible to consider the use of piles as settlement reducers.

b) Second stage to assess where piles are required and the general characteristics of the piles. If the raft alone adequate or nearly adequate capacity to carry the total load but does not satisfy the serviceability limit state from the Preliminary stage; the performance of raft without piles is evaluated in this stage under the actual load pattern to identify where pile is require. Piles may provide below a column upon the following conditions:

- If the maximum moment in the raft below the column exceeds the allowable value for the raft.

- If the maximum shear in the raft below the column exceeds the allowable value for the raft.
- If the maximum contact pressure below the raft exceeds the allowable design value for the soil.
- If the local settlement below the column exceeds the allowable value.

c) **Final detailed design stage** to obtain the optimum number, location and configuration of the piles, and to compute the detailed distributions of settlement, bending moment and shear in the raft, and the pile loads and moments.

2.2 Methods of analysis

The various methods that have been developed for analysing piled raft can be ranged from simplified calculation to more rigorous computer based method. All the analysis methods can be classified into three broad categories: Simplified methods, approximate methods and more rigorous methods.

Simplified methods involve the hand calculation using the theoretical solution for a raft and for a pile in elastic continuum. Simplified methods include those of **Poulos and Davis, Randolph, Burland**. All these methods involve a number of simplifications while considering the soil domain and applying load on the raft.

The approximate computer-based methods are based on elastic theory and mainly have two approaches (H. Poulos 2001). Strip on Springs Approach (GASP) and Plate on Springs Approach (GARP). In the first approach, the raft is represented by a strip and the supporting piles by springs. The analysis is carried out by taking some allowance of interaction factors to obtain the settlements and moments due to the applied loading on that strip section. In the second approach, the raft is represented by an elastic plate, the soil is represented by an elastic continuum and the piles are modelled as interacting springs.

In more rigorous computer-based methods, the interactions of piled raft components are accounted by modelling the actual problem in computer program. The analysis of piled raft foundation by rigorous method, one may use either of the following alternatives (Sinha 2013).

1. Numerical solution, combination of numerical and analytical,
2. Commercially available software based on numerical technique and

3. Mixed technique based on the finite element method (FEM) and boundary element method (BEM), where FEM is used to model raft and boundary element method is used to model pile-soil and raft-soil.

Poulos - Devis - Randolph (PDR) method from simplified methods and the second option, commercially available software based on numerical technique, from more rigorous methods will be discussed here in detail because both methods will be used for preliminary and detail design stage of piled raft foundation in this study.

2.2.1 Simplified methods

a) Poulos- Devis –Randolph (PDR) method

PDR method is the combination of both Poulos-Devis and Randolph methods. This method is used for assessing the overall bearing capacity and load settlement behaviour of piled raft in the preliminary design stage. There are two phases in the preliminary stage of piled raft foundation as described by (H. Poulos 2002). In the first phase of preliminary design stage the performance of raft foundation without piles is assessed in terms of its bearing capacity and average settlement using conventional approach. This process helps to identify a proper design philosophy and to estimate the number piles which satisfy the requirements.

In the second phase, the performance of raft under column loading is evaluated to decide where these piles should be located.

For assessing vertical bearing capacity of a piled raft foundation using simple approaches, the ultimate load capacity can generally be taken as the lesser of the following two values (H. Poulos 2002).

- a. The sum of the ultimate capacities of the raft plus all the piles
- b. The ultimate capacity of a block containing the piles and the raft, plus that of the portion of the raft outside the periphery of the piles.

The required numbers of piles which satisfy the overall bearing capacity and the allowable average settlement of the foundation can be estimated by developing load settlement curve of the foundation. The curve is depend on the stiffness of individual foundation component

$$K_{pr} = \frac{K_p + K_r (1 - \alpha_{cp})}{\left(1 - \frac{\alpha_{cp}^2 K_r}{K_p}\right)} \quad (2.4)$$

Where: K_{pr} = stiffness of piled raft, K_p = stiffness of the pile group

K_r = stiffness of the raft alone, α_{cp} = raft – pile interaction factor.

The stiffness of the raft (K_r) is computed by hand using (Eq.2.5a) using elastic theory of (Mayne 1999). This theory assumes the raft to be an equivalent circular footing, and considering the centre of a flexible raft.

$$K_r = \frac{E_s d}{(1-\nu^2) I_F I_E} \quad (2.5a)$$

Where: K_r = Raft stiffness, d = equivalent diameter of the raft, ν = Poisson's ratio of the soil
 E_s = Equivalent elastic modulus of soil, E_r = Elastic modulus of the raft, t = thickness of the raft, Z_E = depth of the foundation, I_F = correction factor for rigidity, I_E = correction factor for embedment

$$I_E = 1 - \frac{1}{3.5 \exp(1.22\nu - 0.4) \left[\frac{d}{Z_e} + 1.6 \right]} \quad (2.5b)$$

$$I_F = \frac{\pi}{4} + \frac{1}{4.66 + 10 * K_F} \quad (2.5c)$$

$$K_F = \frac{E_r}{E_s} * \left(\frac{2t}{d} \right)^3 \quad (2.5d)$$

The stiffness of pile group (K_{pg}) can be estimated first by computing single pile stiffness using (Eq.2.6) from approximate solutions of (H. Poulos 2001), then multiplying this stiffness by group efficiency ($R = n^{1-e}$), where n = the number of piles, e = efficiency exponent, typically in the range 0.2-0.6, but varying with spacing.

$$K_p = G_s r_o \frac{\frac{4\eta}{(1-\nu)\xi} + \frac{2\pi L_p \tanh \mu L_p}{\zeta a \mu L_p}}{1 + \frac{1}{\pi\lambda} \frac{4\eta}{(1-\nu)\xi} \frac{L_p \tanh \mu L_p}{a \mu L_p}} \quad (2.6)$$

Where: G_s = soil shear modulus at the depth L_p , L_p = pile length, r_o = pile radius, $\eta = r_b / r_o$, r_b = pile radius at its base, $\zeta = G_s / G_b$, G_b = shear modulus of the soil below the pile base, $\rho = G_{1/2} / G$ = medium shear modulus at the soil layer around the pile, $\lambda = E_p / G_s$ = pile – stiffness ratio, E_p = elastic modulus of pile material, $\zeta = \ln(r_m / r_o)$, $r_m = L_p \{0.25 + \xi[2.5\rho(1-\nu) - 0.25]\}$ = maximum radius of influence, $\mu L_p = \frac{L_p}{r_o} \sqrt{\frac{2}{\zeta\lambda}}$.

The raft – pile interaction α_{cp} can be estimated by equation (2.7)

$$\alpha_{cp} = 1 - \frac{\ln\left(\frac{r_c}{r_o}\right)}{\zeta} \quad (2.7)$$

where r_c = average radius of raft (corresponding to an area equal to the raft area divided by number of piles); r_o = radius of pile; $\zeta = \ln(r_m / r_o)$, $r_m = L_p \{0.25 + \xi[2.5\rho(1-\nu) - 0.25]\}$; $\xi = E_s / E_b$, $\rho = E_{sav} / E_{sb}$; ν = Poisson's ratio; L = pile length; E_s = Soil young's modulus at level of pile tip; E_{sb} = Soil young's modulus of bearing stratum below pile tip; E_{sav} = Average young's modulus along pile shaft as shown in figure 2.4a.

Randolph reported that the raft-pile interaction factor has a tendency to be equal to 0.8 for large group of piles (Sinha 2013). Therefore, the overall stiffness may be simplified to (Eq.2.8).

$$K_{pr} = \frac{1 - 0.6\left(\frac{K_r}{K_p}\right)}{1 - 0.6\left(\frac{K_r}{K_p}\right)} K_p \quad (2.8)$$

The proportion of the total applied load carried by the raft is:

$$\frac{P_r}{P_t} = \frac{0.2}{1 - 0.8\left(\frac{K_r}{K_p}\right)} \frac{K_r}{K_p} = X \quad (2.9)$$

Equation 2.8 is used to develop a tri-linear load-settlement curve of the piled raft system as described by (Katzenbach 2013). First the stiffness of the piled raft foundation (Randolph approach Figure 2.4a) is computed for number of piles being considered. This stiffness will remain operative until the pile capacity is fully mobilized. Total applied load, P_A , at which the pile capacity is reached, is given by:

$$P_A = \frac{P_u}{\alpha_{pr}} \quad (2.10)$$

Where: P_u = the ultimate load capacity of the pile group

α_{pr} = proportion of load carried by the pile group, $\alpha_{pr} = 1 - X$

X = proportion of load carried by the raft (Eq. 2.9)

Beyond that point (Point A in Figure 2.4), the stiffness of the foundation system is that of the raft alone and this holds until the ultimate load capacity of the piled raft foundation system is reached (Point B in Figure 2.4). At this stage, the load-settlement relationship becomes horizontal.

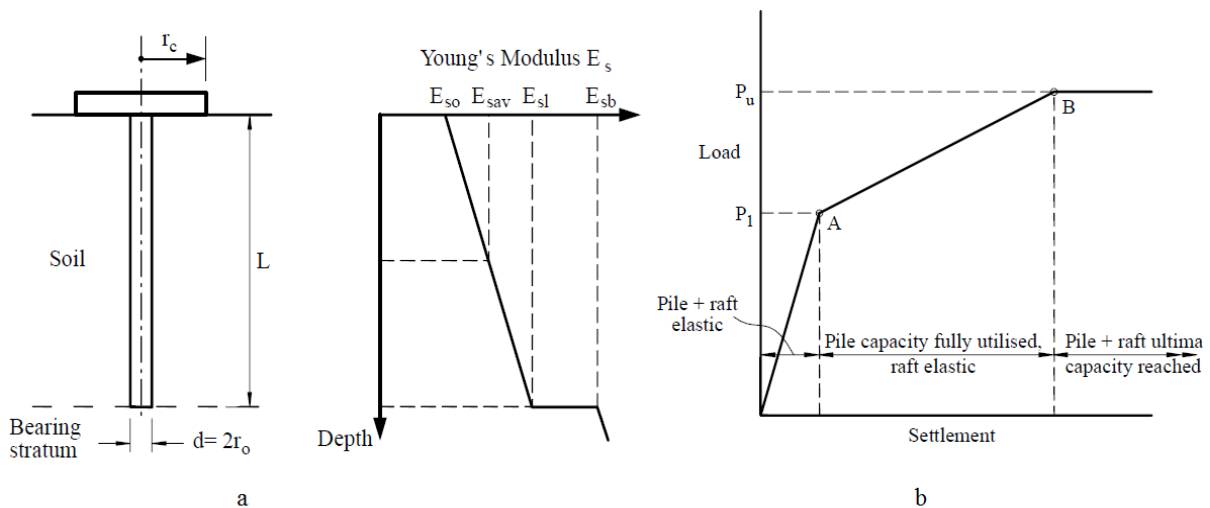


Figure: 2.4 a) Piled raft - unit Randolph approach b) Simplified load settlement curve (H. Poulos 2002)

b) Burland's approach

Burland has developed some simplified process of design when piles are designed to act as settlement reducer and to develop their full capacity at the design load (Burland 1977).

- I. Estimate the total long-term load-settlement relationship for the raft without piles (Figure 2.5 a). The design load P_o gives a total settlement S_o .
- II. Assess an acceptable design settlement S_a , which should include a margin of safety.
- III. P_1 is the load carried by the raft corresponding to S_a .
- IV. The load excess $P_o - P_1$ is assumed to be carried by settlement-reducing piles. The shaft resistance of these piles will be fully mobilized and therefore no factor of safety is applied. However, Burland suggests that a “mobilization factor” of about 0.9 be applied to the ‘conservative best estimate’ of ultimate shaft capacity P_{su} .
- V. If the piles are located below columns which carry load in excess of P_{su} , the piled raft may be analysed as a raft (Figure 2.5b) on which reduced column loads act. At such columns, the reduced load Q_r is $Q_r = Q - 0.9 P_{su}$.
- VI. The bending moment in the raft can be obtained by analysing the piled raft as a raft subjected to the reduced loads Q_r .

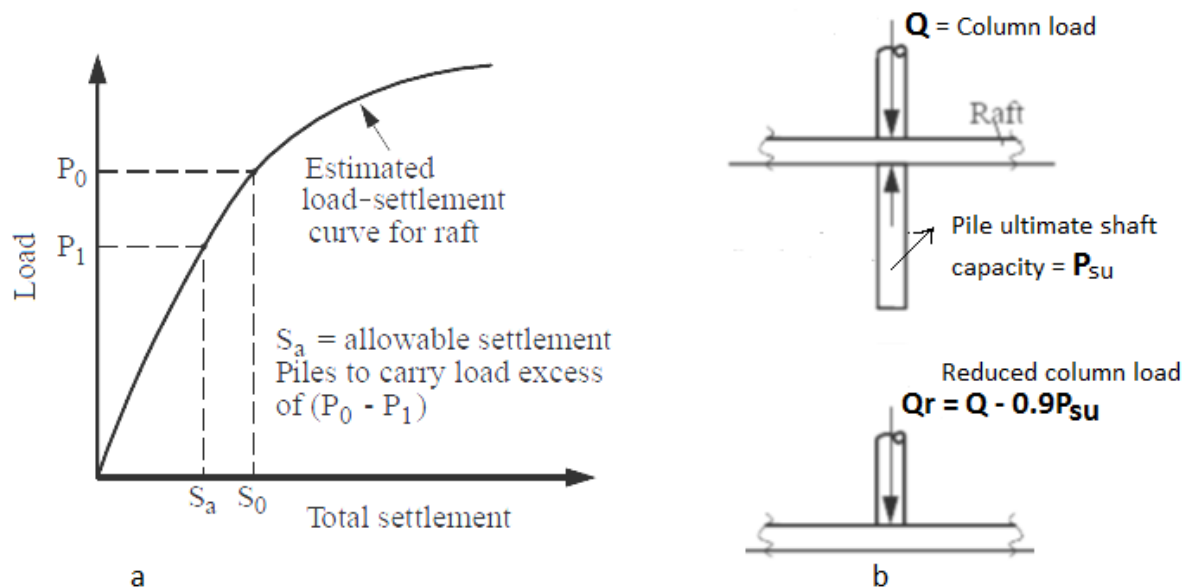


Figure: 2.5 a) Load settlement curve for raft b) Simplified representation of pile – raft unit (Burland 1977).

2.2.2 More rigorous methods

In the preliminary design stage only bearing capacity and average settlement of rafts are considered as discussed in section (2.2). This helps to decide the number of piles required and where these piles should be located, but other issues such as differential settlement, raft shear force

and bending moment, distribution of load among the piles etc. are not considered. So it is necessary to carry out a more detailed design in order to assess the detailed distribution of settlement and decide upon the optimum locations and arrangement of the piles. The raft bending moments and shears, and the pile loads, should also be obtained for the structural design of the foundation.

The numerical methods employed to simulate the complex piled raft foundation are mainly the Finite Element Method (FEM), Boundary Element Method (BEM), Finite Difference Method (FDM) or a combination of two or more of these methods.

2.3 Related studies

The performance of piled raft foundation is affected by many factors. These factors occur as result of soil, pile and raft characteristics. Many researchers have examined the performance of piled rafts considering the effects of soil type, pile dimensions and configurations, and raft dimensions on its load sharing and load settlement behaviour. From literature, the most parameters that affect the Load sharing and settlement behaviour of piled rafts are relative stiffness among the foundation components. The relative stiffness of the foundation also depends on stiffness of the soil, dimension and configuration of the foundation components. Therefore some of the previous works that are related to this study are reviewed as follows.

The effect of various parameters related to pile and raft on the normalized settlement of piled raft adopting two dimensional plane strain model has studied by (Prakoso 2001) (Figure 2.6).

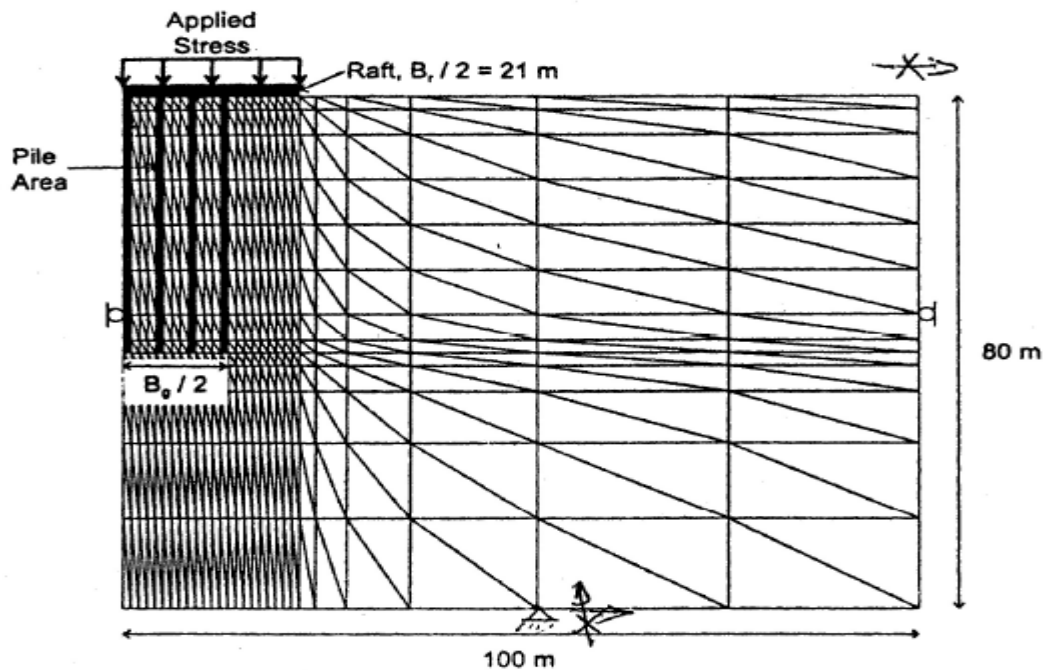


Figure 2.6 Plane Strain idealization of piled raft (Prakoso 2001).

The modulus of pile wall was computed from the term equivalent modulus which was a function of number of piles, width or diameter of the pile and soil modulus. The study concluded that the ratio between the width of the raft and the length of the pile played an important role on settlement behavior of piled raft. The piled raft with ratio equal to unity was very effective in reducing overall settlement, where as a ratio 0.5 was very effective in minimizing the differential settlement. Further it was concluded that a pile to raft area ratio of 5% to 6% was adequate to reduce the overall settlement. The results were mostly in the form of non-dimensional parameters. While the contribution was very useful as a parametric study, it has only a very limited application. The procedure would be ideal for a single group of large number of piles, in a row (Prakoso 2001).

An experimental study was carried out by (Giretti 2009) to understand load sharing mechanism of piled raft foundation and he found that the load sharing mechanism between piles and raft is depend on pile – subsoil stiffness. According to his finding, at initial stage the applied load is taken by the piles since piles are stiffer than soil, then when the piles reached their ultimate load capacity the excess load is carried by the raft.

A three dimension finite element analysis has performed by (Sinha 2013) on sand to investigate load sharing behaviour between the piles and raft and he concludes that soil property, raft geometry and pile geometry (diameter, length, spacing of piles) are some of the

parameters that affect load sharing behaviour of piled raft foundation. Piles take more loads at small spacing and raft take more load at large spacing of piles but pile spacing greater than $7D$ is ineffective.

A finite element analysis of piled raft foundation in sand under vertical load have been carried out by (Omeman 2013) to study the load sharing behaviour between raft and piles by considering different configuration of piles beneath the raft and he suggest that the load sharing mechanism between piles and raft in piled raft foundation is depend on the ratio stiffness of the pile group to stiffness of the raft. In addition to this, settlement of the foundations plays an important role in distributing the load between the foundations elements (e.g. raft and the piles). For example, the load carried by the raft can increase with the increase in the settlement of any particular piled-raft system. Therefore, predicting the settlement of piled-raft foundations is the first step to determine the load sharing between the piles and the raft.

A series of 2D numerical analyses piled raft foundation have been performed by (Ningombam 2013). The effects of raft thickness, soil modulus and pile spacing on load sharing of piled rafts were studied through this numerical study. The result reveals that the percentage of load shared by the raft increases with increasing pile spacing and soil modulus and decreasing with increasing raft thickness. The thinner the raft the more deflect that increase contact between raft and soil which increases the load shared by the raft. In soft soil, whatever load comes to the raft is taking care by the pile through skin friction and alternatively, if the soil is hard soil, whatever load comes to the raft is taking care by the underneath raft soil.

A parametric study have been carried out by (Sinha 2013) on the performance of piled raft considering different parameters using finite element method. The result reveals that raft settlement increases with the increase in pile spacing and decreases with the increase in the pile size and length whereas the settlement of the raft decreases with the increase of the angle of shearing resistance and the cohesion of the soil. For pile spacing greater than 6 times the pile size, the system tends to function as a raft foundation

A number of case studies have been carried out by (Khalid 2016) to investigate the effect of piled raft area ratio on load sharing mechanism of piled raft foundation using finite element method. The author concludes that the ratio of total piles area to raft area has important

influence on load sharing of piled raft component. The piles load sharing varies from (15%) at piles-raft ratio of about (1 %) corresponding to pile spacing of (7D) to (42%) at piles/raft ratio equals to (7%) corresponding to a closely pile spacing of (3D) which means that the piles are taking more loading share and there is a tendency to behave as a mere piled system as the spacing between piles is reduced.

A detailed 3D analysis on piled raft foundation have been conducted by (Aluniam 2013) to examine the effect of raft flexibility, width of raft and diameter of piles on load sharing behaviour of piles and raft using finite element method. The result showed that the percentage of load shared by the piles increase as pile diameter increase and decreases as raft width increase. But raft flexibility has significantly affected the load shared by piles and raft. Flexibility of raft is depending on raft thickness and pile spacing. The variation in load carried by the pile is noticeable at $S/D=4$; the load carried by the piles is about 35% and 45% for raft thickness, $t=0.3$ m and $t=2$ m, respectively. The raft flexibility, $K_f=0.2$ and 8.73 for these two cases.

On the other hand, $K_f=0.29$ and 0.02 if the spacing is $S/D=10$ for the same raft thickness values. Due to the narrow range of K_f for the large spacing case, the variation in percentage of load carried by the piles is insignificant, and it is approximately 25%. Therefore, from this result one may notice that piles spacing have a great influence on load sharing behaviour of piled raft foundation.

A parametric study has been carried out by (Der-Guey 2016) on the load sharing behaviour of piled raft considering different parameters using finite different method. The parametric study results show that the load- carrying ratio of the raft depends on the number of piles of the pile group and the level of loading applied to the piled-raft foundation (the raft may carry 15% to 70% of the total applied load to piled raft).The load carried by the raft increasing when the loading to piled raft increases and the number of piles decreases.

CHAPTER THREE

SITE CHARACTERISATION AND SOIL PROPERTIES

3.1 General description of the case study

The New Headquarter Building Commercial Bank of Ethiopia (CBE) project is located in central Addis Ababa, Kirkos sub city, around the national theatre. The building complex consists of mainly office buildings, including 1 tower building of 46 floors, 2 podiums of 5 floors (business centre and conference centre) and 1 underground parking. The height of the tower is 186.4 m. Foundation of the main building (tower) is composed of pile groups connected together with raft slab. The foundation is founded at the depth of 21 m under the ground surface. The estimated load on the foundation is **960 MN**.

The foundation system consists of 46 bored piles connected together with a raft thickness of 3m. Length of the bored piles is ranged from 5 to 10 m with two different pile diameters. 12 piles have a diameter of $D = 1.8$ m and 34 piles have a diameter $D = 2.2$ m. The load coming from superstructure is transferred to raft through columns (16 in number) and shear wall. Location of these columns is along the periphery of the raft and they coincided with the peripheral bored piles, whereas the shear wall is located around the centre of the raft. Location of the site and geometry of the raft with the configuration of 46 piles and the shear wall is shown in Figure 3.1.

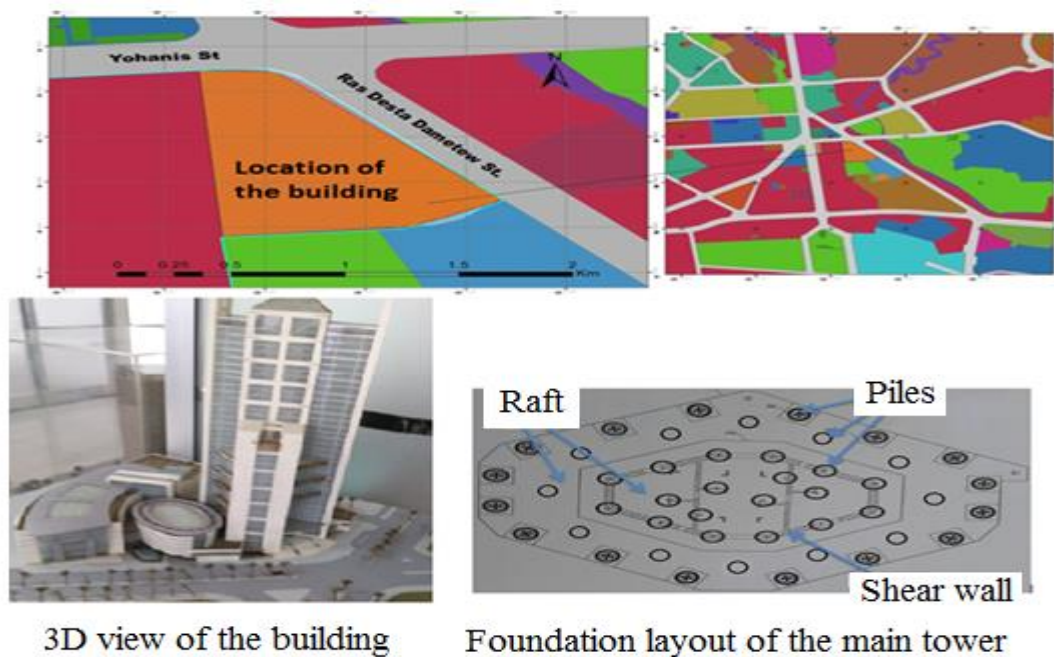


Figure 3.1: Layout of the main tower foundation

3.1.2 Geological conditions of the site

Subsurface profile of the site and the engineering properties of the soil strata are investigated by Construction Design Share Company. The company carried out Core drilling of ten (10) boreholes with a maximum drilling depth of 90m along with in situ and laboratory tests. The following geological layers have been identified by the company.

Table 3.1 Description of soil layers

Soil layers	Description
A	Loose to medium dense, poorly grade fill material it is mainly composed of various proportion of demolished material (concrete, brick), sand silt and clay.
B	Soft, dark brown to grey , organic, silty clay
C	Medium dense, yellowish brown silty sand
D	Weak, grey, highly weathered Basalt. Occasionally, fresh or discoloured rock is present
E	Medium strong, grey, intensively fractured and fragmented, slightly to moderately weathered Basalt. Fractured surfaces are rough and yellowish stained. Joints are nearly horizontal and partially filled with altered material.
F	Very weak, pinkish to grownish, highly to completely weathered scoiraceous basalt. It is localy decomposed into silty clay. This layer is considered as an aquifer
F'	Weak to very weak, pinkish grey, highly to completely weathered scoiraceous basalt. It is often mixed with dark grey swelling silty clay(Fault Gourge).
G	Strong, grey, slightly fractured, fresh to faintly weathered Basalt. Fracture surfaces are slightly rough, mostly sub vertical to vertical and often filled with secondary material(Calcite)
H	Medium strong, light grey, moderately fractured, slightly, weathered trachy Basalt
I	Stiff, dark to light grey , swelling clayey SILT(Altered Ash)
R	Medium dense, brick red, sandy clayey SILT with some gravel

3.1.1 Location of ground water table

In all bore holes, the ground water level has been found averagely 5.5 m below the natural ground level. Since the foundation is located 21 m below the ground surface, submerged unit weight of the soil (γ') was used in the analysis.

The subsurface profile from Borehole numbers (BH6, BH7 and BH8) are used to determine the design parameters for the foundation since the foundation of the main tower is at the location of these boreholes. The soil layers below the foundation of main tower are simplified from Borehole numbers (BH6, BH7 and BH8) as shown in Figure 3.2. Detail information of the subsurface profile of these boreholes is attached in appendix.

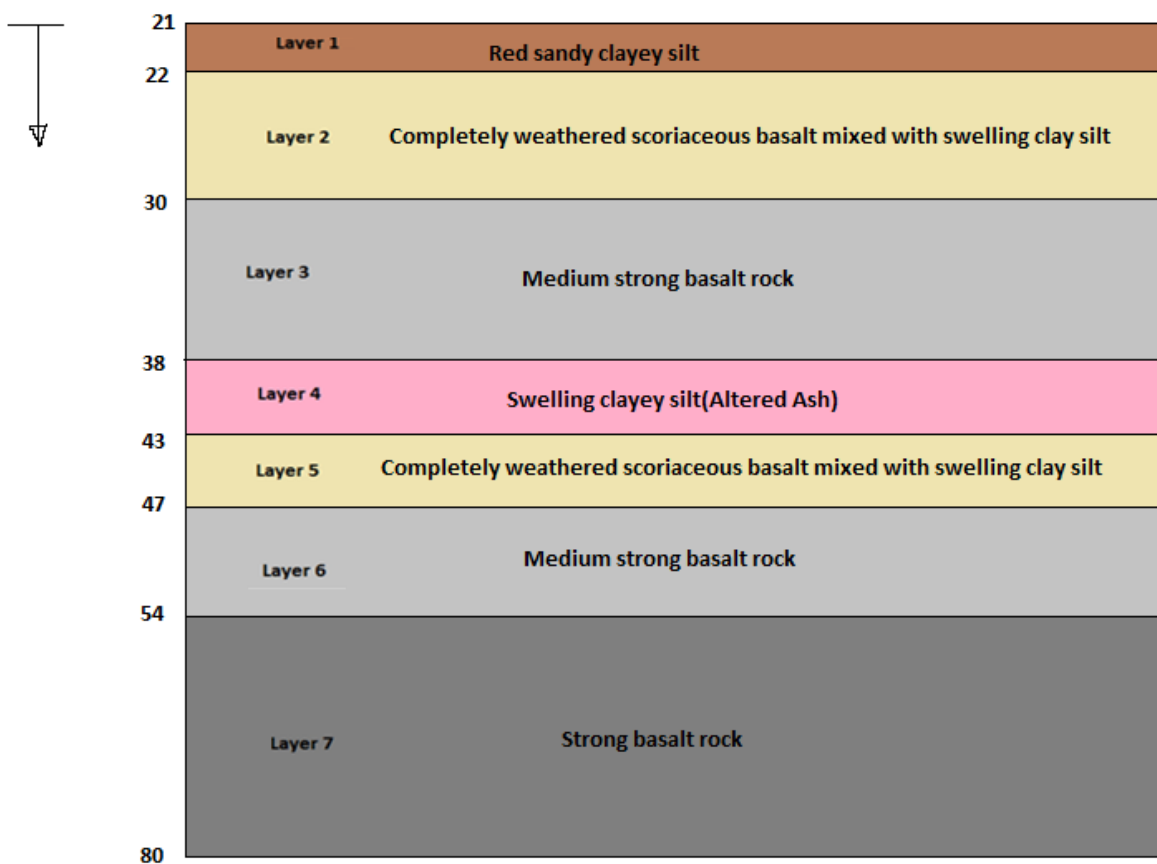


Figure 3.2: Soil profile below the foundation level

As shown in Figure 3.2 there are seven layers of soil strata below the foundation of the main tower level (bottom surface of the raft). The soil layers above the foundation level is removed for the construction 4 basement floors. At the foundation level there is a thin layer of sandy clayey silt with a thickness of 1 m. The next layer is completely weathered basalt mixed with swelling clay. This layer is located from 22 m to 30 m and from 43 m to 47 m. The completely weathered basalt mixed with swelling clay silt is underlain by a layer of medium strong basalt similarly this layer is found from 30 m to 38 m and from 47 m to 54 m. Under

the medium strong basalt layer, a weak strata (swelling clayey silt or altered ash) is encountered with the thickness of about 5 m. The last layer is strong basalt which is extended below the medium strong layer.

3.2 Determination of basic soil parameter for the numerical model

Almost all of the soil parameters used for the analysis of the foundation is obtained from geotechnical reports; however some necessary data are not included in the investigation report.

Therefore the parameters which could not be obtained directly from the laboratory tests, have been indirectly derived by using empirical correlations based on the recommendations of different literature, international geotechnical codes and standards. The data used for this study can be categorized in the following group;

- 1) Stiffness parameters; modulus of elasticity (E_s) and Poisson's ratio (ν)
- 2) Shear strength parameters; angle of internal friction(ϕ) and soil cohesion(c)
- 3) Unit weight or density of the soil(γ)
- 4) Consolidation parameters

1) Stiffness parameters

a) Stiffness Modulus (E_s)

The stiffness modulus E_s is a basic parameter that describes the load-settlement behaviour of soils and governs the results of settlement related problems.

Several methods are available for estimating the stiffness modulus of a soil as described by (Bowles 1996). Unconfined compression tests, tri-axial compression tests and in situ tests are among the test methods. The geotechnical investigation report has included stiffness modulus of values for the different layers but some of them are not conducted. The elastic modulus of three layers; red sandy clayey silt (Layer 1), swelling clayey silt (layer4) and completely weathered rock (layer 2&5) were not given in the soil investigation report. Therefore, elastic moduli of layers (1& 2) have been determined using German code (Eq.3.1) (DIN4094-1 2002). Elastic modulus of the completely weathered rock (layer 2&5) is taken from literature recommendations given by (Gu 2008). After the modulus elasticity (E_s) is determined from correlation then, it is converted to E using (Eq.3.2) which is used as input value in ABAQUS software. The result is summarized in Table 3.2.

Using code provisions: German code (DIN 4094-1(2002) and DIN 4094-2)

$$E_s = \mu P_a \left(\frac{\sigma_z + 0.5 \Delta \sigma_z}{P_a} \right)^w \quad (3.1)$$

Where: μ = stiffness coefficient depends on value of N- SPT

w = stiffness exponent depends on soil type

σ_z = overburden pressure at a depth z below the foundation level

$\Delta \sigma_z$ = additional vertical stress due to load from the superstructure at a depth z

P_a = atmospheric pressure taken as 101.4 kN/m² according to Hayward and

Oguntoyinbo (1987)

$$E = E_s \left(\frac{1 - \nu - 2\nu^2}{1 - \nu} \right) \quad (3.2)$$

b) Poisson's Ratio, ν

Poisson's ratio is property of materials that describes volume change of the material in a direction perpendicular to application of a load. It is defined as the ratio of the lateral expansion to the axial compression of soils. [36] (Bowles 1996) recommend a range of values of Poisson's ratio between 0.4 and 0.5 for most clay and 0.2 to 0.4 for medium to dense cohesion less soil. In this research, the value of poisson's ratio used for the different layers are given in Table 32.

Table 3.2: Modulus stiffness determination according to German code

	Layer1	Layer2	Layer3	Layer4	Layer5	Layer6	Layer7
SPT(N)	28	44	>50	28	44	>50	>50
Depth to its centre(m)	0.5	5	-	19.5	22	-	-
Poisson's ratio(ν)	0.3	0.3	0.25	0.3	0.3	0.25	0.25
Stiffness coef. (μ)	450	450		450	450	-	-
Exponent coef.(w)	0.6	0.6	-	0.6	0.6	-	-
Over burden pressure (σ_z) in kN/m ²	153.5	-	-	384.25	-	-	-
Additional vertical stress ($\Delta \sigma_z$) in kN/m ²	990	-	-	710	-	-	-
Stiffness modulus(E_s) in MN/m ²	138.92	-	-	150.28	-	-	-
Young's modulus (E) in MN/m ²	121.061	400	1000	130.95	400	1000	1500

Layer1[Red sandy clayey silt], **Layer 3&6** [Medium strong rock basalt], **Layer 7** [Strong rock basal], **Layer 2&5**[Completely weathered basalt mixed with swelling clayey silt], **Layer 4** [Swelling clayey silt or Altered Ash

2. Shear Strength Parameters

Shear strength parameters, namely, angle of internal friction (ϕ') and cohesion (c') are taken from literature recommendation given by (Braiuđ 2013). The shear parameters of the selling clayey silt was taken for layers 2 and 5 (completely weathered basalt mixing with swelling clayey silt), by assuming the worst case layer

3. Unit weight or density of the soil

The Soil density is another soil property that the software takes as in put data. Therefore the total unit weight of all layers was taken from the investigation report and given in Table3.3.

Table3.3: Shear strength parameters of the soil

	Frictional angle	Cohesion	Total unit weight
Sym.	ϕ [°]	c[kN/m ²]	[kN/m ³]
Red sandy clayey silt[layer1]	27	25	17
Completely weathered basalt mixed with swelling clayey sit [Layer 2]	42.8	25	22
Medium strong rock basalt [Layer 3]	45	5000	24
Swelling clayey silt(Altered Ash) [Layer 4]	27	25	17.7
Completely weathered basalt mixed with swelling clayey silt [Layer 5]	42.8	25	22
Medium strong rock basalt [Layer 6]	45	5000	24
Strong rock basalt [Layer 7]	45	5000	28

4. Consolidation parameter

Consolidation parameters that required for this study are compression index (C_c) and swelling index (C_r) since these parameters are required to determine cap hardening in cap plasticity constitutive model. The soil properties of some layers were not included in the soil investigation report made by CDSC. Therefore their properties (C_c and C_r) were determined using empirical correlations. There are different correlation methods are available in literature for determining these parameters. C_c is related with liquid limit (LL) by (Arora 1987). The value of C_c and C_r are also given in range. In this study these parameters are taken from the range of specified by (Budhu 2011). These values are given in Table 3.4 shown below.

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD

Table 3.4: Consolidation parameters of the soil

Soil strata	Compression index(C_c)	Swelling index(C_s)
Red sandy clayey silt(layer1)	0.2	0.028
Completely weathered scoraceous basalt. It is locally decomposed into silty clay (Layer 2 and 5)	0.15	0.022
Medium strong basalt rock(Layer 3 and 6)	0.1	0.015
Swelling clayey SILT or Altered Ash(Layer 4)	0.2	0.03
Strong basalt rock(Layer 7)	0.1	0.015

CHAPTER FOUR

PRELIMINARY DESIGN OF PILED RAFT USING SIMPLIFIED METHOD

4.1 Introduction

Prior to the detailed geotechnical design, a feasibility assessment is necessary by considering various foundation schemes. This is to investigate the adequacy of the raft alone, both in regard to ultimate bearing capacity and settlement. If the raft alone is not adequate, the number of piles required below it which satisfy the design requirements will be determined using simplified method.

A geotechnical assessment was carried out for the following foundation schemes:

- A raft alone, without piles;
- A raft with 15 piles;
- A raft with 30 piles;
- A raft with 45 piles
- A raft with 60 piles

For the purposes of the preliminary assessment, the piles were assumed to be 2m diameter and extending to the medium strong basalt with an average length of about 8m. The raft was converted approximately to a rectangular plate 30m by 46m in plan, and 3m in thickness.

The raft capacity is estimated using EBCS-7 and it can carry a maximum pressure of 2.16MPa. This indicates that the total superstructure load can be carried by the raft alone with average of settlement of 6.86 cm. The capacity of single pile is determined using α method for its shaft and its end resistance is taken from compressive test result. This gives a total capacity of 11.54MN.

Settlement and load sharing between the piles and the raft is also investigated using the stiffness of the piled raft, raft and piles. The result shows that the raft can carry 24.11% and 14.37% of the total load for 15 and 60 numbers of piles respectively.

4.2 Assessment for bearing capacity of the raft and piles

a) Raft bearing capacity

The adequacy of the raft alone in terms of bearing capacity and settlement should be investigated first before adapting piled raft system. As mentioned above, the actual geometry of the raft is simplified to an equivalent rectangle (30m by 46m). The soil profile and the location of the raft and piles are given in Figure 4.1 below.

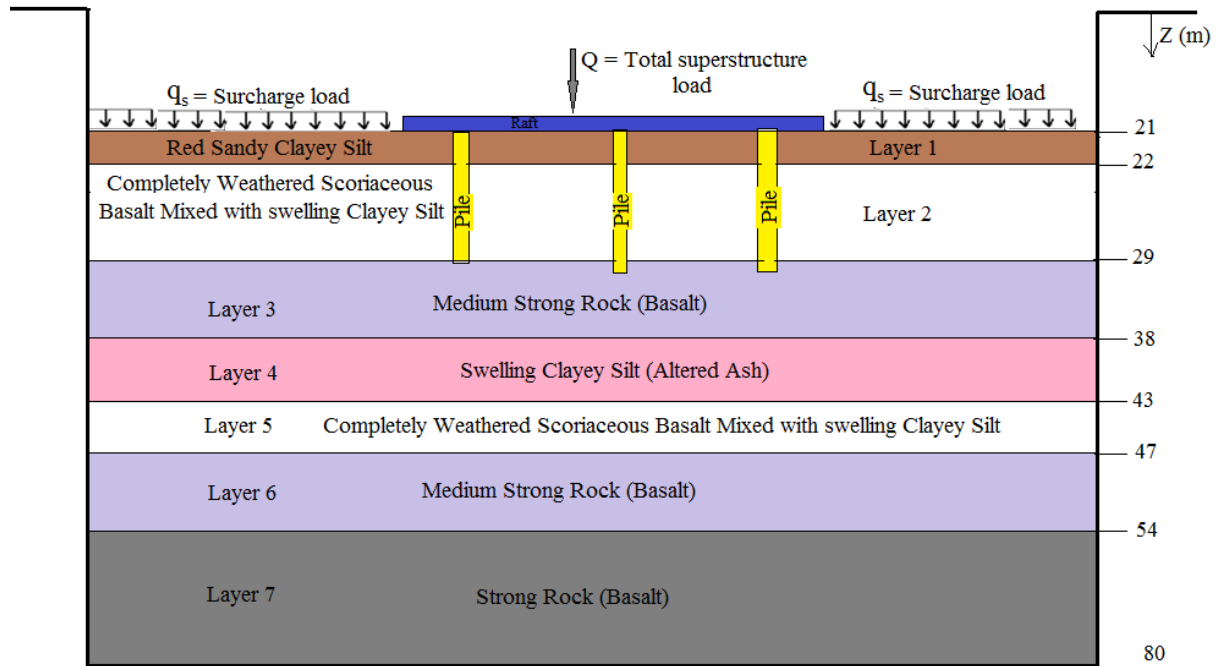


Figure 4.1: The elements foundation with Soil profile

The ultimate capacity of the raft alone can be determined using EBCS-7(Eq.4.1) in case of drained condition (long term).

$$q_{ult} = c N_c s_c + q' N_q s_q + 0.5 \gamma' N_\gamma s_\gamma r_\gamma \quad (4.1)$$

$$\text{Where: } N_q = e^{\pi \tan \phi} \tan^2 \left(45 + \frac{\phi}{2} \right)$$

$$N_c = (N_q - 1) \cot \phi$$

$$N_\gamma = (N_q - 1) \tan \phi$$

$$s_c = 1 + 0.2(B'/L')$$

$$S_q = 1 + (B'/L') \sin \phi$$

$$S_\gamma = 1 - 0.3(B'/L')$$

$$r_\gamma = 0.7B'$$

Taking the soil parameter from Table 4.1 and substituting in to (Eq. 4.1), the ultimate bearing capacity of the raft (q_{ult}) becomes: $q_{ult} = 6\text{MPa}$.

Taking a factor of safety of 3 the allowable bearing capacity (q_a) is 2 MPa. Since the basement is not backfilled the allowable bearing capacity is increased by the effective stress that is removed from the foundation level.

$$q_a = \frac{q_{ult}}{FS} + q'; \text{ Where: } (q' = \gamma'h, h = \text{foundation depth and } \gamma' = \text{effective unit weight})$$

$$q' = \gamma'h = (17.7 - 9.8) * 21 = 166$$

$$q_a = 2 + 0.166 = \mathbf{2.166 \text{ MPa}}$$

Multiplying this stress (q_a) by area of the raft gives 3024MN. The total superstructure load on the foundation including the raft weight is 1426 GN. The capacity of the raft is much greater than the given super structural load. Therefore the raft can carry double as much load. It can be said that bearing capacity failure may not occur on this building.

Table 4.1: Summary of soil parameters used for simplified design method

	Layer1	Layer2	Layer3	Layer4	Layer5	Layer6	Layer7
Total unit weight (γ_t) in kN/m^3	17	22	14	17.7	22	24	28
Cohesion (c) in kN/m^2	25	25	5000	25	5000	5000	5000
Poisson's ratio (ν)	0.35	0.3	0.25	0.35	0.3	0.25	0.25
Internal frictional angle, ϕ ($^\circ$)	27	42.8	45	27	42.8	45	45
Young's modulus (E) in MN/m^2	121.06	400	1000	130.95	400	1000	1500
Compressive strength, q_u (MN/m^2)	-	0.8	24	-	0.8	24	24

Layer1 [Red sandy clayey silt], **Layer 3&6** [Medium strong rock basalt], **Layer 7** [Strong rock basal], **Layer 2&5** [Completely weathered basalt mixed with swelling clayey silt], **Layer 4** [Swelling clayey silt or Altered Ash]

b) Bearing capacity of a single pile

The ultimate capacity of pile is derived from both its side and tip resistance. The side or shaft resistance of the pile is calculated using the method, whereas the tip resistance is taken from the compressive strength test result of the investigation report.

1) Skin frictional resistance using α method(Q_s):

$$Q_s = \sum A_s f_s \quad (4.2)$$

Where: $f_s = \alpha c + qk \tan \delta$,

A_s = effective pile surface area

c = average soil cohesion = 30kpa

q = effective vertical stress on the element

α = adhesion factor = 0.55

k = coefficient of lateral earth pressure = $1 - \sin \phi = 0.5$

δ = effective frictional angle between soil and pile material = 0.7ϕ

ϕ = angle internal friction of the soil = 37°

$$\begin{aligned} Q_s &= (3.14 * 2 * 10) * [(0.55 * 30) + ((166 + (17.7 - 9.8) * 10 / 2) * 0.5 * 0.48)] \\ &= (62.83) * [16.5 + 167.73] \\ &= \mathbf{12.06MN} \end{aligned}$$

Taking factor of safety (FS = 3)

$$Q_{s \text{ Allowable}} = 12.06 / 3 = \mathbf{4 \text{ MN}}$$

2) End bearing

$$Q_p = A_p q$$

Where: A_p = Cross sectional Area of the pile

q = compressive strength of the medium strong basalt layer ($q = q_{uit} / 10 = 2.4 \text{ Mpa}$)

$$\begin{aligned} Q_p &= (3.14 * 2^2 / 4) * 2.4 \\ &= \mathbf{7.536MN} \end{aligned}$$

Allowable total bearing capacity of the pile (Q_T) is the summation of shaft resistance and end bearing resistance

$$Q_T = 4 + 7.536$$

$$= 11.54 \text{ MN}$$

4.3 Assessment for average settlement of piled raft

a) Stiffness of the piled raft

For assessment of load - settlement behaviour of the foundation an approximate analysis of the piled raft was used as described by (H. Poulos 2002). This method uses the equations developed by Randolph (1994) to compute the stiffness of a piled raft, in terms of the raft and pile group stiffness values, and also the load sharing between the raft and the piles. A tri-linear load-settlement curve is derived from this process.

the pile group stiffness (K_{pg}) was estimated first by computing single pile stiffness from approximate solutions of [5], then multiplying this stiffness by group efficiency ($R = n^{1-e}$), where n = the number of piles, e = efficiency exponent, typically in the range 0.2-0.6, but varying with spacing. The efficiency component e can be calculated using (Eq.4.3). The correction factors (c_1, c_2, c_3, c_4) are determined using design charts given by (Fleming et al 1992). The design charts given by (Fleming 1992) are included in Appendix C.

$$e = e_1 \left(\frac{L}{d}\right) * c_1 \left(\frac{E_p}{G}\right) * c_2 \left(\frac{s}{d}\right) * c_3(\nu) * c_4(\rho) \quad (4.3)$$

The single pile stiffness is calculated using (Eq.4.4) and gives 4.75 GPa. The total pile group stiffness is determined as shown in Table 4.2.

$$K_p = G_s r_o \frac{\frac{4\eta}{(1-\nu)\xi} + \frac{2\pi L_p \tanh \mu L_p}{\zeta a \mu L_p}}{1 + \frac{1}{\pi\lambda} \frac{4\eta}{(1-\nu)\xi} \frac{L_p \tanh \mu L_p}{a \mu L_p}} \quad (4.4)$$

Where: G_s = soil shear modulus at the depth L_p , L_p = pile length, r_o = pile radius, $\eta = r_b / r_o$, r_b = pile radius at its base $\zeta = G_s / G_b$, G_b = shear modulus of the soil below the pile base, $\rho = G_{1/2} / G$ = medium shear modulus at the soil layer around the pile,

$\lambda = E_p / G_s =$ pile – stiffness ratio, $E_p =$ elastic modulus of pile material, $\zeta = \ln(r_m / r_o)$

$$r_m = L_p \{0.25 + \zeta[2.5\rho(1 - \nu) - 0.25]\} = \text{maximum radius of influence, } \mu L_p = \frac{L_p}{r_o} \sqrt{\frac{2}{\zeta\lambda}}$$

Table 4.2: Single and group pile stiffness.

SN.	Number of piles	Average Spacing (m)	Singe pile stiffness, K_p (GN/m)	Efficiency component e	Group efficiency $R = n^{1-e}$	Group pile stiffness K_{pg} (GN/m)
1	15	11	5.48	0.488	$15^{(1-0.488)} = 4$	19
2	30	7.7	5.48	0.533	$15^{(1-0.533)} = 4.89$	23.25
3	45	6.3	5.48	0.561	$15^{(1-0.561)} = 5.32$	25.26
4	60	5.4	5.48	0.589	$15^{(1-0.589)} = 5.38$	25.56

In order to determine the value of the raft stiffness K_r , first the equivalent soil Modulus and Poisson's ratio is determined by approximate graphical design method given by (Fraser 1976). By weighting the elastic parameters of each layer in a multi-layered system according to its influence on settlement an 'equivalent' modulus and Poisson's ratio can be determined for the overall system. This is by assuming that the summation of total settlement of n-layered soil is equated with the settlement of an equivalent single homogeneous layer. The 7 layers (Fig 3.0) are converted to single layer using this approach and gives an equivalent soil modulus of, $E_s = 0.3\text{MPa}$. The Detail procedures for converting the 7 layers into single a homogenous layer are included in Appendix C. on the other hand, the stiffness of the raft is determined using (Eq.3.3a) and yields $K_r = 12.91\text{Gpa}$. The detail calculation of the method is included in Appendix C.

$$K_r = \frac{E_s d}{(1 - \nu^2) I_F I_E} \quad (3.3a)$$

Where: $K_r =$ Raft stiffness,

$d =$ equivalent diameter of the raft,

$\nu =$ Poisson's ratio of the soil

$E_r =$ Equivalent elastic modulus of soil,

E_s = Elastic modulus of the raft,

t = thickness of the raft,

Z_E = depth of the foundation,

I_F = correction factor for rigidity,

I_E = correction factor for embedment

$$I_E = 1 - \frac{1}{3.5 \exp(1.22\nu - 0.4) \left[\frac{d}{Z_e} + 1.6 \right]} \quad (3.3b)$$

$$I_F = \frac{\pi}{4} + \frac{1}{4.66 + 10 * K_F} \quad (3.3c)$$

$$K_F = \frac{E_R}{E_S} * \left(\frac{2t}{d} \right)^3 \quad (3.3d)$$

Once have the stiffness of the pile group and the raft are determine, the overall stiffness (K_{pr}) of the piled raft and the load sharing ratio (X and α_{pr}) between the raft and piles can be established. The value of K_{pr} and X are calculated using (Eq.3.4 and 3.5) respectively. The load sharing ratio between the raft and piles are given Table 3.2.

$$K_{pr} = \frac{1 - 0.6 \left(\frac{K_r}{K_p} \right)}{1 - 0.6 \left(\frac{K_r}{K_p} \right)} K_p \quad (3.4)$$

$$\frac{P_r}{P_t} = \frac{0.2}{1 - 0.8 \left(\frac{K_r}{K_p} \right)} \frac{K_r}{K_p} = X \quad (3.5)$$

Where: K_{pr} , K_p and K_r are stiffness of piled raft, single pile and raft respectively

Table 4.3: Piled raft stiffness and load sharing ratio between raft and pile.

SN.	No. of piles	Group pile stiffness K_{pg} (GN/m)	Raft stiffness K_r (GN/m)	Piled raft stiffness K_{pr} (GN/m)	Load carried by the raft (X) in %	Load carried by the piles ($\alpha_{pr} = 100\% - X$)
1	15	19	12.91	19.77	24.11	75.89
2	30	23.25	12.91	23.95	16.74	83.26
3	45	25.26	12.91	25.93	14.64	85.36
4	60	25.56	12.91	26.22	14.37	85.63

b) Load settlement curve

The overall piled raft stiffness (K_{pr}) and the load sharing ratio (α_{pr}) by piles that are determined in Table 4.3 are key parameters for developing the tri-linear load-settlement curve of the piled raft foundation. This tri-linear load settlement curve has been determined using (Eq.3.6b and 3.6c).

Let P_u be the ultimate load capacity of the pile group and α_{pr} = load sharing ratio by the pile group, the load carried by the pile group (P_A) can be determine by (Eq.3b).

$$P_A = \frac{P_u}{\alpha_{pr}} \quad (3.6a)$$

If the total applied load on the foundation is less than or equals to P_A (Eq. 3.6a), the settlement of the piled raft is governed by the stiffness of the piled raft (K_{pr}) and it is estimated using (Eq.3.6b). If the total applied load is greater than P_A , the settlement is governed by the raft stiffness and it is determined using (Eq.3.6c). The average settlement the piled raft is estimated for various loads (up to nearly two times of the actual load) as shown in Table 3.3 and one can be observe from the result the raft settlement 6.86 cm is decreased to 3.65 cm by adding 60 piles below it for the actual load of the foundation.

$$\text{For } P \leq P_A, S = \frac{P}{K_{pr}} \quad (3.6 b)$$

$$\text{For } P > P_A, S = S_A + \frac{P - P_A}{K_r} \quad (3.6 c)$$

$$S_A = \frac{P_A}{K_{pr}} \quad (3.6 d)$$

Table 4.4: Average settlement of piled raft with different number of piles

Number of piles below the raft	0	15	30	45	60
Average settlement (cm) at (P = 100MN)	0.86	0.505	0.417	0.386	0.381
Average settlement (cm) at (P = 200MN)	1.71	1.11	0.835	0.771	0.763
Average settlement (cm) at (P = 400MN)	3.43	2.63	1.67	1.54	1.52
Average settlement (cm) at (P = 600MN)	5.14	4.34	3.31	2.31	2.22
Average settlement (cm) at (P = 900MN)	6.86	6.26	5.23	4.6	3.65
Average settlement (cm) at (P = 1200MN)	10.29	9.49	8.46	7.42	6.44
Average settlement (cm) at (P = 1500MN)	12.86	12.06	11.03	9.99	09.015

The load settlement curve of the piled raft system is shown in Figure 4.2 and this curve reveals that in the first stage of the curve (the first linear relationship between load settlements) the total settlement is governed by the pile group and raft, then in the second stage of the curve the piled raft is governed by the raft alone and it settles continuously after the ultimate capacity of the raft and piles is reached.

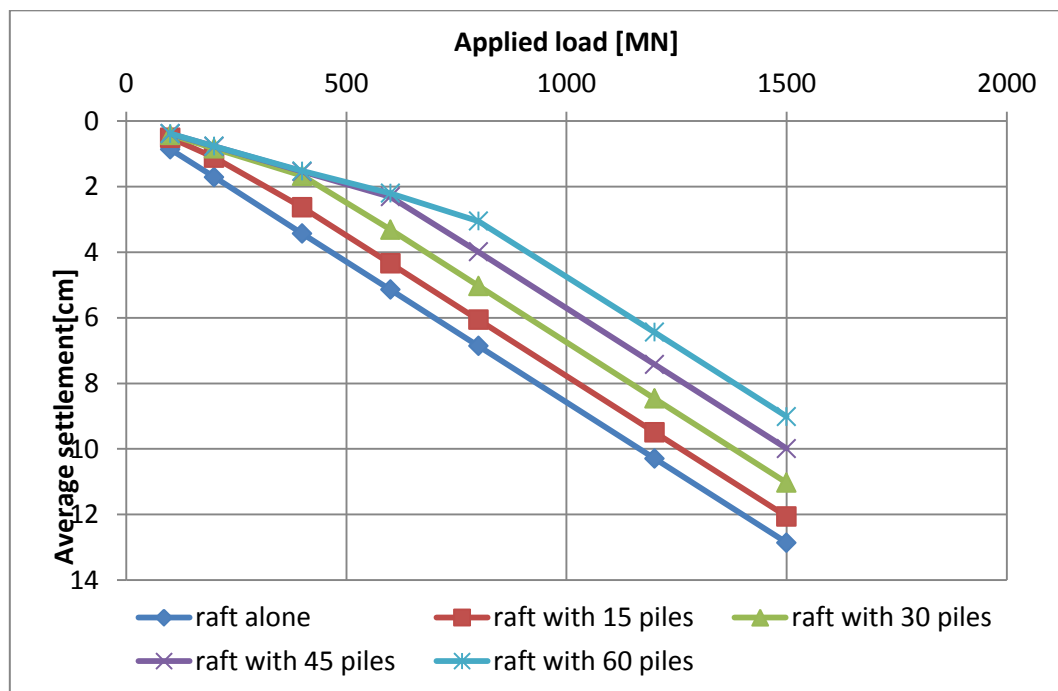


Figure 4.2: Load settlement relation of piled raft for preliminary designed.

CHAPTER FIVE

FINITE ELEMENT MODELLING AND ANALYSIS OF THE PILED RAFT FOUNDATION

5.1 Introduction

With the rapid development of computers, the application of numerical methods for solving geotechnical problems is becoming more recognized by both geotechnical researchers and other engineers. Numerical models allow for multiple simulations to be explored, and they also allow researchers to gain a better understanding of the mechanisms involved.

Finite element method is a common tool used for advanced numerical modelling within various fields of engineering. It is also the most powerful tool among the other method of numerical analysis: For the same reason, the finite element method was selected in this thesis to predict the load – settlement relationship and load sharing behaviour between piles and raft in piled raft foundations. The finite element software ABAQUS was used to simulate the interaction between the pile, raft and soil. It is a multi – purpose computer package that allows user to investigate mechanical, structural, and geotechnical problems under static and dynamic loadings. The package has the capability to model complex interaction between different bodies and to account the initial stress states of the material.

In this chapter the procedures in finite element methods required for modelling piled raft foundations are included. The different constitutive soil and concrete relationships used for the study are also included. In the last section of the chapter, validation of the model is carried out and its result is compared with other methods available in the literature.

5.2 Formulation of Finite Element Method (FEM)

The first step in FEM analysis is to define the problem domain, which usually includes all the materials enclosed by a set of restraints or boundary conditions. The problem domain can be broken (discretized) into a series of continuum elements to form a mesh. The elements in the problem domain are connected together by nodes that transfer forces and displacements between elements. The force applied on the element is represented by a load vector act of the forces that at the nodes.

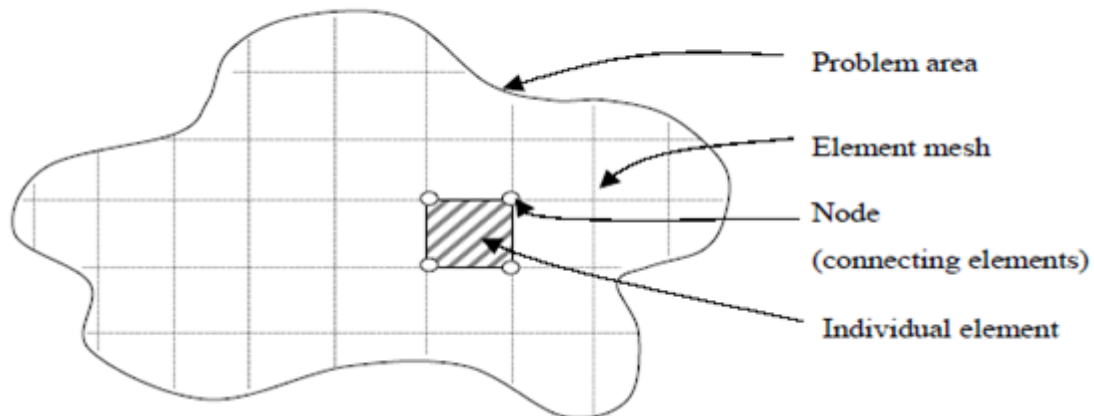


Figure 5.1: Typical mesh problem domain for FEM (Kate 2005)

Once the problem domain is defined and the element type is chosen, then individual element matrices can be formed. The individual elements have an element property matrix, a vector of unknown degree of freedom, and a resultant vector of element nodal forces. This relationship for static loading is described in the simple form of

$$[k]\{d\} = \{f\} \quad (5.1)$$

Where: $[k]$ = element property matrix, commonly stiffness or general property matrix

$\{d\}$ = vector of unknown degree of freedom (nodal displacement)

$\{f\}$ = vector of element nodal forces

The element functions are gathered in the global equation system containing material and geometric data. The unknown nodal displacement is solved from the global equation system and the values between the nodes are determined either by linear or polynomial approximated shape (or interpolation) functions.

The accuracy of a finite element analysis result depends on the element type, mesh size, boundary conditions, interface elements and constitutive models employed. Therefore, proper selection of these parameters is required for accurate results. Some of the parameters used for this study are reviewed as follows.

5.2.1 Element type

Different element types are available in ABAQUS software (i.e., beam, shell, and solid elements) which are implemented based on the nature of projects. The most commonly used in 3D analysis are tetrahedral or hexahedral continuum elements (Vakili 2015). These have been found very successful in predicting the behaviour of piled raft foundations. For each

type of elements, there are normally linear or quadratic elements with full or reduced integration. In general, quadratic elements are more accurate than linear elements but more expensive in calculations if the same mesh is used. Linear hexahedral elements with reduced integration (C3D8R) in ABAQUS can be used in large scale and complex analyses, on the balance of accuracy and efficiency (Sinha 2013). However, higher order elements are recommended for more accurate purposes when computational resource is not a problem. Due to lack of high processor computer resource, linear hexahedral elements with reduced integration (C3D8R) were used in this study to model the different layers of soil and linear wedge elements with reduced integration (C3D8R) for the soil around the piles and for the piles and raft themselves as well.

5.2.2 Boundary conditions

The boundaries of the soil in the model must be constrained in the vertical and lateral directions where the displacement of the soil due to the foundation load is negligible. In order to minimize the boundary effects in the model many authors suggested different locations of these boundaries. The horizontal length extent of 30D (30 times of the pile diameter) and vertical depth extent of 2L (two times the pile length) is sufficient after which no appreciable stress and strain variation effect was observed (Sinha 2013).

Therefore, boundaries of the model were placed at sufficient distances from the foundation so that the influence of the boundaries on the deformations of the foundation is minimized. The horizontal boundaries are constrained by the location of shoring walls (which retain the excavated depth) when the wall location is greater than (30D) and at 30D when the wall is nearer than 30D. The vertical boundary is fixed at depth of 5L(5 times of the pile length) from the bottom of raft. Three separate boundary conditions were imposed onto the model. Due to symmetry, the problem is reduced to half of its original size. Nodes on both lateral boundaries of the model are fixed against horizontal movement ($u_x = 0$ and $u_y = 0$), yet free to move in the vertical direction. Meanwhile, nodes on the bottom boundary of the model are fixed against all directions ($u_x = u_y = u_z = 0$), whereas the top boundary is free to move in all directions. Nodes on the plane of symmetry are restrained from moving normal to plane of symmetry, but can move along the plane of symmetry as shown in Figure 5.2.

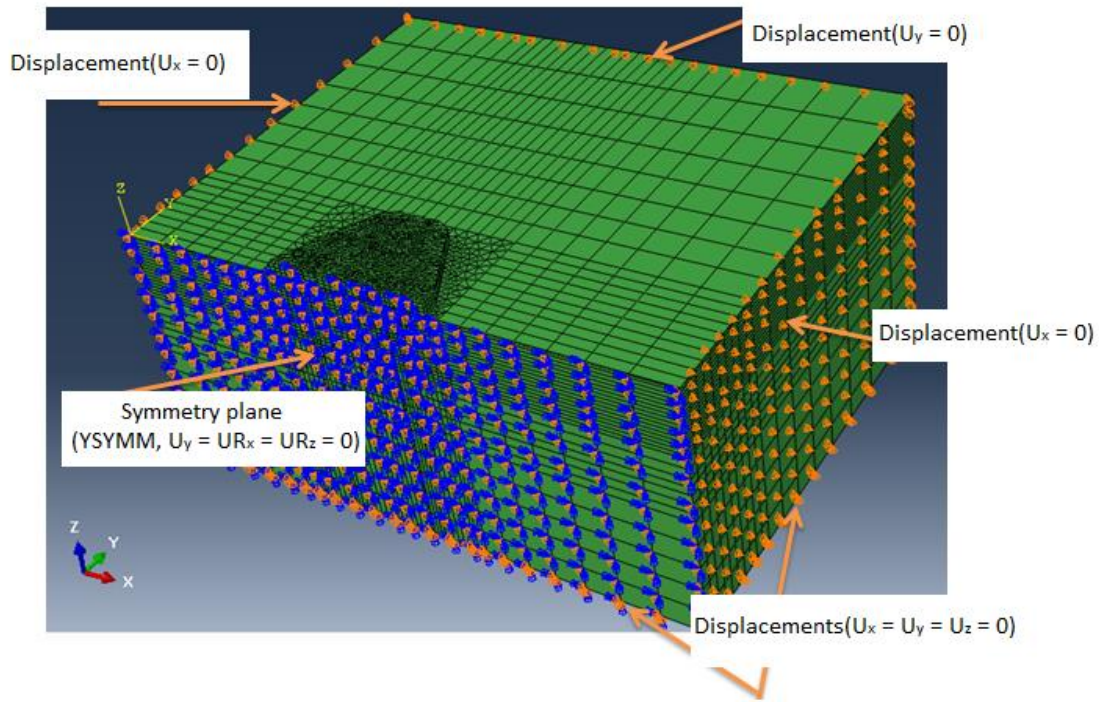


Figure 5.2: Boundary conditions

5.2.3 Sensitivity analysis

As mentioned above, in the finite element calculations, the model has to be divided into elements which compose the “finite element mesh”. The finite element mesh size will possibly influence the results. To investigate the influence of number of elements or mesh coarseness on the bearing behaviour of pile raft foundation, five different finite element meshes were performed. These were done by varying their number and sizes keeping material properties and all other parameters constant. The generated number of elements and the total maximum settlement in the foundation is given in Table 5.1.

Table 5.1: Influence of mesh size on maximum Total settlement

Model	Mesh Coarseness	Number of elements	Maximum settlement u_z (mm)	Time required to complete
Model 1	Coarse mesh	45516	76.35	8 minutes
Model 2	Coarse mesh	54015	77.81	10 minutes
Model 3	Medium mesh	74391	80.65	15 minutes
Model 4	Medium mesh	84432	81.23	20 minutes
Model 5	Fine mesh	150257	81.48	365 minutes

Figure 5.3: Shows the influence of mesh density on the total settlement of the building. As we see from this figure maximum settlement is increases with increases number of element till the number of element reach 84432, after that an increasing in number of elements has no significant effect on the maximum settlement of the building. It can be said that mesh is converged with 84432 elements because there is no significant change in deformation when the number of elements increased to 150257. Therefore model14 (medium mesh) was used to investigate the general behaviour of the piled raft foundation.

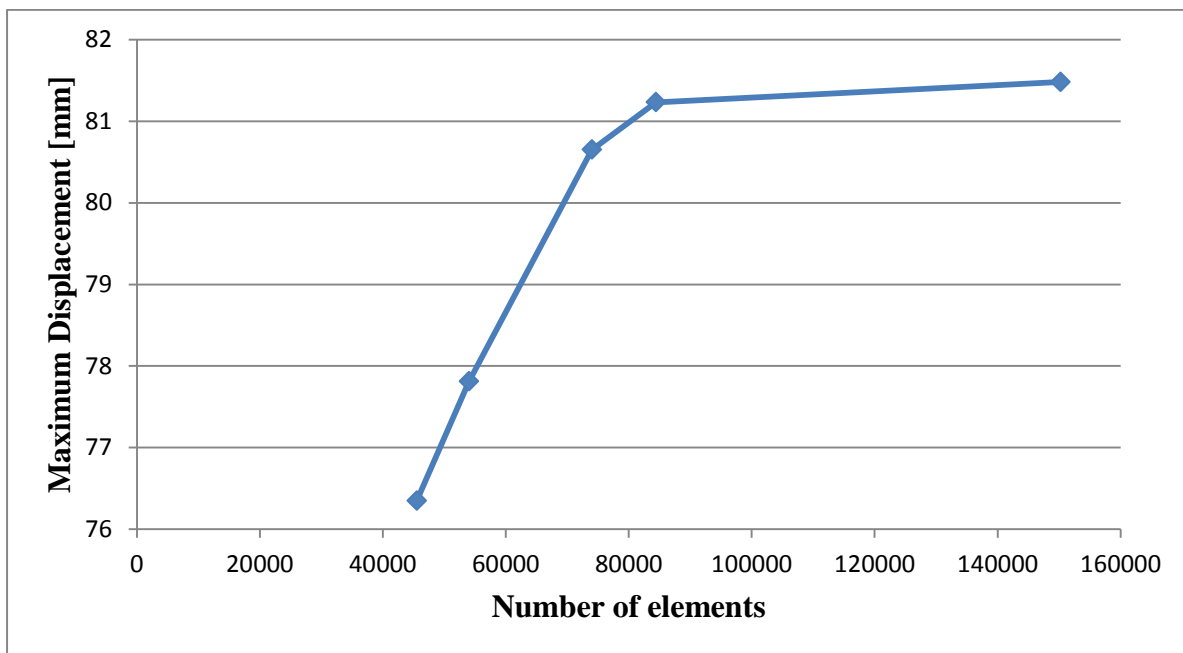


Figure 5.3: Influence of mesh density on maximum settlement of the building

5.2.4 Contact interface

When we model foundations, the contact condition between the soil and the structures must be specified so as to simulate its realistic behaviour. The soil structure interaction is the main driving factor of the load sharing mechanism that requires proper modelling to achieve accurate numerical results (Vakili 2015). There are two types of modelling techniques used for the pile – soil and raft - soil interfaces. One is a slip element and the other is thin layer element. Advanced finite element code such as ABAQUS has a special contact element (slip element) that may be used at the soil structure interface. Due to its simplicity slip element is preferred by (Lee 2010) and (Taghavi 2016). Unfortunately this contact element is costly in terms of time and computer memory. It has also some convergence problem. Alternatively,

thin solid continuum element was used to simulate the contact zone between soil and raft, and between soil and the large-diameter bored instead of special interface elements. In thin layer element method, the contact between structure and soil was described as perfectly rough. This means that no relative motion takes place between the nodes of the finite elements that represent the structure and those of the finite elements that represent the layer of soil. The material behaviour in the contact area (the thin element) was simulated by the material behaviour of the soil. (Reul and Rondoph 2004) use thin-layer element at the pile–soil interface and obtaining a good agreement between the results of pile shaft resistance predicted by bearing capacity theory and the results calculated by numerical analyses. Therefore, thin solid Continuum elements of thickness $0.1D$ ($D = \text{Pile diameter}$) at the pile-soil interface and a thickness of $0.1H$ ($H = \text{Raft thickness}$) at raft - soil interface have been used in this study as recommended by (Reul and Rondoph 2004).

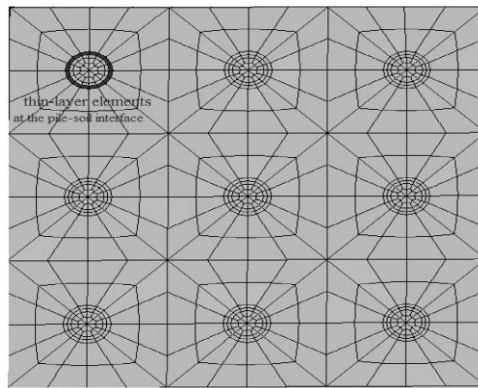


Figure 5.4: Application of thin layer element at the interface between soil and structure taken from [10]

5.2.5 Loading

The present study focuses on behaviour of piled raft foundation system under vertical loads. The load configuration is one of the crucial parameters that affect the failure mechanism of the foundation. The superstructure load transferred through the shear walls (core walls) and columns, acting on the top of the raft. Any load in ABAQUS, whether it is self-weight or external applied load, is applied in incremental time step. The following steps are used for applying boundary conditions and different type of loading conditions. There are six steps in this model, these are

1. initial step
2. primary load step

3. excavation step
4. pile installation step
5. raft installation step and
6. loading step

1. Initial step (type initial condition)

Initial step is used to apply boundary condition and initial stress state field of the soil. In analysing most structures, it is appropriate to begin with a complete mesh of stress-free unreformed elements and subsequently apply the specified loads to obtain the desired stress state. However, buried structures are exceptional that their response depends on the history of the loading, i.e. in-situ state of stress in the ground (Juryk 1993). Therefore it is important to apply initial conditions (pre-defined stress) before applying any external loads. In defining the stress field, vertical stress at two points should be defined and the variation between those two points is considered linear (Thanuja 2014). Here, vertical stress at a point (σ'_v) is determined by considering the number of soil layers that lie above the point considered (n) and given by:

$$\sigma_v = \sum_{i=1}^n (\rho g h_i - u) \quad (5.2)$$

Where: σ_v = effective vertical stress

ρ = density of soil

g = gravity

h_i = thickness of the soil(i^{th} layer)

u = pore water pressure

n = number of soil layer

For the given effective vertical stress field there is corresponding effective horizontal stress and this can be determined by multiplying the effective vertical stress with effective earth coefficient (K_o). K_o can be approximated by $K_o = 1 - \sin\phi$.

2. Primary load step (type Geostatic)

Geostatic stress field procedure is used in ABAQUS to verify that the initial geostatic stress field is in equilibrium with gravity loads and boundary conditions, or to iterate if necessary, to obtain equilibrium condition. The geostatic stress field procedure is normally used as the first step of a geotechnical analysis. In this step all the structural elements, such as the raft and the piles are removed or deactivated and gravity load is applied only to the soil. Ideally, the gravity loads and initial stresses should exactly equilibrate and produce zero deformations.

3. Excavation step (type static general)

The soil excavation can be modelled by deactivating the corresponding finite elements in the model. When the elements are deactivated in the analysis, the contribution of mass and stiffness of that element is removed from the global Mass and Stiffness matrix. The excavated soil above the foundation level was removed in this step in order to account the relief pressure. In addition bore holes were created for the piles to be installed by deactivating all the elements at the pile location as shown in Figure 5.4.

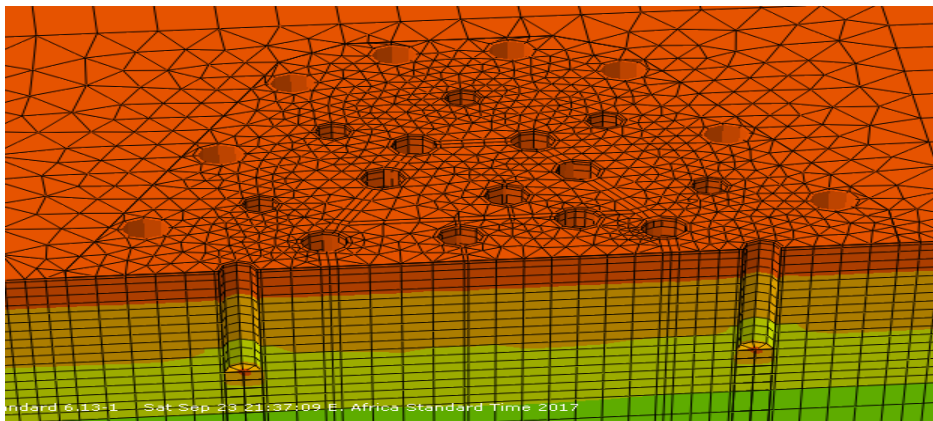


Figure 5.5: Excavation of soil at the locations of piles (enlarged view at pile and raft location)

4. Pile installation step (type static general)

In this step installation of the foundation piles is simulated by activating the piles (previously deactivated at step 2) in place of finite elements representing the pile which is removed or deactivated from soil elements in the preceding step (step 3) as shown in Figure 5.5.

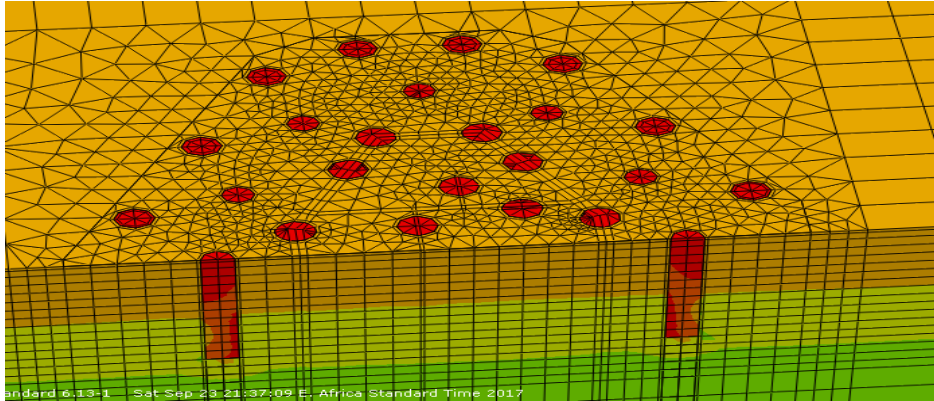


Figure 5.6: Pile installation step (enlarged view at pile and raft location)

5. Raft installation (type static general)

In this step installation of the foundation raft is simulated by activating which is previously deactivated at step 2 as shown in Figure 5.6.

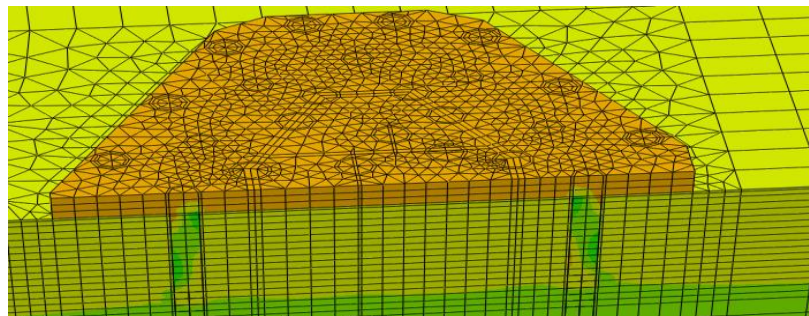


Figure 5.7: Raft installation step (enlarged view at pile and raft location)

6. Loading step (type static general)

The super structural load is applied on the raft through shear wall and columns. The application of load on the developed model is carried out using pressure rather than nodal loads. Nodal loads have a problem of excessive deformation in node of application (Bakroun 2012). So in this model pressure loads chosen for application of the load to avoid excessive deformation. Another reason for choosing pressure load is to simulate the real loads applied on the raft from columns and superstructure on the raft as shown in Figure 5.8.

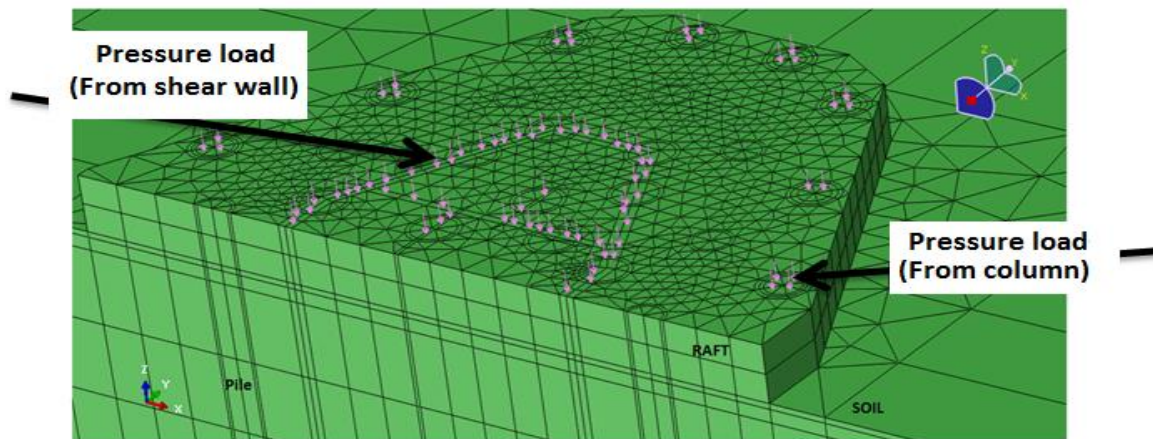


Figure 5.8: Application of the super structural load on the raft (enlarged view at pile and raft location)

5.3 Constitutive models

5.3.1 General

In material engineering a constitutive law or a constitutive equation describes the relationship between a material deformation and external loading. Soil Constitutive models are more complicated than linear elastic models. The result of finite element method is affected by the choice of constitutive model which represents the material behaviour.

There are many constitutive models in literature used to simulate the soil behaviour such as the linear elastic model, Mohr-Coulomb model, modified Drucker-Prager/Cap plasticity models. Since the stress – strain behaviour of soil is non-linear particularly at failure; elastic perfectly-plastic Cap plasticity model was used to simulate the non-linear stress-strain behaviour of the soil.

The piles were assumed as non-displacement concrete piles. The raft was considered as a reinforced concrete slab. The behaviour of the raft and the piles were assumed linear. Therefore, the linear-elastic model was utilized to simulate the materials behaviour of the piles and the raft. For the linear-elastic model two main parameters are used, which are the modulus of elasticity, E , and Poisson's ratio, μ . The following sections will discuss the two constitutive models used in this study, namely the elastic model for concrete and Drucker - Prager /Cap plasticity model for soil.

5.3.2 Elastic material model

The available theory for elasticity was developed and established on the basis of homogenous and isotropic behaviour of construction material (e.g Steel, iron). Elastic constitutive models can be linear or nonlinear Figure 5.9.

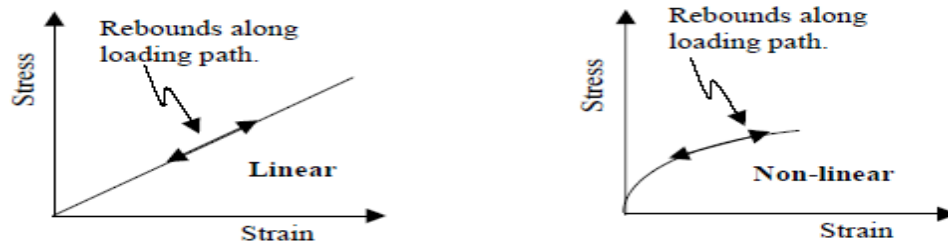


Figure 5.9: Linear and non- linear relation of stress and strain curve (Kate 2005)

Linear elastic constitutive model is probably the most common model used to approximate the stress – strain relationship of materials. Linear elastic models involve two elastic stiffness parameters namely Young’s modulus (E) and Poisson’s ratio (ν). These two parameters are used to relate stresses as function of strain in this manner:

$$\sigma_x = \left\{ \frac{E}{(1 + \nu)(1 - 2\nu)} \right\} (\epsilon_x (1 - \nu) + \nu(\epsilon_y + \epsilon_z)) \quad (5.3a)$$

$$\sigma_y = \left\{ \frac{E}{(1 + \nu)(1 - 2\nu)} \right\} (\epsilon_y (1 - \nu) + \nu(\epsilon_x + \epsilon_z)) \quad (5.3b)$$

$$\sigma_z = \left\{ \frac{E}{(1 + \nu)(1 - 2\nu)} \right\} (\epsilon_z (1 - \nu) + \nu(\epsilon_y + \epsilon_x)) \quad (5.3c)$$

Where: $\sigma_x, \sigma_y, \sigma_z$ = normal stresses in x,y and z directions respectively and

$\epsilon_x, \epsilon_y, \epsilon_z$ = normal strains in x,y and z directions respectively

The behaviour of soil mass, which is a combination of a number of discrete particles, cannot be modelled by the pure elastic theory, in reality; the stress-strain behaviour of soil becomes non-linear, particularly as failure conditions are approached (Sinha 2013). Hence, the researcher’s represent the soil stress-strain constitutive behaviour by means of elasto-plastic, models, which are the combination of the elastic and plastic theories.

5.3.3 Modified Drucker - Prager /Cap plasticity model

The modified Drucker – Prager /Cap plasticity model is the modification of Drucker-Prager plasticity model which was proposed by Drucker and Prager in 1952 for frictional soils only,

later it has been modified with post- yield strain hardening and compression cap in the yield surface.

The Drucker–Prager/cap plasticity model has been widely used in finite element analysis programs for a variety of geotechnical engineering applications. The cap model is appropriate to soil behaviour because it is capable of considering the effect of stress history, stress path, dilatancy, and the effect of the intermediate principal stress. The yield surface of the modified Drucker–Prager/cap plasticity model consists of three parts: a Drucker–Prager shear failure surface (F_s), an elliptical cap (F_c), which intersects the mean effective stress axis at right angle, and a smooth transition region between the shear failure surface and the cap (F_t) as shown in Figure 5.10. Volumetric strain hardening is defined by moving the cap along the hydrostatic axis. The model has two purposes: to bound the yield surface in hydrostatic compression, thus providing an inelastic hardening mechanism to represent plastic compaction, and to help control volume dilatancy when the material yields in shear by providing a softening law created as the material fails on the Drucker-Prager shear failure surface.

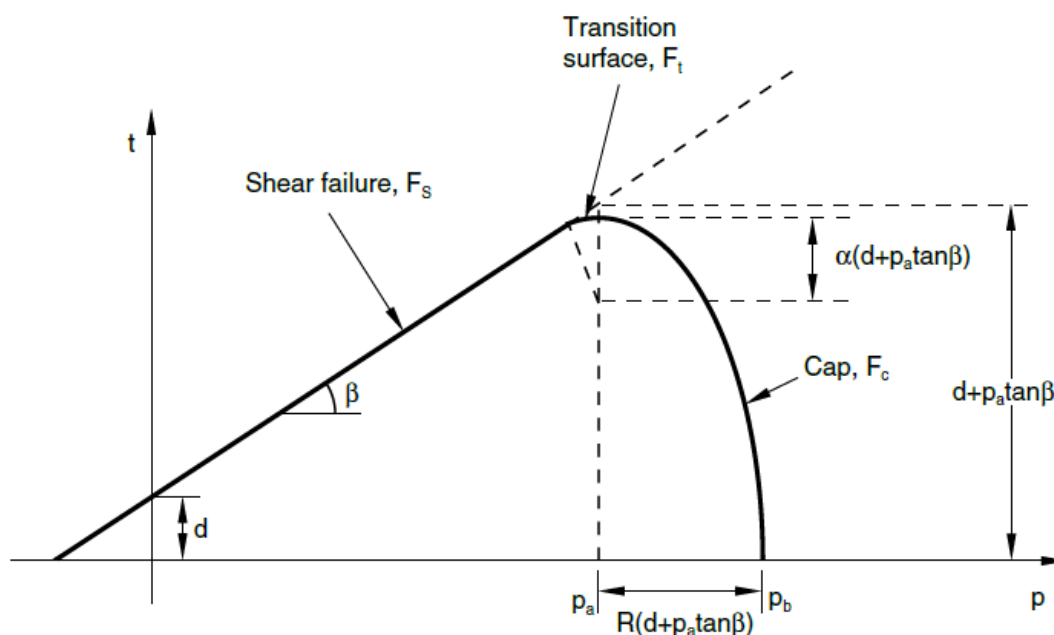


Figure 5.10: Modified Drucker-Prager/Cap model: yield surfaces in the p - t plane
(Abaqus 6.15 manual)

a) The Drucker-Prager Yield or Shear Failure Surface

The Drucker-Prager yield surface is a pressure dependent shear failure surface (F_s), which is a perfectly plastic yield surface (no hardening). Changes of stress inside the yield surfaces cause elastic deformations, while changes of stress on the yield surfaces cause plastic deformations. The failure surfaces in ($p-t$) plane are described by the shear failure surface,

$$F_s = t - P \tan \beta - d = 0 \quad (5.4)$$

Where: t = deviatoric stress with a distinction between compression and extension by

$$t = \frac{q}{2} \left[1 + \frac{1}{K} - \left[1 - \frac{1}{K} \right] \left[\frac{r}{q} \right]^3 \right] \quad (5.5)$$

$$q = \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \quad (5.6)$$

$$P = \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3) \quad (5.7)$$

q = deviatoric stress (Misses equivalent stress)

d = cohesion in the $p-t$ -plane

P = hydrostatic stress

b = slope of the yield surface FS in the $p-t$ -plane

K = shape parameter of the yield surface F_s determined from tri-axial compression and extension tests with usual values, $0.778 < K < 1$.

b) The Elliptical Cap Yield Surface

The cap yield surface is an ellipse with eccentricity = R in the $p-t$ plane. The cap yield surface is dependent on the third stress invariant, r , in the deviatoric plane as shown in Figure 5.11. The cap surface hardens (expands) or softens (shrinks) as a function of the volumetric plastic strain. When the stress state causes yielding on the cap volumetric plastic strain (compaction) results, which causing the cap to expand (hardening). When the stress state causes yielding on the Drucker– Prager shear failure surface volumetric plastic dilation results, causing the cap to shrink (softening). The cap yield surface is given as shown on Eq

$$F_c = \sqrt{(P - P_a)^2 + \left(\frac{Rt}{1 + \alpha - \alpha / \cos \beta} \right)^2} - R(d + P_a \tan \beta) = 0 \quad (5.8)$$

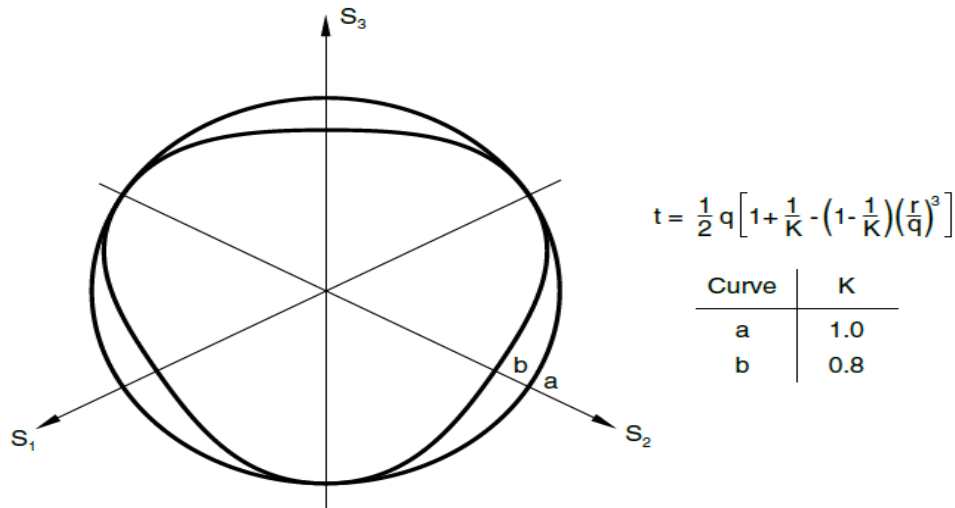


Figure 5.11: Projection of the modified cap yield surface on the π -plane (Abaqus 6.15 manual)

The hardening/softening law is defined by a piecewise linear function relating the hydrostatic compression yield stress, p_b , to the volumetric inelastic strain as shown in Figure 5.11.

The evolution parameter, p_a is related to p_b by

$$P_a = \frac{P_b - Rd}{1 + R \tan \beta} \quad (5.9)$$

c) The Transition Yield Surface

The coupling between the Drucker-Prager yield surface and the elliptical cap is made by a smooth transition curve surface which is defined as

$$F_c = \sqrt{(P - P_a)^2 + \left[1 - \left(1 - \frac{\alpha}{\cos \beta} \right) (d + P_a \tan \beta) \right]^2} - \alpha(d + P_a \tan \beta) = 0 \quad (5.10)$$

5.3.4 Model parameters

The definition of the modified Drucker-Prager cap plastic model includes plasticity and hardening parameters. The plasticity parameters, $d, \beta, R, \varepsilon_{vol}^{pl}$ (initial volumetric plastic

strain), α and K define the shape of the yield surface in the stress space while the hardening law is defined by pairs of the hydrostatic compression yield stress and volumetric inelastic strain during the consolidation process.

a) Plasticity parameters

To determine the parameters d and β at least three triaxial compression tests is required (Helwany 2007). At the failure conditions taken from the tests results can be plotted in the $p-t$ plane. A straight line is then best fitted to the three (or more) data points. The intersection of the line with the t -axis is d and the slope of the line is β . We also need the results of one dimensional consolidation test with several unloading–reloading cycles. This can be used to evaluate the hardening law as a piecewise linear function relating the hydrostatic compression yield stress p_b and the corresponding volumetric plastic strain ε_{vol}^{pl} . The Drucker – Prager model parameters d and β also can be matched to Mohr – Coulomb parameters (C and ϕ) as Eq.5.10 (Helwany 2007). In this thesis, Eq.5.10 was used to determine these parameters (d and β) and the input parameters used for the software are given in Table 5.3 and 5.4

$$\tan \beta = \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (5.10a)$$

$$d = 1 - \frac{1}{3} \tan \beta \frac{\cos \phi'}{1 - \sin \phi'} * 2c' \quad (5.10b)$$

The initial cap yield surface position, ε_{vol}^{pl} defines the initial cap position and it is a small number or zero (0) is taken ABAQUS manual.

The transition surface radius parameter, α , is a small number (typically in the range of 0.01 to 0.05), and relates to the radius of the transition yield surface that provides a smooth intersection between the cap and failure surface. The ratio of the stress in triaxial tension to the flow stress in triaxial compression, K , is assumed to be 0.888 in this research.

b) Cap hardening behaviour

The cap hardening curve is obtained from the isotropic consolidation test results. But the isotropic consolidation test results of some layers are not included in the soil investigation report. Therefore the missed parameters used for developing cap hardening curve were taken from literature review.

The cap hardening curve is determined using Eq.5.11 and the isotropic consolidation parameters are given in (Chapter 3 Table 3.4).

$$\varepsilon_v^p = \frac{C_c - C_r}{2.3(1 + e_o)} \ln \frac{P}{P_o} \quad (5.11)$$

Table 5.2: Cap hardening behaviour of the soil

	layer1	layer2	layer3	layer4	layer5	layer6	layer7
ε_v^p	P [kPa]	P [kPa]	P [kPa]	P [kPa]	P [kPa]	P [kPa]	P [kPa]
0	366	462	646	786.25	874.5	1002.5	1506.5
0.01	512.518	638.4238	918.3368	1000.831	1084.937	1425.128	2084.429
0.03	1004.998	1219.111	1855.842	1621.666	1669.914	2880.002	3990.461
0.08	5411.348	6142.978	10774.28	5419.542	4908.15	16720.15	20235.45
0.1	10611.13	11730.41	21773.45	8781.386	7554.527	33789.29	38739.04
0.15	57134.95	59108.34	126408	29347.04	22203.99	196167.2	196444
0.18	156886.7	155973.3	363147.4	60529.21	42399.94	563553.1	520346.5
0.2	307639.5	297840.9	733874.7	98076.62	65261.16	1138869	996158.8
0.3	8919133	7562330	24735269	1095388	563770.4	38385615	25615824

Layer1 [Red sandy clayey silt], **Layer 3&6** [Medium strong rock basalt], **Layer 7** [Strong rock basal], **Layer 2&5**[Completely weathered basalt mixed with swelling clayey silt], **Layer 4** [Swelling clayey silt or Altered Ash]

Table 5.3: Summary of basic concrete parameters used for the software

	Modulus of Elasticity	Poisson's ratio	Effective Unit weight
Symbols and units	E[GN/m ²]	V	γ' [kN/m ³]
Raft	35	0.2	15
Pile	36	0.2	15

**INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD**

Table 5.4: Summary of basic soil parameters used for the software

	Sym.	Layer1	Layer2	Layer3	Layer4	Layer5	Layer6	Layer7
Modulus of Elasticity	E [MN/m ²]	121	400	1000	131	400	1000	1500
Effective Unit weight	γ' [kN/m ³]	7	12	14	7.7	12	14	18
Poissons ratio(ν)	-	0.3	0.3	0.25	0.3	0.3	0.25	0.25
Frictional angle	ϕ' [°]	27	42.8	45	27	42.8	45	45
Cohesion	C [kN/m ²]	25	25	5000	25	5000	5000	5000
Slope of the yield surface Fs in the p-t plane	β [°]	47	58	61	47	58	61	61
Intersection of yield surface Fs with the t axis	d [kN/m ²]	52.4	53.65	9600	52.4	9600	9600	9600
Initial cap position	ϵ_{vo}^{pl}	0	0	0	0	0	0	0
Flow stress ratio	K	0.888	0.888	0.888	0.888	0.888	0.888	0.888
Cap Eccentricity	R	0.5	0.6	0.8	0.5	0.8	0.8	0.8

Layer1[Red sandy clayey silt], **Layer 3&6** [Medium strong rock basalt], **Layer 7** [Strong rock basal], **Layer 2&5**[Completely weathered basalt mixed with swelling clayey silt], **Layer 4** [Swelling clayey silt or Altered Ash]

5.4 Model validation

The software which is used for developing the model must be validated before its result is accepted and applied to simulate the real world problems. Validation is the only way to justify the predictions of a numerical model to the true physics concerned (Sinha 2013).

To check the validity of the results the model (ABAQUS), an example of piled-raft foundations was analysed. The model reported in the American Society of Civil Engineers (ASCE) Technical Committee – 18 (TC -18) report in (H. G. Poulos 2001), has been used in this thesis for validation purpose. Fig. 5.12 shows the layout of the piled-raft foundation considered in this analysis.

Notes: • $P_2 = 2P_1$

• For 3 piles, piles are located below P_2

• For 9 piles, piles are located below P_1 & P_2

• For 15 piles, piles are located below P_1 & P_2 and at position A

Bearing capacity of raft = 0.3 MPa

Load capacity of each pile

= 0.786 MN (Tension)

= 0.873 MN (Compression)

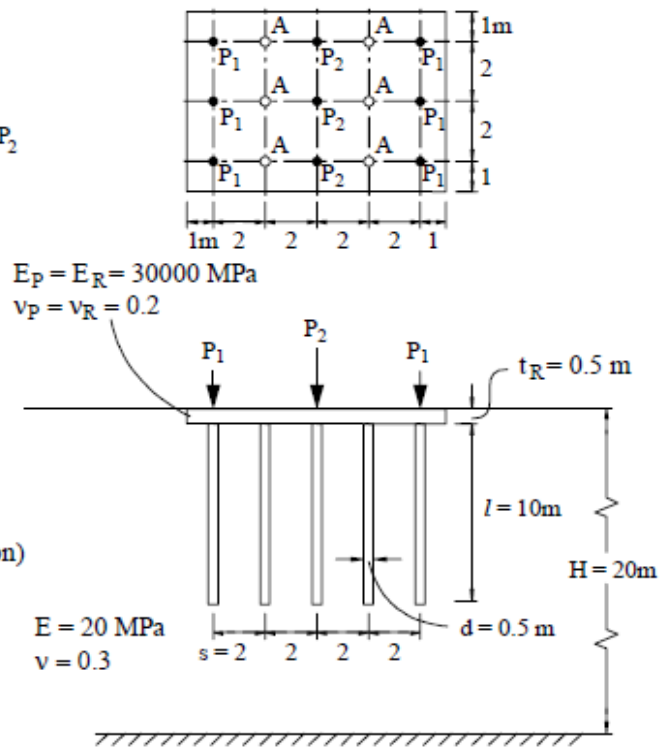


Figure 5.12: Layouts of piled-raft foundations taken from [H. G. Poulos 2001].

As shown in Figure 5.12 the raft dimension is 10m by 10m with 0.5m thickness and the 9 piles are 10m in length and 0.5m in diameter. One quarter of the foundation has been taken into account for simulation purpose. The middle three piles in transverse direction were subjected to the concentrated load of 2MN, while the edge piles were subjected to the concentrated load of 1MN. The result of settlement contour simulated in this model is given in Figure 5.13.

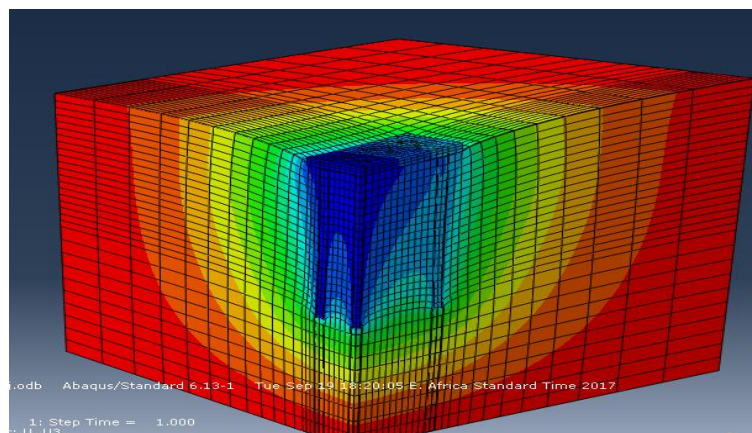


Figure 5.13: ABAQUS settlement contours in the z (vertical) direction

The Load-settlement predictions using the developed ABAQUS 3D model is in good agreement with the predictions of Poulos-Davis-Randolph (PDR), Geotechnical Analysis of Raft with Piles (GARP5), Geotechnical Analysis of Strip on Piles (GASP) and FLAC 3D. Comparison between the results of the developed model and other models is summarized in Table 5.5

Table 5.5: Comparison of the results of ABAQUS 3D model with other models for total of 12MN

Model	Central Settlement (mm)	Corner pile Settlement(mm)
FLAC 2D Predicted by Poulos(2001)	65.9	60.5
FLAC 3D Predicted by Poulos(2001)	39.9	35.8
PDR Method	36.8	-
GASP	33.8	22
GARP5	34.2	26
PLAXIS 2D Predicted by Omeman(2013)	32	26
The developed ABAQUS 3D (in this thesis)	31	26.4

CHAPTER SIX

RESULTS AND DISCUSSIONS

6.1 Introduction

The main objective of this study is to investigate the contribution of the raft to bearing behaviour of piled raft foundation and to assess load sharing mechanisms between the raft and piles. As shown in chapter 5, an ABAQUS model has been developed to study the behaviour of piled raft foundation in case of pile tips resting on a stiff stratum. In addition, to ensure the reliability of the results obtained by the developed model, comparison study was carried out between the results given by (H. G. Poulos 2001) and those obtained from the current FE model with similar real structure geometry, loading and properties.

The scope of this study is to investigate the contribution of raft, on settlement, load sharing behaviour of the piled raft foundation and to assess its failure behaviour in the cases where, weak stratum located under hard stratum below the pile tips. Therefore, the results obtained by the developed model are related with these parameters. Furthermore, discussions are made by considering comparisons to previous works related to the specific topic under consideration.

6.2 Behaviour of the raft alone

The raft was analysed assuming that it can support the entire load. Figure 6.1 shows the contour of the vertical settlement. It undergoes total settlement of 5.1 cm and differential settlement of 1.14 cm. When this result is compared with the result of the simplified method, the simplified method gives higher settlement (6.86 cm). The maximum, minimum and tilting of the foundation is given in Table 6.1. As expected the location of the maximum and minimum settlements of the raft occur at the centre and corner respectively. It is usually the differential settlement rather than total settlement which causes damage to structures. In order to accommodate the structure itself from harmful distortion, EBCS- 7 and (Bowles 1996) limits the maximum differential settlement and tilting of any building are 75% of total settlement and 1/500 respectively. It should be noted that the maximum differential settlement and tilting of the building are less than the maximum specified allowable value. Therefore these results cause no negative effect on the serviceability of the building. As discussed in Chapter 4, the adequacy of the raft alone in terms of bearing capacity and settlement requirements should be investigated first before adapting piled raft system. The

results showed that the raft can carry double as much the superstructure load. Settlement of the raft is also within the acceptable limit given by EBCS- 7 and (Bowles 1996) recommendations. Therefore, it can be concluded that the raft is adequate both in terms of bearing terms of bearing capacity and settlement requirements.

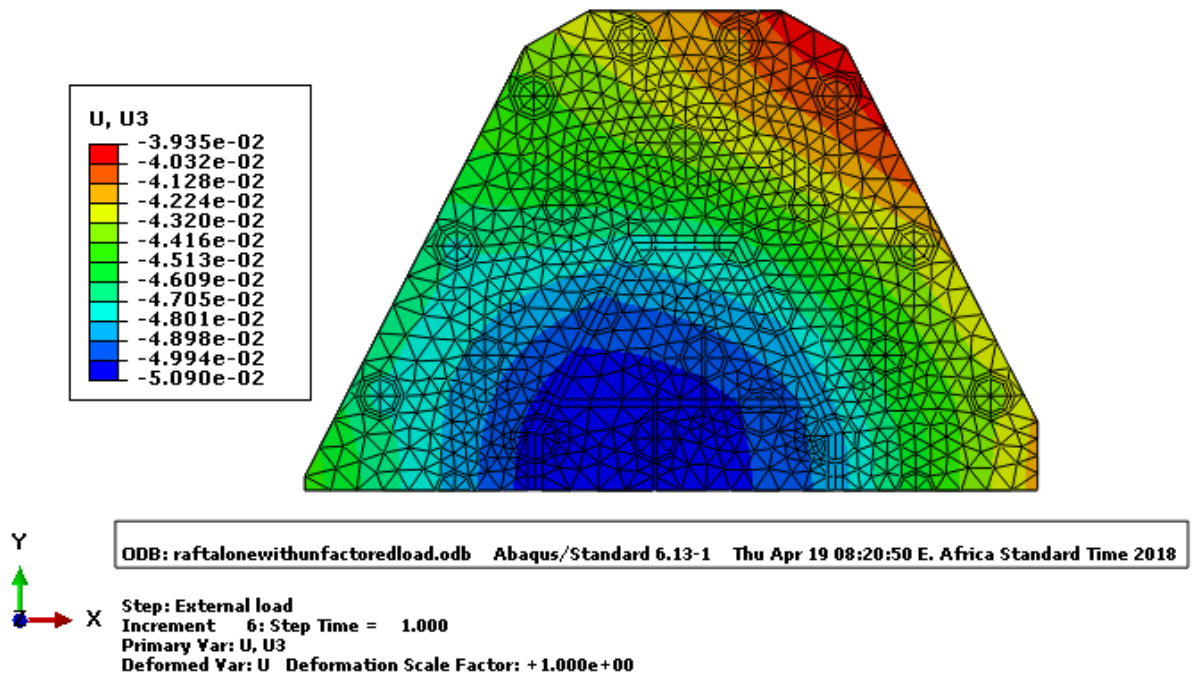


Figure 6.1: Contours of vertical displacement the raft at full load (units: m)

Table 6.1: Shows the differential settlement and tilting of the building on raft alone

Maximum settlement [mm]	Minimum settlement [mm]	Differential settlement [mm]	Tilting
50.9	39.5	11.4	0.00044

6.3 General behaviour of piled raft foundation

As mentioned in Chapter 3 the actual geometry of the piles is enlarged (belled shape) at their tips. But this belled shape was not considered in the model. Length of the bored pile varies (ranged from 5m to 10 m) so as to reach the hard stratum. The variable soil profile was simplified to horizontal because of its difficulty to consider in the model. In order to assess the behaviour of piled raft under consideration, two cases have been taken. In the first case the actual length of piles was considered. In this case some pile did not reach the hard stratum due to simplification made for the soil formation. In the second case, an average length of the piles was analysed. Here all pile tips are extended up to the hard stratum.

Case 1: Using actual length of the piles

Figure 6.2 shows contours of settlement of the raft at total working load. The location of the maximum and minimum settlements of the raft occur at the centre and corner respectively. The results of the analysis show maximum and minimum settlements of the raft as 4.23 cm and 2.7 cm respectively. This gives 1.53 cm maximum differential settlement and 1/1695 tilting of the building as shown in Table 6.2. It can be noticed that the maximum settlement of piled raft foundation is almost the same with the maximum settlement the raft alone. The maximum settlement reduction of the raft by adding 46 bored piles below it is less than 1cm. In addition maximum differential settlement and tilting of the piled raft foundation and the raft alone are also the same. So, based on the result, it can be concluded that inserting piles below the raft has no contribution for settlement reduction.

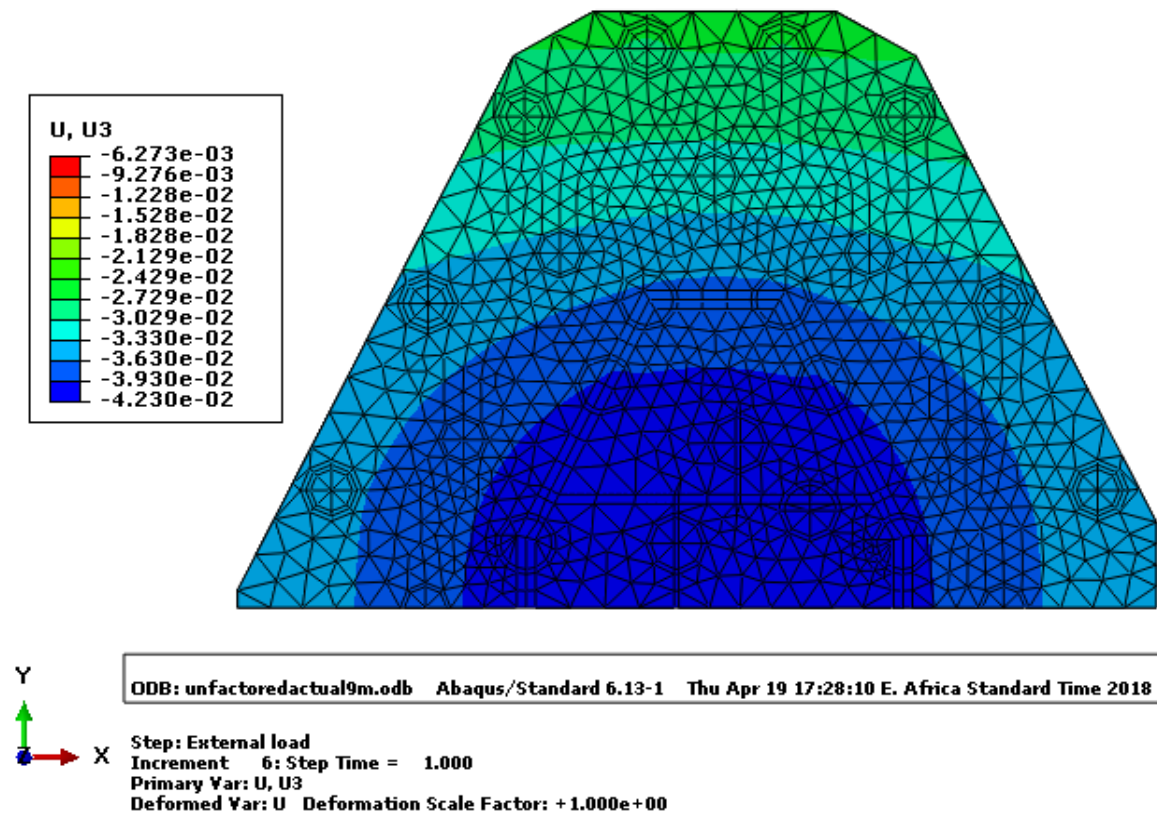


Figure 6.2: Contours of vertical displacement at full load for case 1 (units: m)

Table 6.2: Shows the differential settlement and tilting of the building

Maximum settlement [mm]	Minimum settlement [mm]	Differential settlement [mm]	Tilting
42.3	27	15.3	0.00059

Axial load distributions are plotted in Figure 6.3. The maximum axial load is at the top of the pile; it reduces with depth, and minimum at the tip of the pile. The axial load carried by the piles at the centre of the raft is very small compared to the piles around the edges. Similar behaviour was also reported by (Maharaj 2004). This is due to pile group effects. Central pile became less stiff and the axial load carried by it will be small as compared to the piles around the edges.

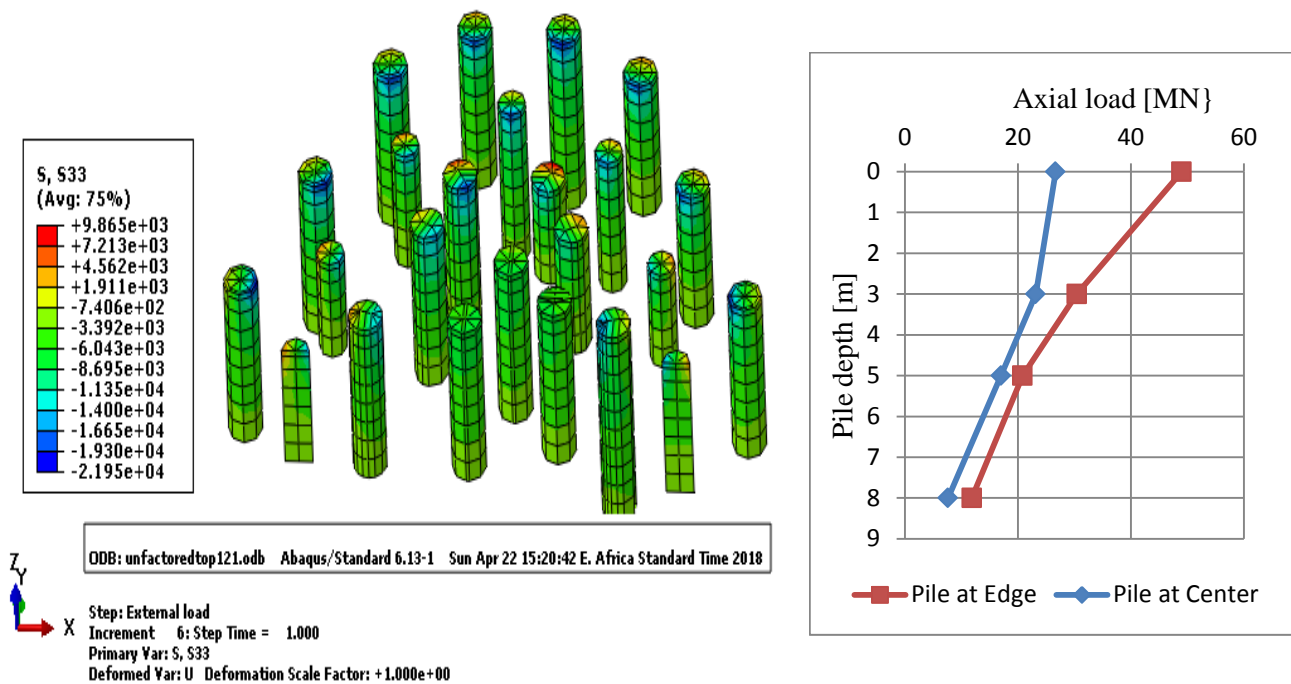


Figure 6.3: Axial load distribution of piles

Case 2: Using the average length of the piles

Settlement contours of the raft are shown in Figure 6.4. The results of the analysis shows maximum and minimum settlements of the raft are 3.9 cm and 2.85 cm respectively as shown in Table 6.3. This gives 1.05 cm maximum differential settlement and 1/2439 tilting of the building. These results are similar with the results of case 1. Therefore the simplification made for the soil formation has no significant effect on settlements of the foundation.

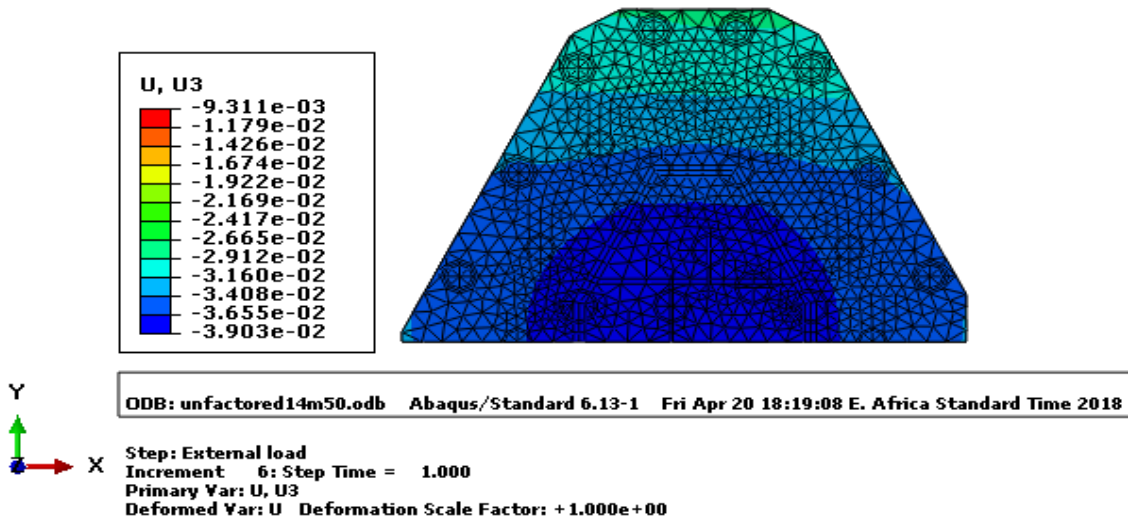


Figure 6.4: Contours of vertical displacement at full load for case 2 (units: m)

Table 6.3: Shows the differential settlement and tilting of the building

Maximum settlement [mm]	Minimum settlement [mm]	Differential settlement [mm]	Tilting
39.03	28.5	10.53	0.00041

As mentioned earlier, in the model some piles were not extended up to the hard stratum. To see the effect of soil layer at pile tips on axial load distribution one typical pile location has been taken for comparison. Figure 6.5 shows axial load distribution of the pile when soil layer at pile tip is varied. As shown in the figure axial load distribution of the pile is almost twice when its tip is resting on the hard stratum as compared when its tip is resting on a weak stratum.

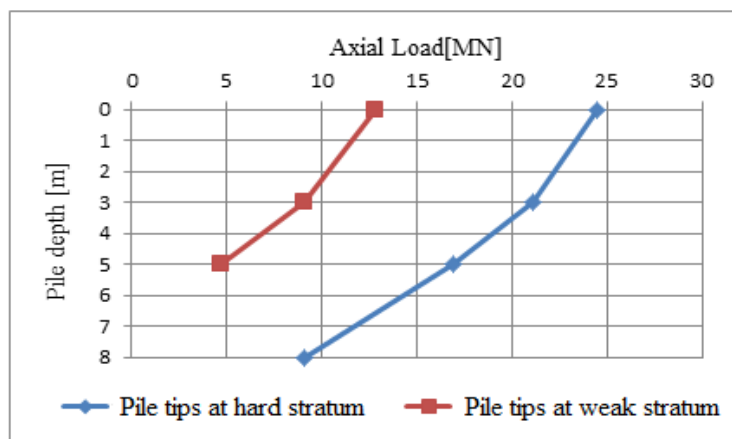


Figure 6.5: Axial load distribution of pile with pile tips on different soil layers

Figure 6.6 shows the typical load-settlement curves for a selected key node of a piled raft foundation unit. As expected, the settlement increased with the increasing load levels. Unlike the case of classical foundations with group of piles, no sudden increase of the settlements without increasing load is observed. It can be seen from the figure that the graph is almost linear. This reveals that the stiffness of the foundation is governed by the piles alone and the ultimate capacity of the piles also has not been utilized yet.

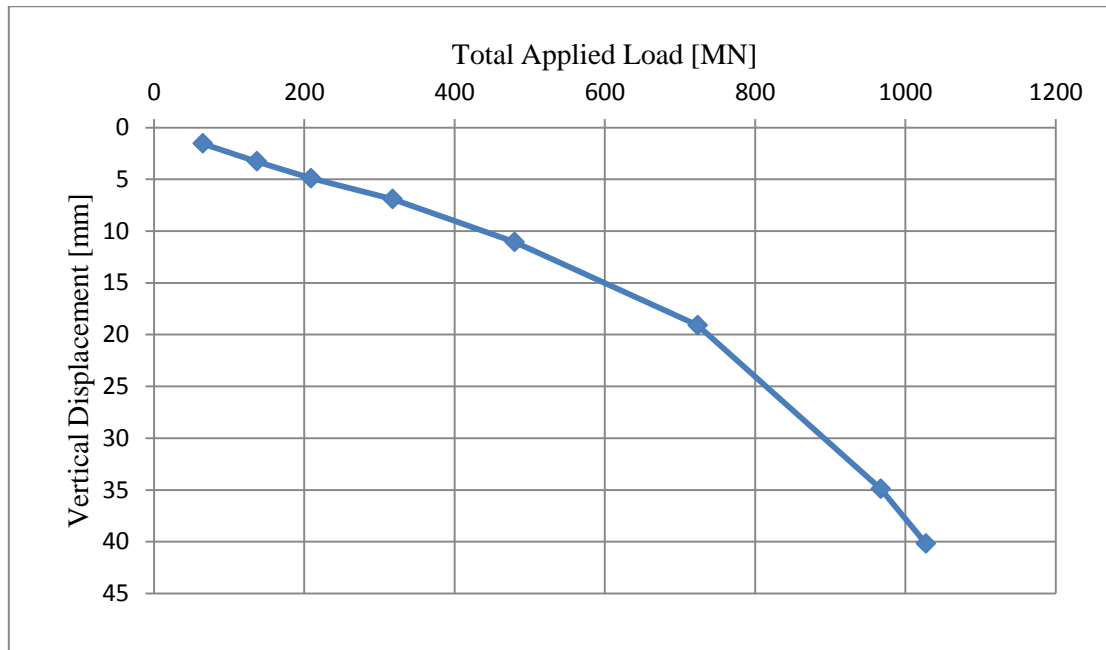


Figure 6.6 Load displacement behaviour of the piled raft foundation

6.4 Load sharing behaviour of piled raft foundation

The percentage of load taken by each component (raft & piles) depends on a number of factors such as soil property, raft geometry, pile geometry (diameter, length) and its spacing. In this study investigation was made for the actual soil properties of the site and actual geometry of the foundation system. Table 6.3 shows the computed settlement of the piled raft foundation, total gravity load transferred from the superstructure to the raft and the load sharing ratios between piles and raft.

Table 6.4: Shows load sharing behaviour between piles and raft with applied vertical load

Percentage of the total load in (%)	Load applied on the raft [MN]	Load shared by the piles[MN]	Load sharing ratio of piles (α_{pr})in %	Load sharing ratio of raft (X) in %	Total settlement [mm]
10	160.8	176	100	0	5.99
20	256.86	295	100	0	9.5
35	400.8	465	98.3	1.7	14.8
65	688.8	754	95.2	4.8	26.23
80	832.8	880	93.6	6.4	33.82
95	976.8	1030	93.1	6.9	36.49
100	1024.8	1090	92.9	7.1	42.3

As we can see from Table 6.4 at the total superstructure load, load sharing ratio by the piles, α_{pr} is 92.9% for case 1(using actual pile length). For Case 2(average pile length) α_{pr} gives 96%.When results of the FE method is compared with the simplified method, the simplified method underestimate the load shared by the piles. It gives 84.36% of the total load transferred by the piles. This is due to many simplifications were made in simplified method.

The piled raft coefficient was not only dependent on the system geometries of the foundation but also on the load level for most cases. The load sharing between the raft and the piles as a function of the total applied load reported in Figure 6.7 shows a significant reduction of the total load carried by the piles with increasing load level. Similar behaviour was reported by (Katzenbach 2013). At small load level almost the entire superstructure load is carried by the piles (horizontal portion of the graph) shown in the figure. When the load of superstructure is small, the settlement of the foundation is also small; at this stage load shared by the raft is insignificant or small because interaction between the raft and the soil is negligible. When the magnitude of the load increases load carried by the piles decreases and became almost constant as shown in the figure. At this stage the ultimate capacities of the piles are reached after that if there is any additional load it will be taken by the raft. When total load of the structure increases settlement of the building also increase, as a result of the raft – soil interaction, the contact pressure below the raft increase. At this stage, the load sharing ratio by the piles decreases and load sharing ratio of the raft increase. Generally as shown in the figure the load shared by the piles is greater than 93%, this shows almost all the superstructure load are transferred by the piles. This is due to tip of the piles is resting on the

medium strong rock; the entire load is transferred to the rock through the piles. In addition, the settlement of the foundation is also very small, as a result the raft – soil interaction has no significant effect and load shared by the raft is negligible.

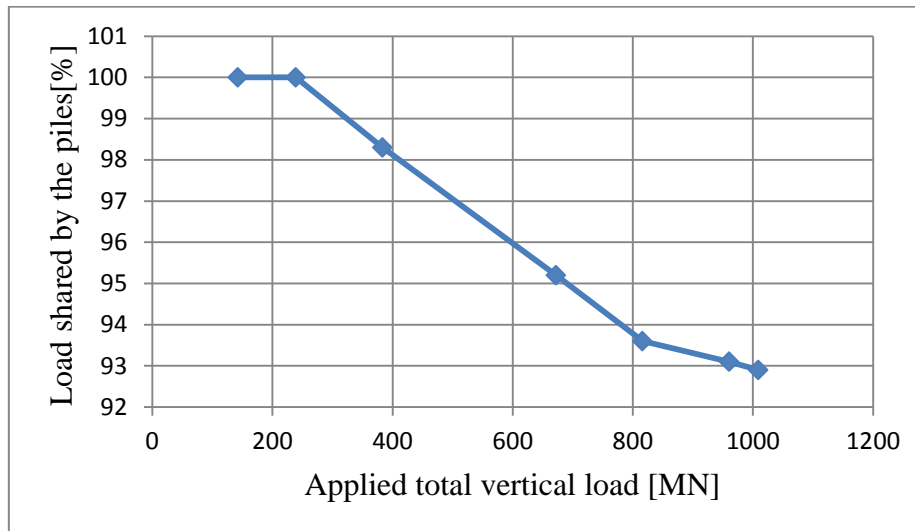


Figure 6.7 Load sharing behaviour with applied vertical load

According to piled raft foundation guideline (Jean-Louis n.d.), the application of piled raft foundation is effective in cases where the soil under the raft is not very weak in comparison to that under the pile tip. Thus it is common to define the dimensionless stiffness ratio $E_{\text{soil layer at pile tips}} / E_{\text{top layer}}$ as the ratio of stiffness of the soil layers at the tip of the piles to that at the top of piles or just under the raft. The guideline restricts the use of Piled raft foundation for the range of $E_{\text{soil layer at pile tips}} / E_{\text{top layer}}$ greater than 10, because contribution of the raft is very small (piles raft coefficient α_{pr} is >0.9 to all cases). But in this study, the actual stiffness ratio of the soil layers at the tip of the piles to that at the top of piles or just under the raft ($E_{\text{soil layer at pile tips}} / E_{\text{top layer}}$) is 8.26/1. For this stiffness ratio load shared by the piles α_{pr} is 92.9%. This value is nearly acceptable as compared with the recommendation given by the piled raft guideline.

To apply the recommendation given by the piled raft foundation guideline for stratified soil condition, in case of pile tips resting on hard stratum, a parametric study was carried out. Six different stiffness ratios $E_{\text{soil layer at pile tips}} / E_{\text{top layer}}$ were analysed. These ratios are three above the limit value (10/1) and three below the limit value. Only the stiffness of the weak layer (beneath raft) is varied keeping the stiffness of hard stratum (where the pile tips resting on).

Table 6.5 shows the analysed cases of different stiffness ratio with load sharing ratio by the piles.

Table 6.5: Load sharing behaviour of the foundation with different stiffness ratios

S.N	Stiffness ratio = $E_{\text{soil layer at pile tips}} / E_{\text{top layer}}$	Load shared by the piles α_{pr} in %
1	25	100
2	20	99.8
3	12.5	96.5
4	8.33	92.25
5	2	86.3
6	1.25	84

The effects of the stiffness of the top weak layer on load sharing ratio of the piles have been studied using the stiffness ratio-versus-load shared by the piles plots shown in Figures 6.8. For lower stiffness ratios ($E_{\text{soil layer at pile tips}} / E_{\text{top layer}} < 10$), the percentage of load carried by the piles is increasing linearly with increasing $E_{\text{soil layer at pile tips}} / E_{\text{top layer}}$ as shown in the figures and becomes 100% for $E_{\text{soil layer at pile tips}} / E_{\text{top layer}}$ values greater than 20.

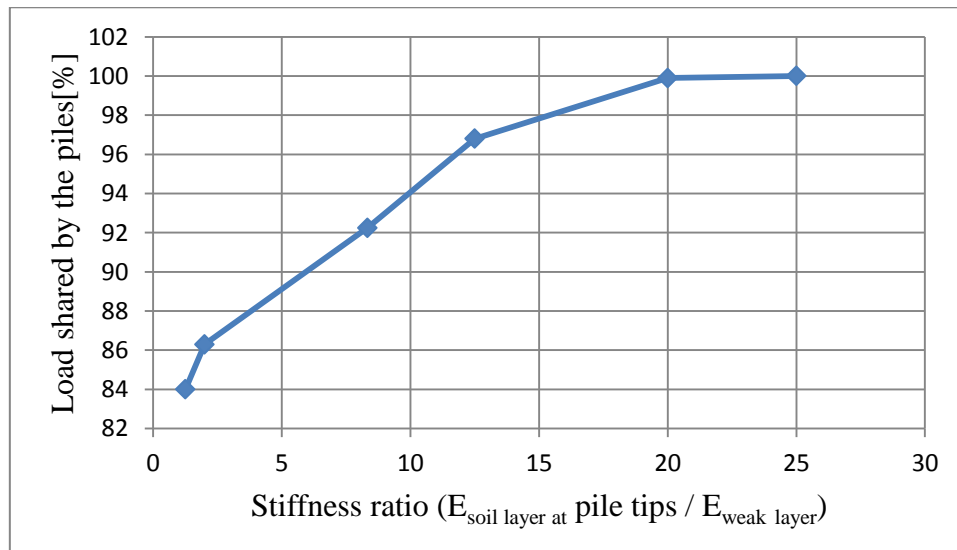


Figure 6.8: Shows the effects of stiffness ratio on load sharing behaviour of pile

6.5 Effect of raft – soil interaction on behaviour of piled raft foundation

a) Effect of raft – soil interaction on settlements of the building

In order to see the effect of raft- soil interaction on an individual group of piles behaviour, the piles with raft have been analysed with the same material properties of soil and pile as

considered in the piled raft foundation. The same discretisation has been used; only the raft - soil interaction was ignored (the contact between raft and soil is not allowed). In the analysis, magnitude and way of application of the loads on the raft is the same as in the piled raft foundation analysis have been applied (with full contact between raft and soil).

Figure 6.9 shows contours of settlement of the raft at total working load. The positions of the maximum and minimum settlements are slightly altered from their location. The maximum settlement is shifted from centre point. It is located almost on the corner of symmetry line shown in the figure. The results of the analysis shows maximum and minimum settlements of the raft are 4.28 cm and 2.72 cm respectively as shown in Table 6.6. The maximum differential settlement and tilting of the whole building is 1.56 cm and 0.00061 respectively. It should be noted that although the raft – soil interaction is ignored in the model, the maximum differential settlement and tilting of the building are similar with the results obtained by considering the raft-soil interaction. It can be conclude that the raft soil interaction has no effect on the foundation behaviour. As mentioned earlier the pile tips are located on the medium strong layer, as a result the settlement of the foundation became small. Due to this reason the raft – soil interaction is not active.

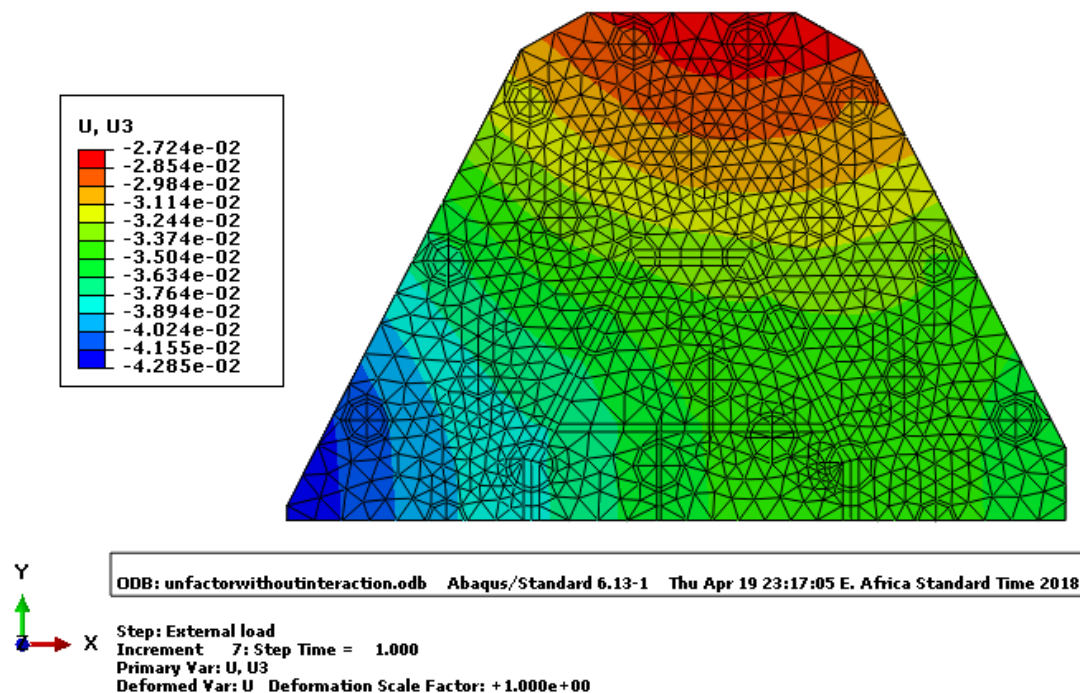


Figure 6.9: Contours of vertical displacement at full load without raft soil interaction (Units: m)

Table 6.6: Shows the differential settlement and tilting of the building without raft soil interaction

Maximum settlement [mm]	Minimum settlement [mm]	Differential settlement [mm]	Tilting
42.8	27.2	15.6	0.00061

b) Effect of raft – soil interaction on axial load distribution of piles

Here we cannot talk about load sharing mechanism; since the contact between raft and soil was ignored all loads are carried by the pile groups. To see the effect of raft – soil interaction on axial load distribution of piles, the two previously selected piles (one from centre and one from edge) were chosen. The comparison of axial load distribution between these piles is also plotted in Figure 6.10. As expected, the maximum axial load is at the top of the pile; it reduces with depth, and minimum at the tip of the pile. The axial load carried by both piles is almost the same both in their magnitude and their pattern. This shows that the raft – soil interaction has no significant effect on the axial load distribution of the piles.

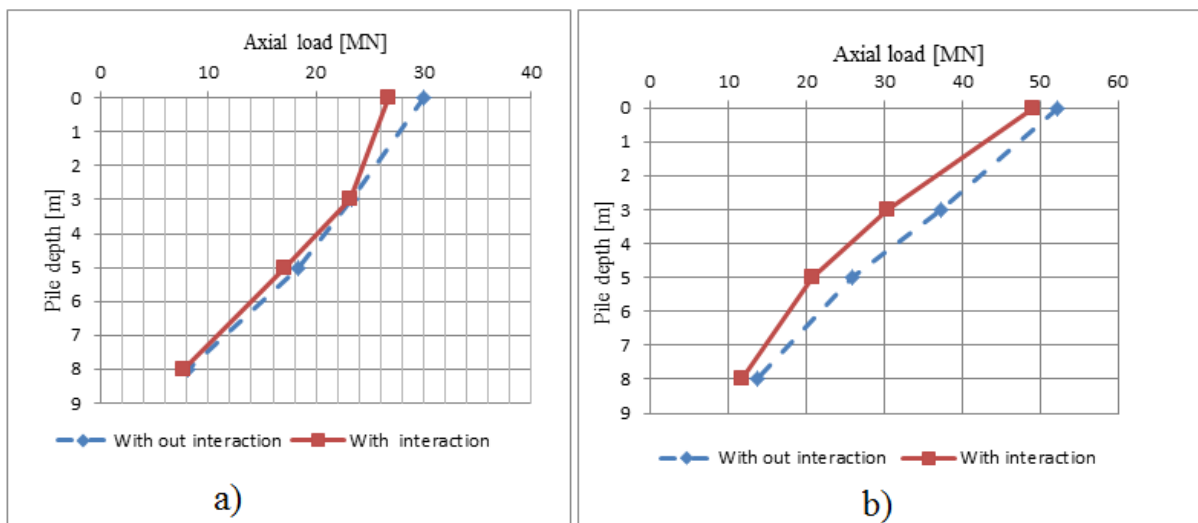


Figure 6.10: Axial load distributions of piles a) Central piles b) Edge piles

c) Effect of raft – soil interaction on load - settlement behaviour of the building

Fig 6.11 shows the typical load-settlement curves for the selected key node of a piled raft foundation unit with and without considering the raft – soil interaction. As it can be shown in

the graph both curves exhibit the same stiffness. This indicated that the raft – soil interaction has no significant contribution.

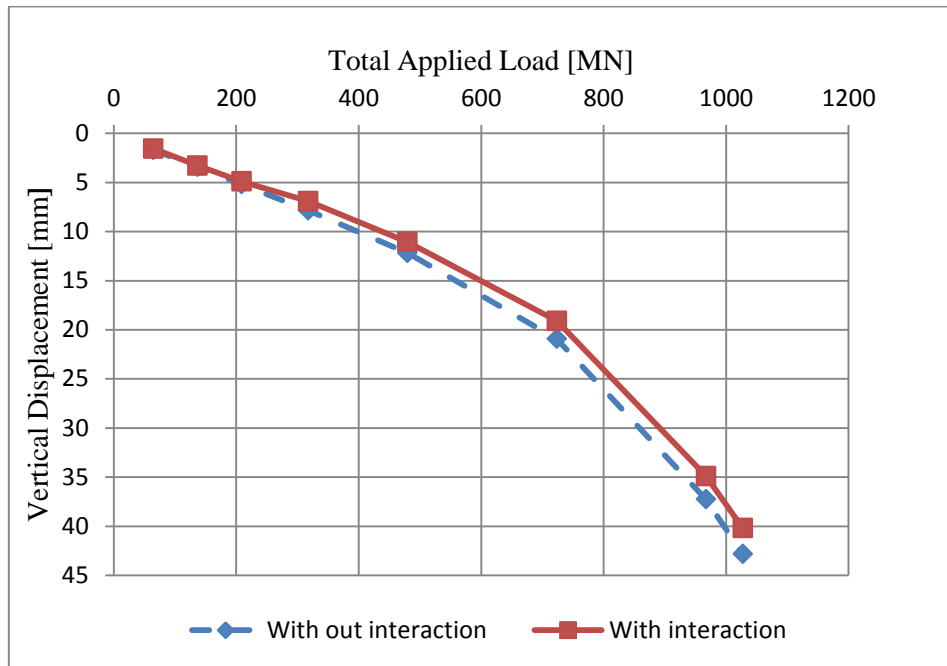


Figure 6.11 Effects of raft – soil interaction on load displacement behaviour of the piled raft foundation

6.6 Effect of weak layer on behaviour of piled raft foundation

This section deals with an extensive study of the influence of weak layer, located under hard strata below pile tips, on the settlements and load sharing behaviours of the piled raft foundation. To investigate the effect of this weak layer on the settlements and load sharing behaviour of the piled raft foundation, the analysis was carried out by varying the following two parameters.

- By varying location of the weak layer keeping all other parameters constant
- By varying stiffness of this weak layer keeping all other parameters constant

In addition to its realistic location of this weak layer, five different locations are considered in the analysis. These are, one at the pile tips, four below pile tips (below 1 m, 3 m, 6m and 14 m). In addition to its actual stiffness this weak layer, four different stiffness ratio of the weak layer to stiffness of hard strata ($E_{\text{weak layer}} / E_{\text{soil layer at pile tips}}$) is considered in the model.

Effect of the weak layer on settlements of the building

Contour plots of the raft settlements show that the positions of the maximum and minimum settlements are not considerably altered by stiffness variation. The effects of the stiffness of the weak layer on maximum and differential settlements of the foundation have been studied using the stiffness ratio-versus-settlement plots shown in Figure 6.12. For lower stiffness ratios ($E_{\text{weak layer}} / E_{\text{soil layer at pile tips}} < 0.2$), both the maximum and differential settlements are decreasing abruptly with increasing $E_{\text{weak layer}} / E_{\text{soil layer at pile tips}}$ as shown in the figures. This shows that, a slight reduction of the soil stiffness in the weaker range results in an excessive increase in maximum and differential settlements of the building. The influence of the weak layer, which located below pile tips, on the maximum and differential settlements decreases with increasing $E_{\text{weak layer}} / E_{\text{soil layer at pile tips}}$ and becomes insignificant for $E_{\text{weak layer}} / E_{\text{soil layer at pile tips}}$ values greater than 0.5 for all locations of the weak layer.

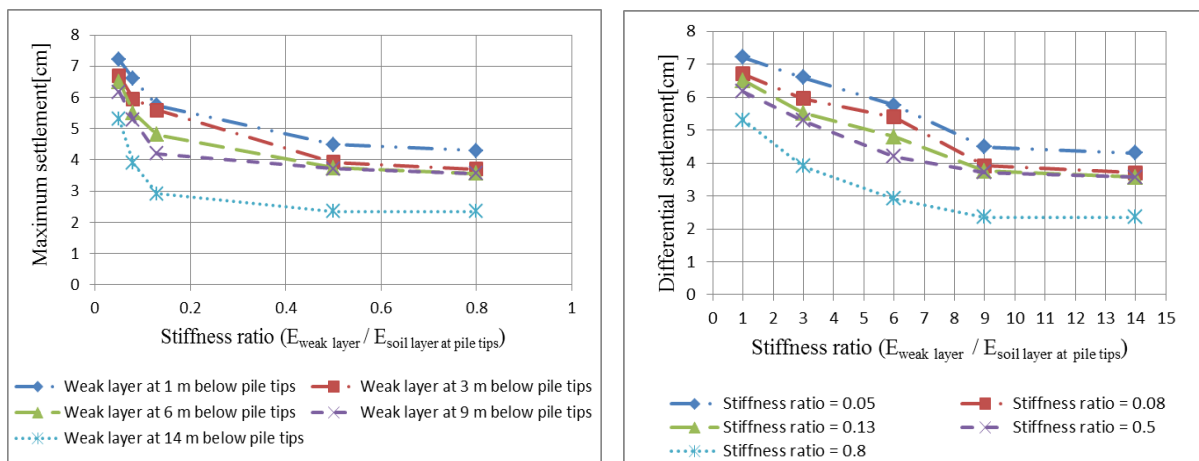


Figure 6.12: Shows the effects of stiffness ratios on maximum and differential settlements of the building

The effects of the locations of the weak layer on maximum and differential settlements of the foundation have been studied using the location of the weak layer -versus-settlements plots shown in Figures 6.13. As we can see from the graph both total settlement and differential settlement are approaching to the same value when the stiffness ratio ($E_{\text{weak layer}} / E_{\text{soil layer at pile tips}} > 0.5$). In addition the locations of the weak layer have negligible effects on maximum and differential settlements when the weak stratum is located below 9 m for all stiffness ratios as shown in the figures. It can be said that the effect of weak layer on the piled raft behaviour is negligible even its location is below 1 m for relative stiffness between the weak and hard strata is greater than 0.5.

For all stiffness ratios (0.05 to 0.8) considered in this study the behaviour piled raft is not affected if the weak stratum is located below 9 m.

The effect of this weak layer clearly showed on total and differential settlement of the foundation. Location of the weak layer has more effect on settlement rather than the relative stiffness of weak to hard strata.

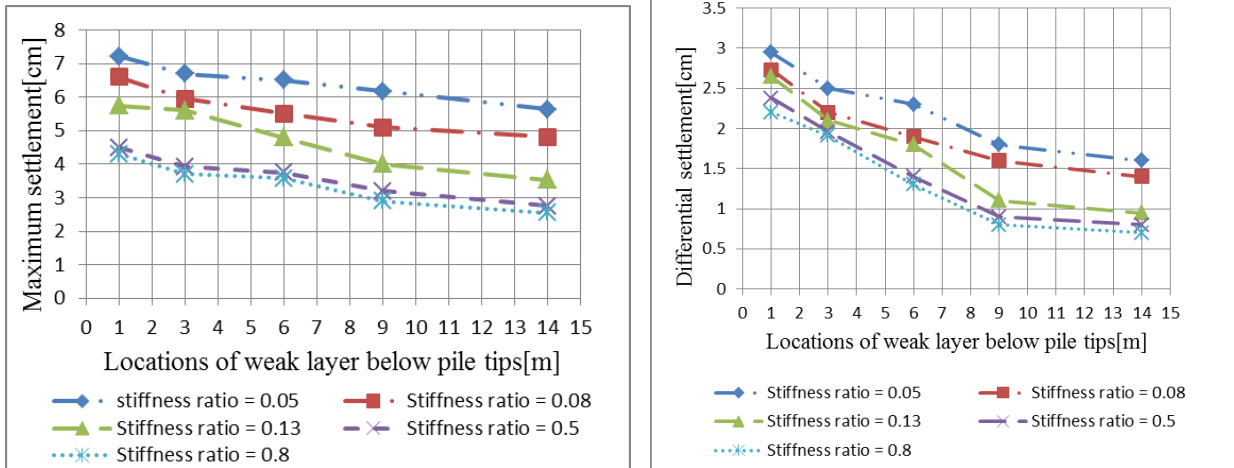


Figure 6.13: Shows maximum and differential settlements of the building with location of the weak layer under pile tips

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The main objective of this thesis is to study the load sharing behaviour of piled raft foundation considering the effect of weak layer beneath the raft and another weak layer located under the hard stratum below pile tips using Finite Element Method. The main conclusions and recommendations drawn through the numerical modelling of piled raft foundation developed in this study are summarized in this chapter.

This paper has illustrated the process of design of a piled raft foundation using two stage procedures, in the first stage an initial assessment of piled raft foundation was carried out using simplified method , and in the second stage a detailed design of the foundation and assessing its behaviour was carried out using finite element method.

In this study a numerical model was developed using ABAQUS software to analyse piled-raft foundations as three dimensional problems. The model, accounts for the effect of interaction factors among pile, raft and soil. The model was validated by comparing its results with the results of other numerical models available in the literature. The results of this numerical model were found in reasonable agreement with the results of other numerical models available in the literature.

The results of the analyses show that although the foundation is primarily designed as a pile group, the raft alone can carry the superstructure load with allowable settlements. But when the raft is analysed with 46 piles, the raft contribution became insignificant. Only 7% of the total load is transferred by the raft .When this result is compared with simplified method; it is observed that the simplified method analysis gives the higher values of load shared by the raft. It gives 14.6% of the total load is transfer directly to soil by the raft.

The most important observations regarding the load sharing and load-settlement behaviour of piled-raft foundations can be summarized as follows:

- Based on the result, the raft alone is adequate both in bearing capacity and settlement requirements.

- It can be concluded that inserting piles below the raft has no contribution for settlement reduction if the pile tips resting on hard stratum.
- The raft has no significant effect both in load sharing and settlement of the piled raft foundation in case of pile tips resting on hard stratum.
- Based on the result, it can be concluded that the effect of weak layer located under hard stratum below pile tips on the piled raft behaviour is negligible for relative stiffness between the weak and hard stratum is greater than 0.5. For all stiffness ratios (0.05 to 0.8) considered in this study, the piled raft behaviour not affected if the weak stratum is located below 9 m.
- The load sharing between the raft and the piles depends not only on the ratio of the stiffness of the piles to that of the raft but also depends on the settlement of the foundations. The load carried by the raft increases significantly when the settlement of the raft increases.
- In this thesis, the result of load shared by the raft is slightly agreed with the recommendation given by the piled raft foundation guideline. The guideline states that If the stiffness ratio of top layer to bottom layer (soil layer at pile tip) is ≤ 0.1 the load shared by the piles higher (it is $> 90\%$).

RECOMMENDATIONS

- The boundary condition (shoring wall) of the actual foundations is irregular and unsymmetrical. The shear wall is located around 80 m from centreline of the raft in one direction and around 10 m in the other direction. In this study, the analysis was carried out by taking the longest side and assuming symmetrical condition. So it is recommended to extend this study by carrying out full three-dimensional analysis to account for effects of boundary condition on behaviour of the piled-raft foundations.
- Most of soil parameters required for numerical analysis used in this study are obtained from different literatures. Only very limited parameters are directly taken from test result. This is due to lack of adequate laboratory and field test results for the different soil layers at different depth. This can be raised as one significant drawback in this study. A better simulation result of FEM could have been obtained, if most soil parameters have been directly determined from laboratory and field test results.
- The actual geometry of the piles is enlarged at their tips. But In this study the enlarged dimension at pile tips is not considered. So it is recommended to investigate

the effect of belled pier in terms of foundation stiffness or the load sharing of piled-raft foundations.

- In this thesis the load sharing mechanism of piled raft in layered soil was investigated where the weak layer is located beneath raft and hard stratum at pile tips. It is recommended to investigate the load sharing mechanism of piled raft foundation where the hard stratum is located beneath the raft and weak layer at pile tips.

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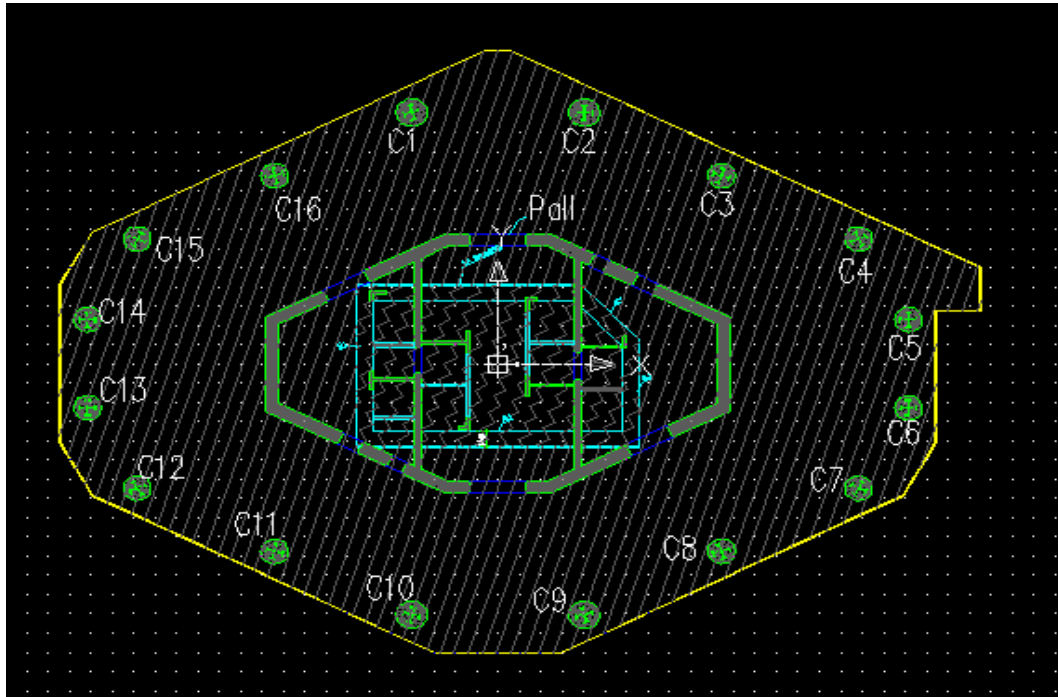
APPENDIX - A

Super structural loads and pile lengths

Super structural loads

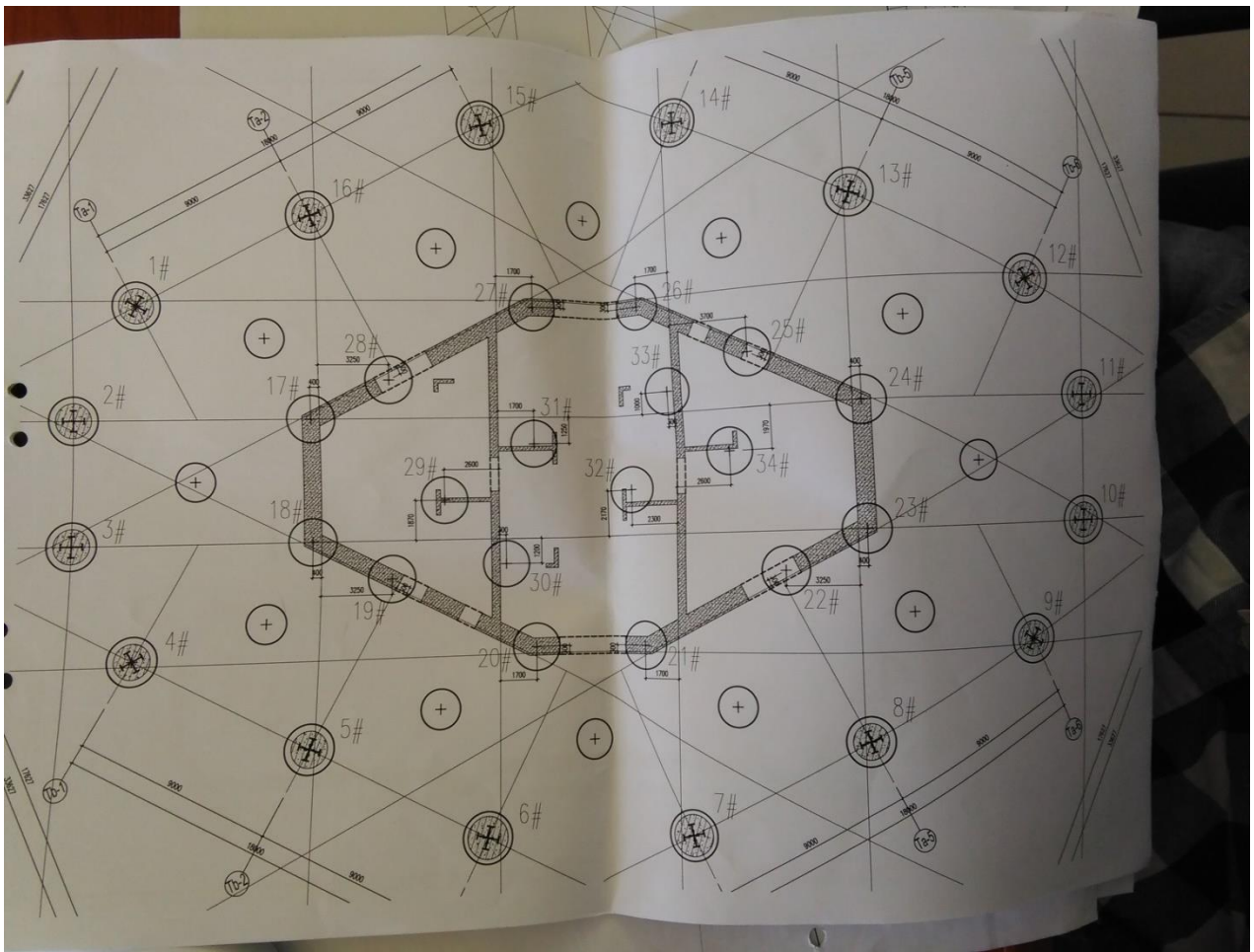
All Loads are in kN		
C2 = 35243.293		C1 = 35863
C3 = 31995.307		C16 = 32757.6
C4 = 28138.643		C15 = 29990.6
C5 = 25829.107		C14 = 27084.5
C6 = 25572.643		C13 = 26431.5
C7 = 25200.329		C12 = 25560.5
C8 = 28199.214		C11 = 30206.3
C9 = 32276.236		C10 = 34430.9
`Pall = 487884.7		
Note : C = Column load Pall = Shear wall load		

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD



Pile lengths

Pile No.	Pile length	Pile No.	Pile length	Pile No.	Pile length	Pile No.	Pile length
3	8.5 m	9	7.1 m	14	7.4 m	43	7 m
4	7.7 m	10	5.7 m	15	7.5 m	46	7 m
5	8.3 m	20	8.2 m	16	6.6 m	11	4.4 m
17	5.3 m	21	7.8 m	26	6.7 m	12	4.3 m
18	7.9 m	22	5.5 m	29	7.8 m	27	9.8 m
19	8.3 m	23	6.6 m	30	7.6 m	32	7.6 m
28	5.8 m	24	5.6 m	31	7.9 m	37	4.3 m
1	6.5 m	39	6.5 m	33	7.5 m	38	5.7 m
13	7 m.1	40	4.8 m	34	7.8 m	44	7 m
25	7.7 m	41	6.4 m	25	7.5 m	45	4.5 m
7	6.5 m	2	9.3 m	36	6.3 m		
8	5.6 m	6	8 m	42	7.4 m		



APPENDIX - B

Bore hole loge data

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD

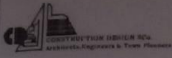
CONSTRUCTION DESIGN SCo. <small>Architectural, Engineering & Cost Planning</small>		Company Name CONSTRUCTION DESIGN SCo.	FORM NO OF/CDSCO/104							
Title Borehole Log Sheet		Issue No 1	Page No Page 1 of 5							
PROJECT NEW HEAD QUARTERS BUILDING (CBE) BORING TYPE ROTARY CORING			BH 6							
LOCATION ADDIS ABABA		GROUND WATER LEVEL 5.50m								
CLIENT CHINA STATE CONSTRUCTION ENGINEERING Co. LTD		BH ELEVATION 2353m								
DATE STARTED 25/06/2015		INCLINATION VERTICAL								
DATE COMPLETED 20/07/2015		GOORDINATES 0473012E 0996695N								
DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. 'N' VALUE	DEPTH (m)	PROFILE	T.C.R (%)	R.Q.D (%)	STRATA DESCRIPTION	REMARK
0	116	116			0.00		100	100	Loose to medium dense, variegated poorly graded fill material. It is mainly composed of varying proportions of demolished materials (concrete and brick gravel) sand, silt and clay	
1					1.00		100	100		
2					2.20		100	100		
3					3.80		100	100	Medium dense, yellowish brown silty SAND	
4					4.80		100	100	Soft, dark brown to grey organic silty CLAY	
5					5.40		100	100		
6					5.60		100	100	Weak, grey, highly weathered BASALT. Occasionally, fresh or discolored rock is present	
7					6.00		100	100		
8					7.00		100	100		
9					7.40		100	100		
10					8.20		100	100	Medium strong, intensively fractured and fragmented slightly to moderately weathered BASALT. Fracture surfaces are rough and yellowish stained. Joints are nearly horizontal and Filled with altered materials	
11					8.30		100	100		
12					8.20		100	100		
13					9.00		100	28		
14					10.30		100	100		
15					11.20		100	100		
					11.70		100	100	Very weak, pinkish to brownish, friable, highly to completely weathered scoriaceous BASALT. It is locally decomposed into silty CLAY. This layered is considered as an aquifer	
					12.80		100	100		
					13.00		100	100		
					14.60		100	100		
					15.00		100	100		

(Nc) CONE PENETRATION TEST
 N BLOWS/30cm
 RS ROCK SAMPLE
 W WATER SAMPLE
 TCR TOTAL CORE RECOVERY
 RQD ROCK QUALITY DESIGNATION
 [] DISTURBED SOIL SAMPLE
 [] UNDISTURBED SOIL SAMPLE
 SPT STANDARD PENETRATION
 [] END OF DRILLING


CREW **Zewudu Germa** DRAWN BY **Bezunesh W/Tsadik**
 SUPERVISOR **Berhanu Beyene** SIG
 LOGGED BY **Lamesgen Melese** SIG
 APPROVED BY **Matewos Bekele** SIG

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD

		Company Name CONSTRUCTION DESIGN SCo.	FORM NO OF/CDSCO/104							
Title Borehole Log Sheet		Issue No 1	Page No Page 2 of 5							
PROJECT NEW HEAD QUARTERS BUILDING (CBE) BORING TYPE, ROTARY CORING LOCATION ADDIS ABABA GROUND WATER LEVEL 5.50m CLIENT CHINA STATE CONSTRUCTION ENGINEERING Co. LTD BH ELEVATION 2353m DATE STARTED 25/06/2015 INCLINATION VERTICAL DATE COMPLETED 20/07/2015 COORDINATES 0473012E 0996695N			BH 6							
DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. / N VALUE/	DEPTH (m)	PROFILE	TCR (%)	ROD (%)	STRATA DESCRIPTION	REMARK
15					15.00					
16					15.70					
17					16.30					
18					17.00					
19					17.50					
20					17.80					
21					18.20					
22					18.60					
23					19.00					
24					19.80					
25					20.20					
26					20.50					
27					21.40					
28					22.00					
29					22.40					
30					22.80					
					23.50					
					25.00					
					26.30					
					27.50					
					28.30					
					29.10					
					30.00					
									Weak, grey, highly weathered BASALT. Occasionally, fresh or discolored rock is present	
									Medium strong, intensively fractured and fragmented slightly to moderately weathered BASALT. Fracture surfaces are rough and yellowish stained. Joints are nearly horizontal and Filled with altered materials	
									Dense brick red, silty SAND/sandy SILT with clay	
									Stiff, brown, clayey SILT with sand (Palcosol)	
									Weak to very weak, pincish grey, highly to completely weathered scoriaceous BASALT, It is often mixed with dark grey, swelling silty CLAY (Fault gouge)	
(N)	CONE PENETRATION TEST									
N	BLOWS/30cm									
RS	ROCK SAMPLE									
W	WATER SAMPLE									
TCR	TOTAL CORE RECOVERY									
RQD	ROCK QUALITY DESIGNATION									
DS	DISTURBED SOIL SAMPLE									
US	UNDISTURBED SOIL SAMPLE									
SPT	STANDARD PENETRATION									
END	END OF DRILLING									
					CREW Zewudu Germa		DRAWN BY Bezunesh W/Tsadik			
					SUPERVISOR Berhanu Beyene.		SIG <i>[Signature]</i>			
					LOGGED BY Lamesgen Melese		SIG <i>[Signature]</i>			
					APPROVED BY Matewos Bekele		SIG <i>[Signature]</i>			
PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE										

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD

		Company Name CONSTRUCTION DESIGN Co. OF/CDSCO/104	
Title Borehole Log Sheet		Issue No 1	Page No Page 3 of 5


PROJECT NEW HEAD QUARTERS BUILDING (CBE) BORING TYPE ROTARY CORING		BH 6
LOCATION ADDIS ABABA	GROUND WATER LEVEL 5.50m	
CLIENT CHINA STATE CONSTRUCTION ENGINEERING Co. LTD BH ELEVATION 2353m		
DATE STARTED 25/06/2015	INCLINATION VERTICAL	
DATE COMPLETED 20/07/2015	GOORDINATES 0473012E 0996695N	

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	SPT 'N' VALUE/	DEPTH (m)	PROFILE	TCR (%)	ROD (%)	STRATA DESCRIPTION	REMARK
30					30.40		100		Weak to very weak, pincish grey, highly to completely weathered scoriaceous BASALT, It is often mixed with dark grey, swelling silty CLAY (Fault gouge)	
31					30.70		100			
32					32.00		100		Medium strong, intensively fractured and fragmented slightly to moderately weathered BASALT. Fracture surfaces are rough and yellowish stained. Joints are nearly horizontal and Filled with altered materials	
33			RS		32.50		90	29		
34					33.50		89	0	Stiff, dark to light greg, swelling clayey SILT (Alternated Ash)	
35					35.50		92	0		
36					36.00		85	0	Weak to very weak, pincish grey, highly to completely weathered scoriaceous BASALT, It is often mixed with dark grey, swelling silty CLAY (Fault gouge)	
37					38.00		100			
38			W		38.20		100			
39					38.60		100			
40					39.50		100			
41					40.50		100			
42					41.80		100			
43					42.70		100			
44					43.00		100			
45					44.00		100			

(N) CONE PENETRATION TEST N BLOWS/30cm RS ROCK SAMPLE W WATER SAMPLE TCR TOTAL CORE RECOVERY RQD ROCK QUALITY DESIGNATION DS DISTURBED SOIL SAMPLE US UNDISTURBED SOIL SAMPLE SPT STANDARD PENETRATION END OF DRILLING	CREW <u>Zewudu Germa</u> DRAWN BY <u>Bezunesh W/Tsadik</u> SUPERVISOR <u>Berhanu Beyene.</u> SIG _____ LOGGED BY <u>Lamesgen Melese</u> SIG _____ APPROVED BY <u>Matewos Bekele</u> SIG _____
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PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

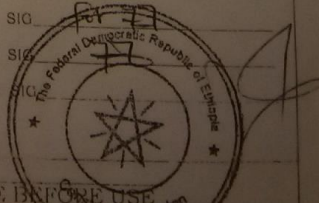
INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD

		Company Name CONSTRUCTION DESIGN SCo.	FORM NO OF/CDS/CO/104
Title Borehole Log Sheet		Issue No 1	Page No Page 4 of 5

PROJECT NEW HEAD QUARTERS BUILDING (CBE) BORING TYPE ROTARY CORING		BH 6
LOCATION ADDIS ABABA	GROUND WATER LEVEL 5.50m	
CLIENT CHINA STATE CONSTRUCTION ENGINEERING Co. LTD BH ELEVATION 2353m		
DATE STARTED 25/06/2015	INCLINATION VERTICAL	
DATE COMPLETED 20/07/2015		
GOORDINATES 0473012E 0996695N		







DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. / N-VALUE	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
45					45.00		100		Weak to very weak, pincish grey, highly to completely weathered scoriaceous BASALT, It is often mixed with dark grey, swelling silty CLAY (Fault gouge)	
46					45.20		100			
47					47.00		100		Medium strong, intensively fractured and fragmented slightly to moderately weathered BASALT. Fracture surfaces are rough and yellowish stained. Joints are nearly horizontal and Filled with altered materials	
48					47.60		100	50		
49			RS	18.60	48.60		100	50	Weak to very weak, pincish grey, highly to completely weathered scoriaceous BASALT, It is often mixed with dark grey, swelling silty CLAY (Fault gouge)	
50					49.40		100	30		
51					50.50		90	55	Medium strong, intensively fractured and fragmented slightly to moderately weathered BASALT. Fracture surfaces are rough and yellowish stained. Joints are nearly horizontal and Filled with altered materials	
52					51.50		91	65		
53					52.80		93	31	Strong, grey, slightly fractured, fresh to faintly weathered BASALT. Fracture surfaces are slightly rough, mostly sub-vertical to vertical and healed with cementing materials (calcite)	
54			RS	54.20	54.00		95	80		
55					56.00		90	72		
56			RS	56.40	56.00		90	72		
57					58.00		97	84		
58			RS	58.20	58.80		91	65		
59					58.80		91	65		
60										

(Nc) CONE PENETRATION TEST N BLOWS/30cm RS ROCK SAMPLE W WATER SAMPLE TCR TOTAL CORE RECOVERY RQD ROCK QUALITY DESIGNATION D DISTURBED SOIL SAMPLE S UNDISTURBED SOIL SAMPLE SPT STANDARD PENETRATION ~~~~~ END OF DRILLING	CREW Zewudu Germa DRAWN BY Bezunesh W/Tsadik SUPERVISOR Berhanu Beyene. SIG _____ LOGGED BY Lamesgen Melese SIG _____ APPROVED BY Matewos Bekele SIG _____
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PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD**

		Company Name CONSTRUCTION DESIGN SCo.	OF/CDSco/104							
Title Borehole Log Sheet		Issue No 1	Page No Page 5 of 5							
PROJECT NEW HEAD QUARTERS BUILDING (CBE) BORING TYPE <u>ROTARY CORING</u>			BH 6							
LOCATION <u>ADDIS ABABA</u>		GROUND WATER LEVEL <u>5.50m</u>								
CLIENT <u>CHINA STATE CONSTRUCTION ENGINEERING Co. LTD</u>										
DATE STARTED <u>25/06/2015</u>		INCLINATION <u>VERTICAL</u>								
DATE COMPLETED <u>20/07/2015</u>		GOORDINATES <u>0473012E 0996695N</u>								
DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. / R-VALUE /	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
60					60.00		100		Strong, grey, slightly fractured, fresh to faintly weathered BASALT. Fracture surfaces are slightly rough, mostly sub-vertical to vertical and healed with cementing materials (calcite)	
61		85			61.80		91	65		
62										
63										
64										
65										
66										
67										
68										
69										
70										
71										
72										
73										
74										
75										
(Nc) CONE PENETRATION TEST N BLOWS/30cm RS ROCK SAMPLE W WATER SAMPLE TCR TOTAL CORE RECOVERY RQD ROCK QUALITY DESIGNATION  DISTURBED SOIL SAMPLE  UNDISTURBED SOIL SAMPLE SPT STANDARD PENETRATION  END OF DRILLING					CREW <u>Zewudu Germa</u> DRAWN BY <u>Bezuncsh W/Tsadiq</u> SUPERVISOR <u>Berhanu Beyene.</u> SIG LOGGED BY <u>Lamesgen Melese</u> SIG APPROVED BY <u>Matewos Bekele</u> SIG					
PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEING USED										

**INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD**

	Company Name CONSTRUCTION DESIGN SHARE CO.	Form No Of/CDSCo./117
LABORATORY TEST RESULT		Issue No Page No

Pro. N° = TB-044/2007
Date = 21/07/2015

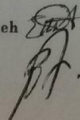
Project :- Head Quarter Building Of CBE
Client :- China State Construction Engineering Ethiopia PLC
Location :- A.A (Ambasader)
Object :- Nineteen Undisturbed Soil Samples

N°	BH N°	Depth (m)	Atterberg Limit			Free Swell (%)	Organic Content (%)	Swelling Pressure (KN/m2)	Natural Moisture Content (%)	Natural Unit Weight (Kg/m3)	Unconfined Compressive Strength (KN/m²)	Triaxial Shear Test		Direct Shear Test	
			LL (%)	PL (%)	PI (%)							C (KN/m²)	Ø Degree	C (KN/m²)	Ø Degree
1	1	4.50	-	-	-	-	-	30.50	1670	64	-	-	14	21	
2	1	18.60	75.48	49.35	26.13	-	-	54.22	1519	-	12	19	-	-	
3	1	43.00	109.17	48.59	60.58	220	-	20.11	1643	-	-	-	19	17	
4	3	5.50	62.09	25.82	36.27	70	-	18.07	1550	-	-	-	17	20	
5	5	6.40	83.06	43.89	39.17	170	-	480	26.17	1876	144	-	-	36	25
6	5	22.00	144.50	49.99	94.51	240	-	30.69	1774	61	-	-	28	7	
7	6	5.00	70.94	33.45	37.49	120	-	260	36.18	1889	130	-	-	30	10
8	6	20.50	119.50	52.25	67.25	-	-	-	64.25	1444	-	90	8	29	25
9	6	38.20	-	-	-	-	-	-	31.45	1658	241	94	5	-	-
10	6	41.00	-	-	-	-	-	-	36.98	1590	254	-	-	26	17
11	6	42.00	-	-	-	-	-	-	30.92	1962	265	84	15	-	-
12	7	3.00	76.03	38.91	37.12	110	-	-	20.50	1823	-	-	-	33	25
13	7	38.50	-	-	-	-	-	-	31.13	1543	386	-	-	20	18
14	7	39.50	77.51	28.25	49.26	140	8.28	-	39.02	1645	-	-	-	30	23
15	7	41.50	88.86	35.30	53.56	-	-	-	39.61	1747	-	85	16	-	-
16	8	40.00	84.09	34.66	49.43	-	-	-	33.36	1740	-	31	12	-	-
17	8	47.50	64.64	21.51	43.13	110	4.20	-	34.20	1790	-	-	-	32	13
18	9	19.00	93.55	55.49	38.06	-	-	-	80.27	1459	-	-	-	1	20
19	9	3.20	-	-	-	-	-	-	35.69	1649	132	-	-	22	16

Note :-

1. Nineteen graphs for grain size distribution test result are drawn and attached herewith.
2. Fourteen graphs for direct shear test result are drawn and attached herewith.
3. Six graph for UU triaxial test result is drawn and attached here with.
4. Eight Time - settlement, void ratio- pressure curve and the necessary data sheets for one - dimensional consolidation test result are attached herewith

Tested by :- Emebet Workneh
Date :- 02/08/2015
Checked by :- Abate Legesse
Date :- 25/08/2015




Approved by :- Biruk Abdi
Date :- 25/08/2015




PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD**


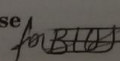
	Company Name	CONSTRUCTION DESIGN SHARE CO.	Form No	OF/CDS Co./117
	LABORATORY TEST RESULT			Issue NO


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Date = 22/09/2015

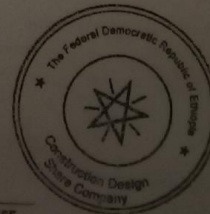
Project :- Head Quarter Building Of CBE
Client :- China State Construction Engineering Ethiopia PLC
Location :- A.A (Ambasader)
Object :- Six Rock Samples

N°	BH N°	Depth (m)	Rock Strength Test		
			Unit Weight (Kg/m ³)	Compressive Strength (KN/m ²)	Modulus of Elasticity (KN/m ²)
1	12	29.00	2684	26702	1027000
2	12	47.50	2701	30237	222331
3	12	53.60	2812	48670	3080380
4	12	56.00	2792	58877	1863196
5	12	69.00	2814	31620	1550000
6	12	73.00	2783	48526	1708662

Note :-
Six graph for unconfined compressive strength test result are drawn and attached herewith.

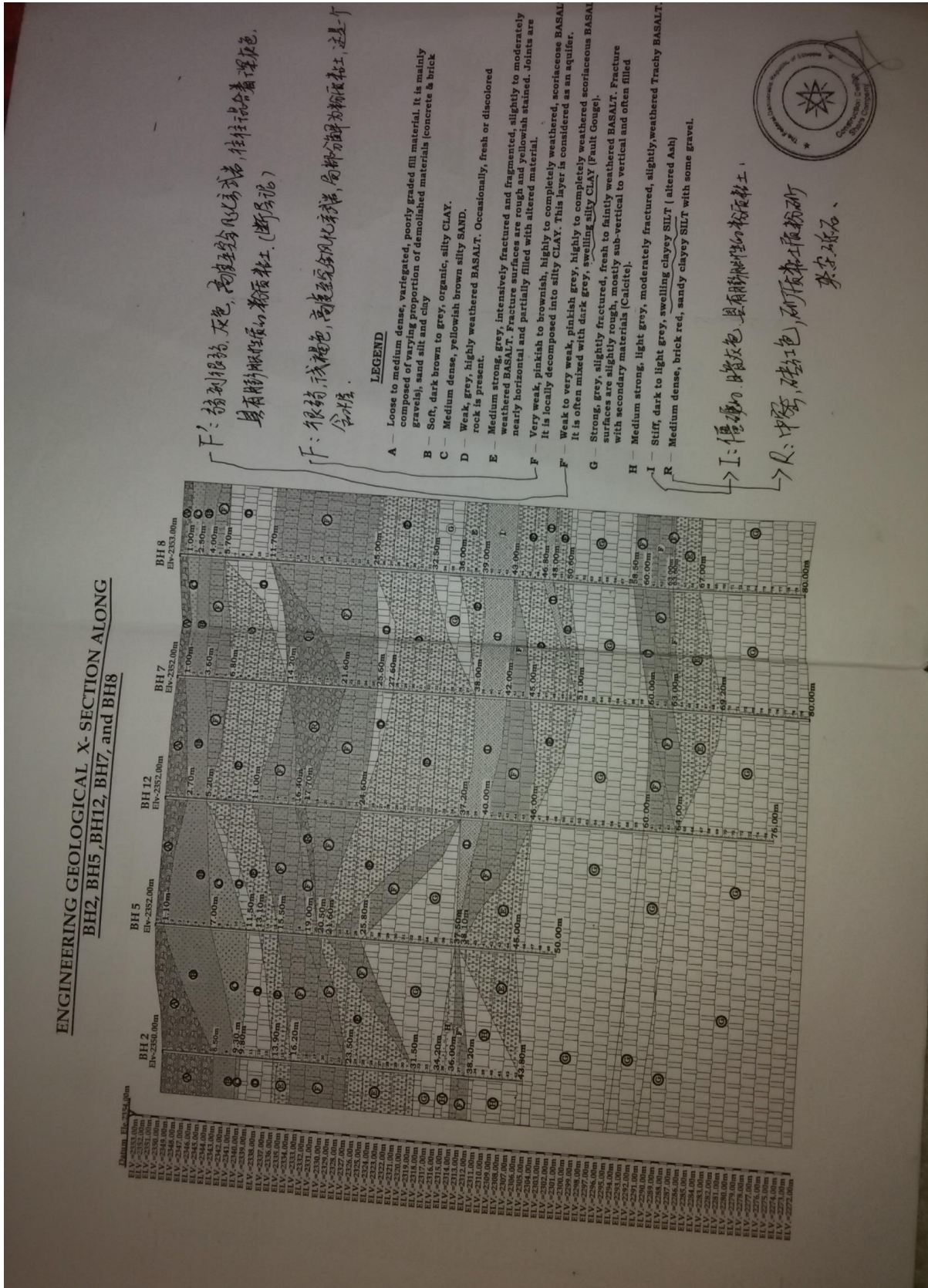
Tested by :- Airmeyas Aychew 
Date :- 11/11/2015
Checked by :- Abate Legesse 
Date :- 17/11/2015

Approved by :- Biruk Abdi 
Date :- 17/11/2015



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INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD



APPENDIX C

Approximation the actual soil profile in to single homogenous layer

**INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD**

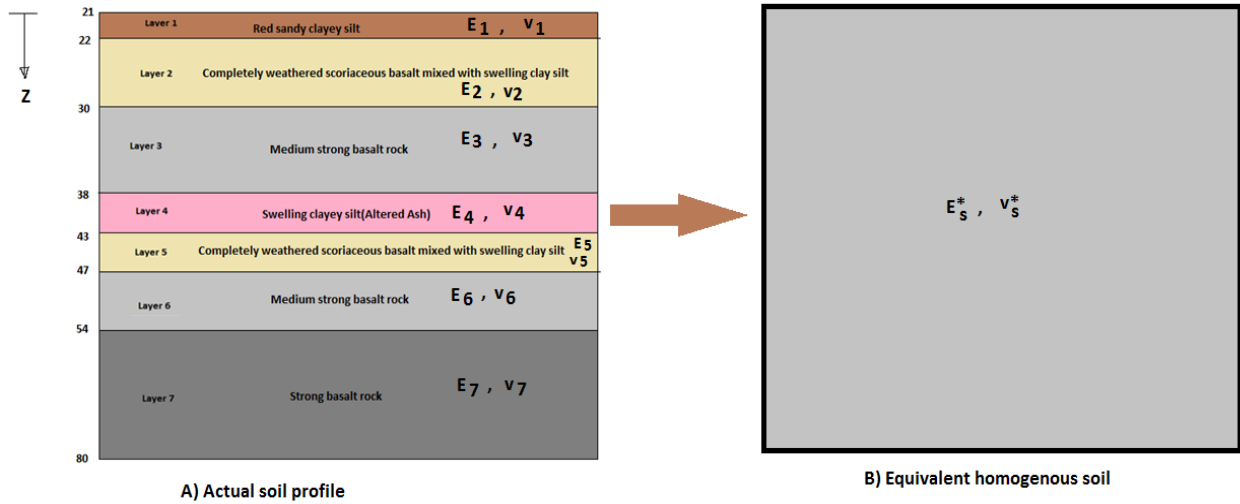


Figure 1: Diagram shows the actual soil profile is converted to equivalent single layer

E^* and ν^* shown in Figure1B are the equivalent young's modulus and equivalent Poisson's ratio for the whole soil given by

$$\frac{1}{E_s^*} = \sum_{i=1}^n \frac{1}{M_{s(i)}} \Delta I_i / \Delta I_{total}$$

$$\nu_s^* = \sum_{i=1}^n \nu_{s(i)} \Delta I_i / \Delta I_{total}$$

Where: $M_{s(i)}$ is the equivalent young's modulus for the soil layer number i given by

$$M_{s(i)} = E_{s(i)} / (1 - \nu_{s(i)}^2)$$

Where: $E_{s(i)}$ and $\nu_{s(i)}$ are the Young's modulus and the Poisson's ratio for the soil layer number i in the n -layered system.

$$\Delta I_i = I(Z_{top}^i) - I(Z_{bottom}^i)$$

$$\Delta I_{total} = I(0) - I(h)$$

Where: Z_{top}^i and Z_{bottom}^i are the depths below the surface of the top and bottom of layer number i , h = the thickness of layer i . The values of Z_{top}^i and Z_{bottom}^i are determined from Figure 2 shown below.

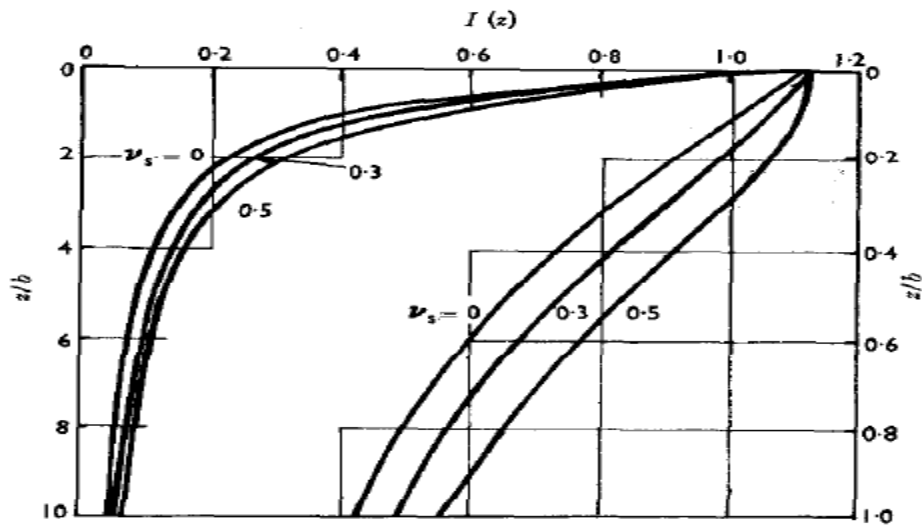


Figure 2: Vertical settlement influence factor []

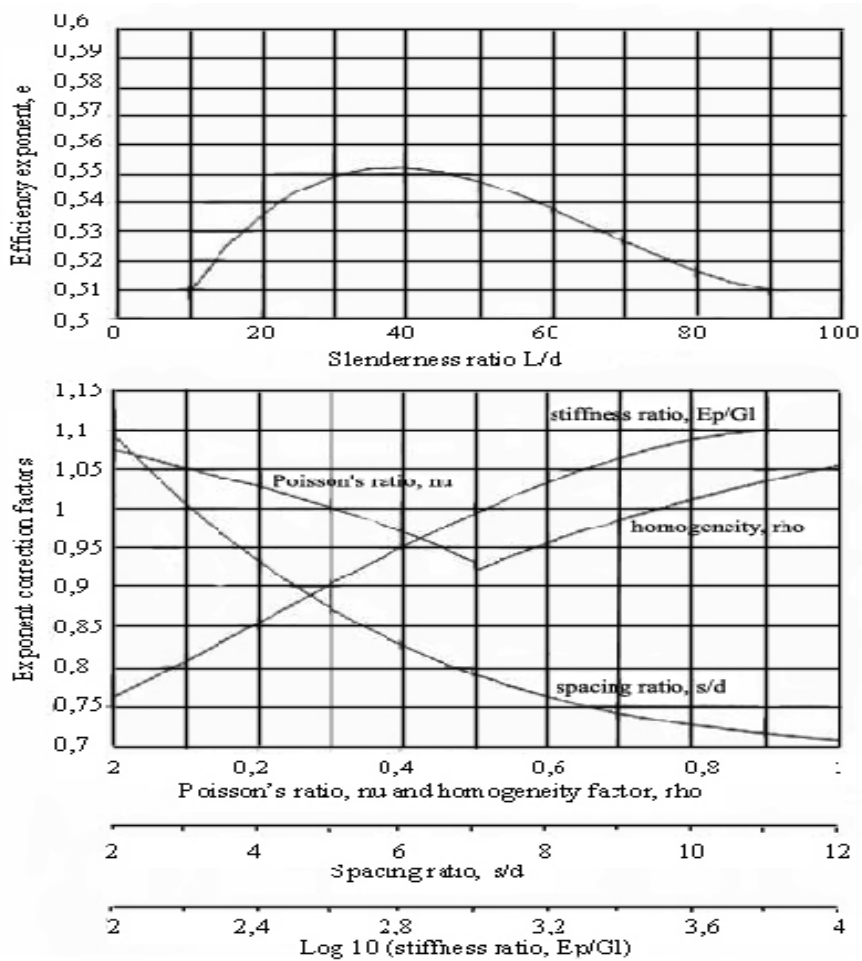


Figure 3 Design charts for the group efficiency and for the correction factors (after Fleming et al, 1992)

APPENDIX D

Cap Hardening Calculation Sheets and Generated Graphs

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON
HARD STRATA USING FINITE ELEMENT METHOD

Cancelation sheet used to generate the cap hardening curve .The mean effective stress is related with the plastic volumetric strain as the following equation

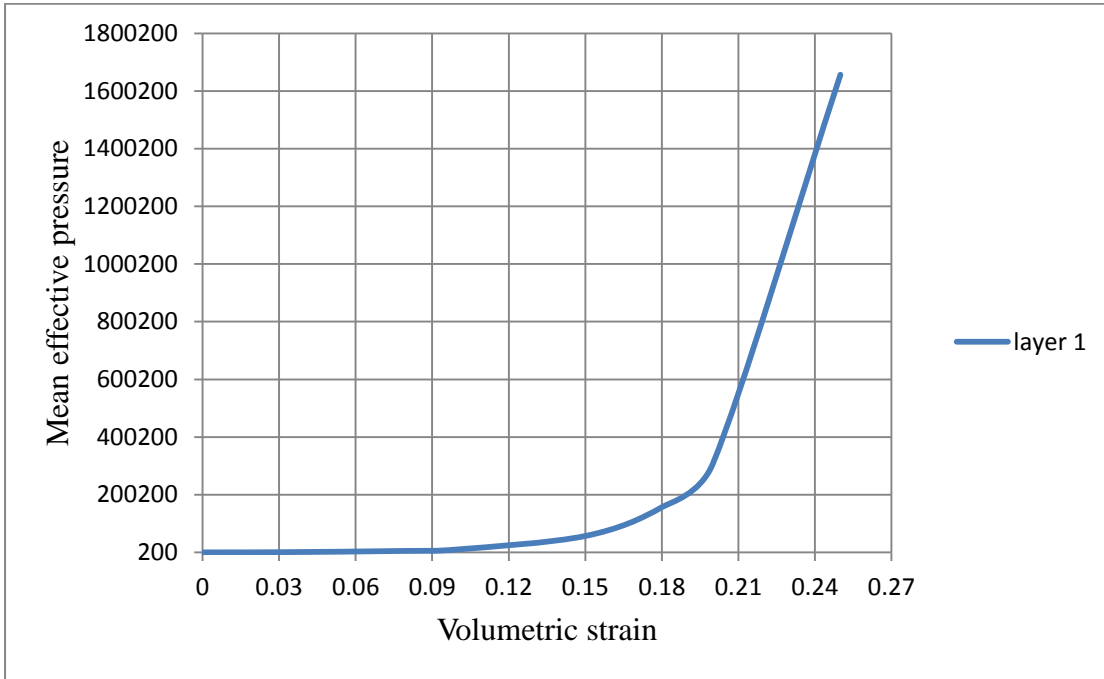
$$\varepsilon_v^p = \frac{C_c - C_s}{2.3(1 + e_o)} \ln \frac{p}{p_o}$$

Cap hardening parameters					
soil layers	Pre consolidation pressure (P _o)	Natural void ratio (e _o)	Compression Index (c _c)	Swelling Index (c _s)	X
layer1	366	1.313	0.186	0.028	0.0297
layer2	452	0.8	0.15	0.022	0.030918
layer3	636.5	0.3	0.1	0.015	0.028428
layer4	799.5	0.92	0.213	0.03	0.04144
layer5	890	0.2	0.15	0.022	0.046377
layer6	1009.5	0.3	0.1	0.015	0.028428
layer7	1513.5	0.2	0.1	0.015	0.030797

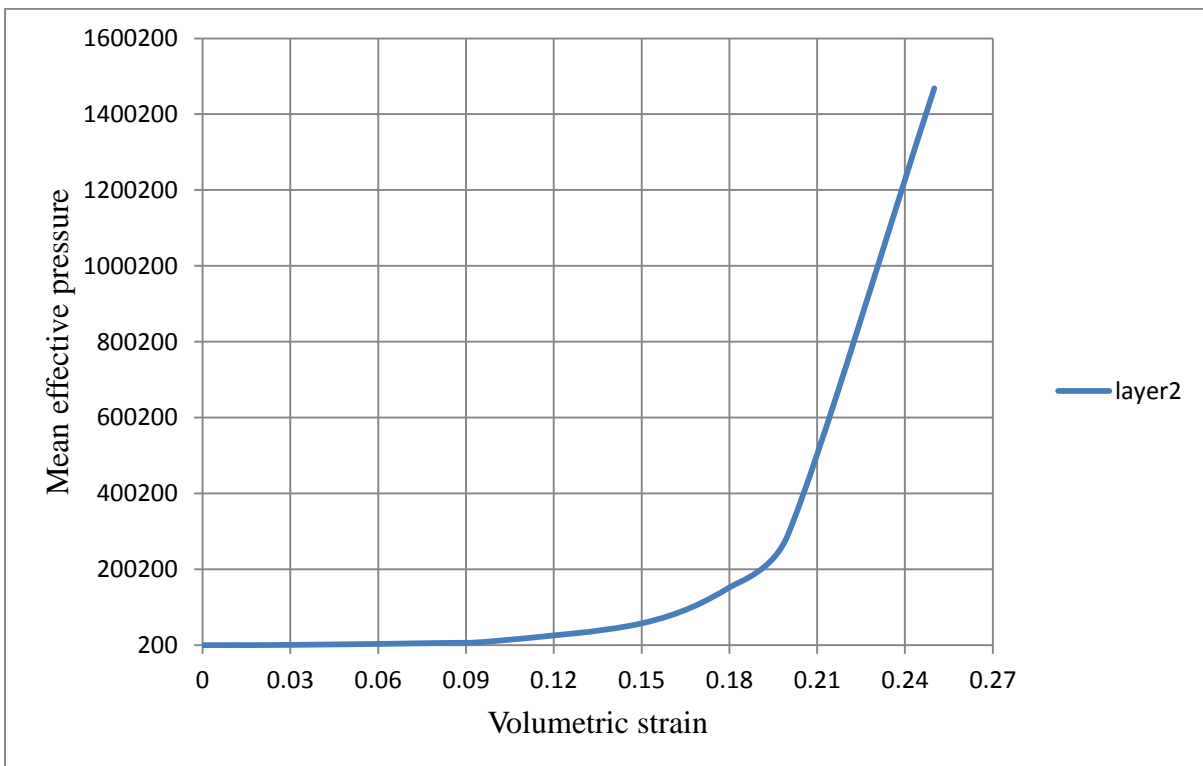
Let $X = \frac{C_c - C_s}{2.3(1 + e_o)}$ and the mean effective yield stress P is given by $P = P_o \exp \frac{\varepsilon_v^p}{X}$

Mean effective yield stress P							
Volumetric Plastic Strain, ε_v^p	layer1	layer2	layer3	layer4	layer5	layer6	layer7
	P in kPa	P in kPa	P in kPa	P in kPa	P in kPa	P in kPa	P in kPa
0	366	452	636.5	799.5	890	1009.5	1513.5
0.01	512.518	624.6051	904.8318	1017.698	1104.167	1435.079	2094.114
0.03	1004.998	1192.724	1828.55	1648.994	1699.512	2900.112	4009.003
0.08	5411.348	6010.013	10615.84	5510.873	4995.144	16836.9	20329.48
0.1	10611.13	11476.5	21453.25	8929.371	7688.427	34025.23	38919.04
0.15	57134.95	57828.94	124549.1	29841.6	22597.54	197537	197356.8
0.18	156886.7	152597.2	357807	61549.26	43151.46	567488.1	522764.3
0.2	307639.5	291394.2	723082.5	99729.42	66417.87	1146821	1000788
0.27	8919133	7398643	24371515	1113847	573762.9	38653644	25734848

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD

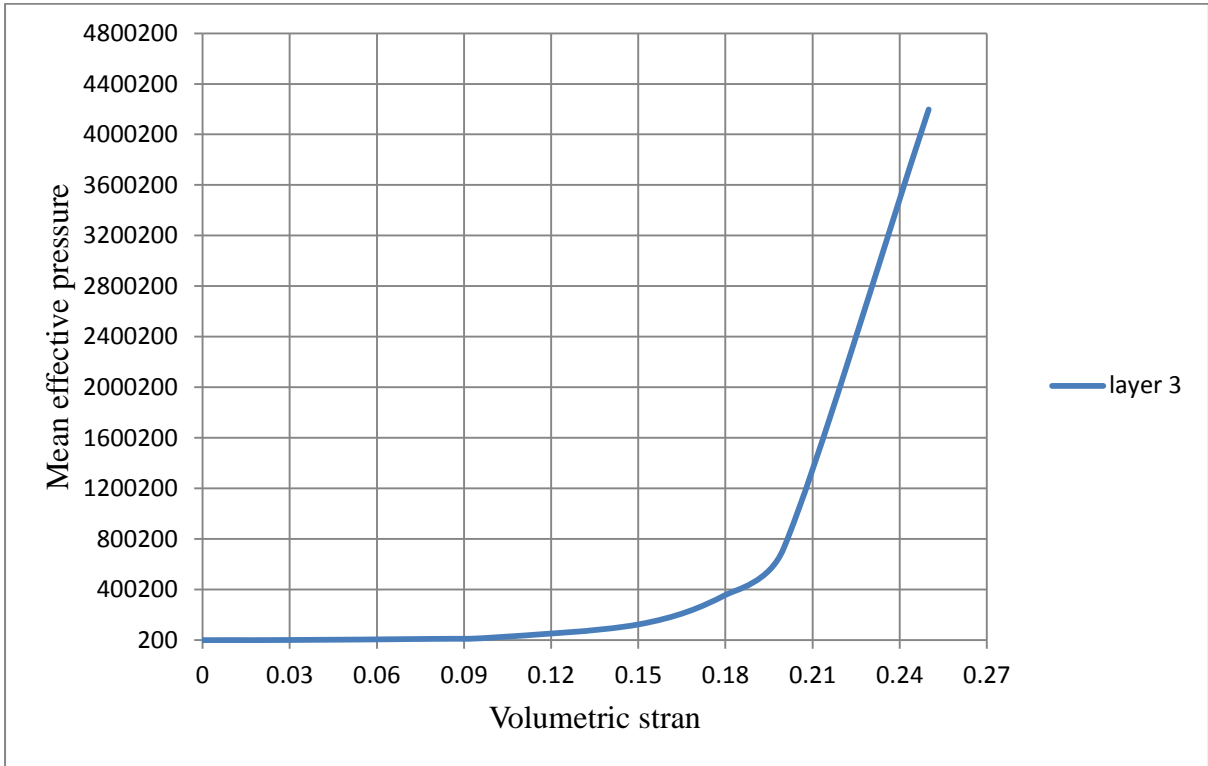


Cap hardening curve for soil layer 1

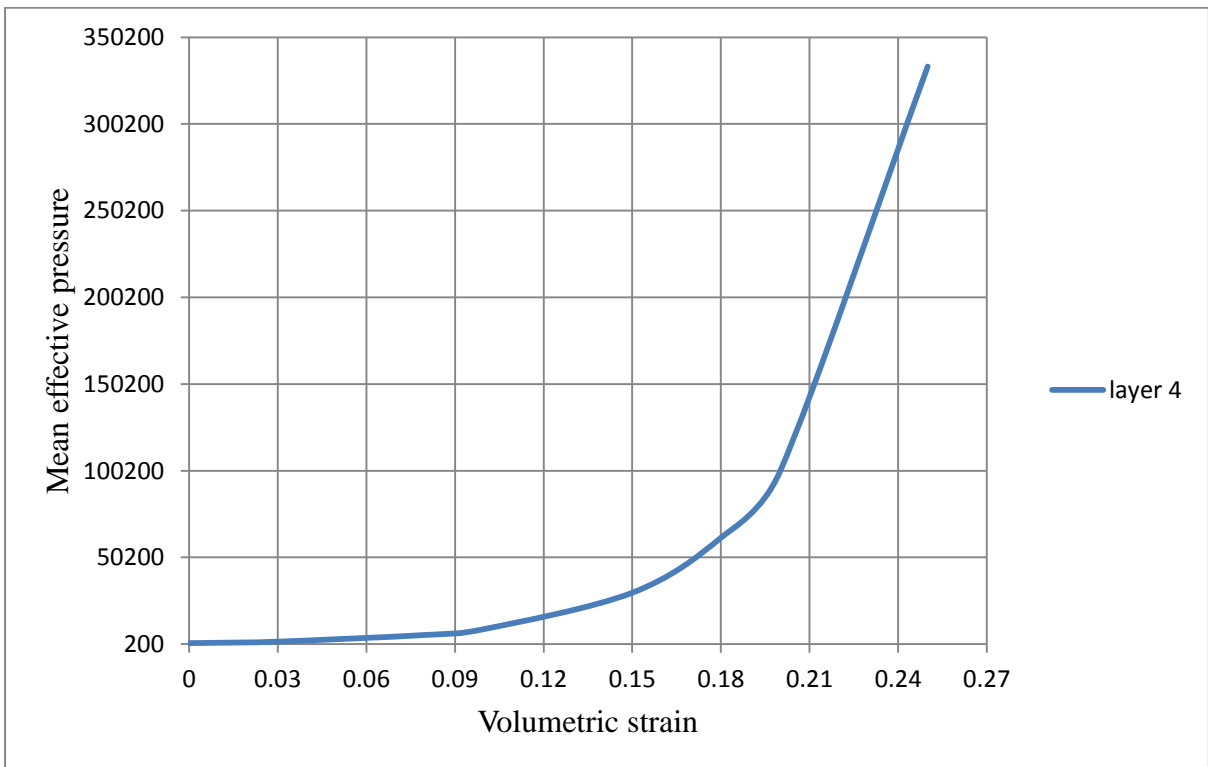


Cap hardening curve for soil layer 2

INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD

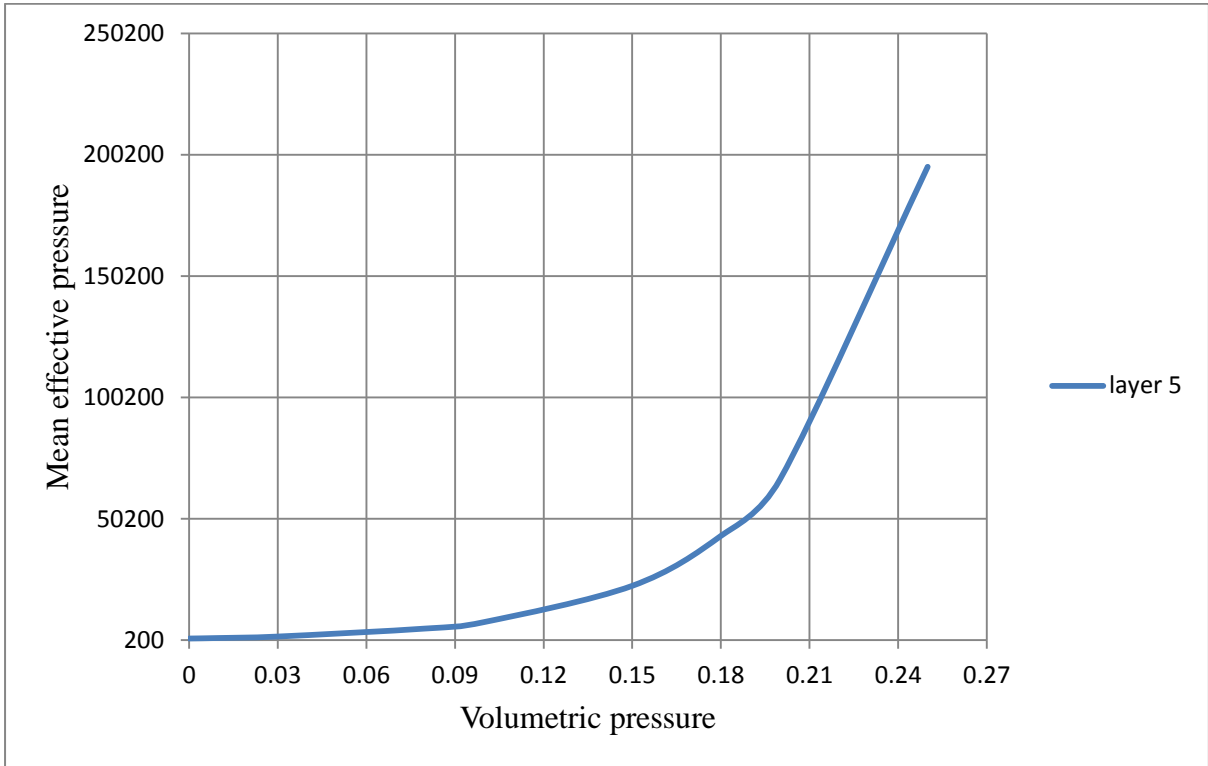


Cap hardening curve for soil layer 3

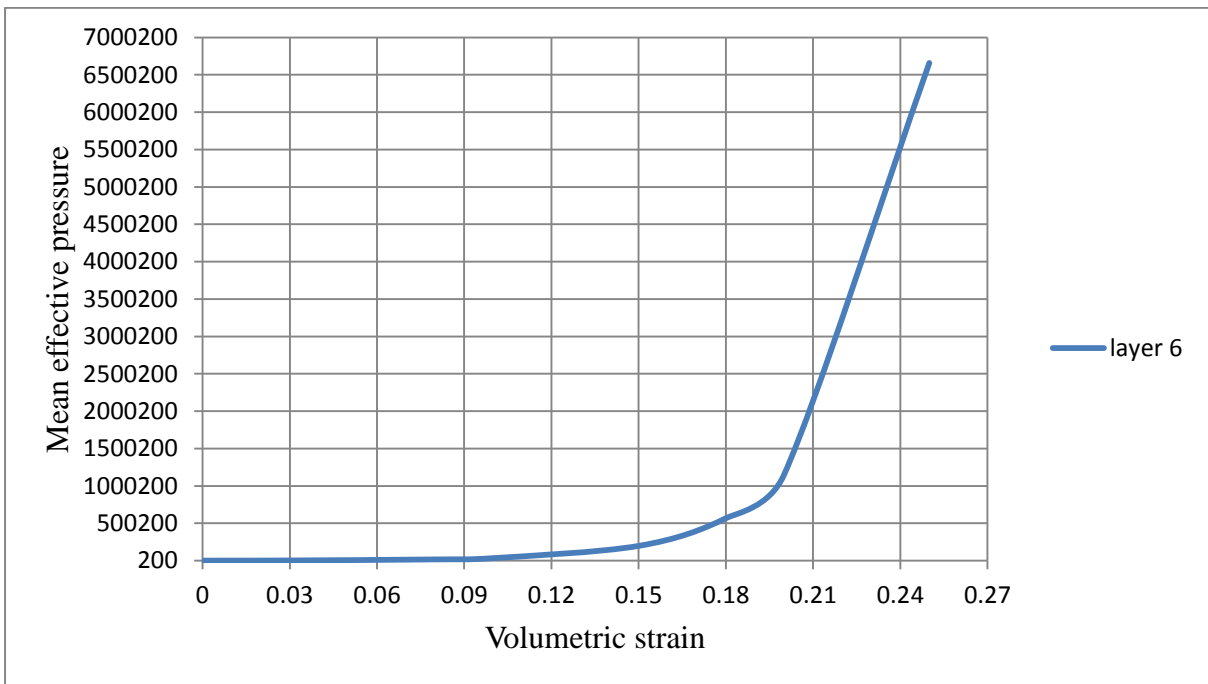


Cap hardening curve for soil layer 4

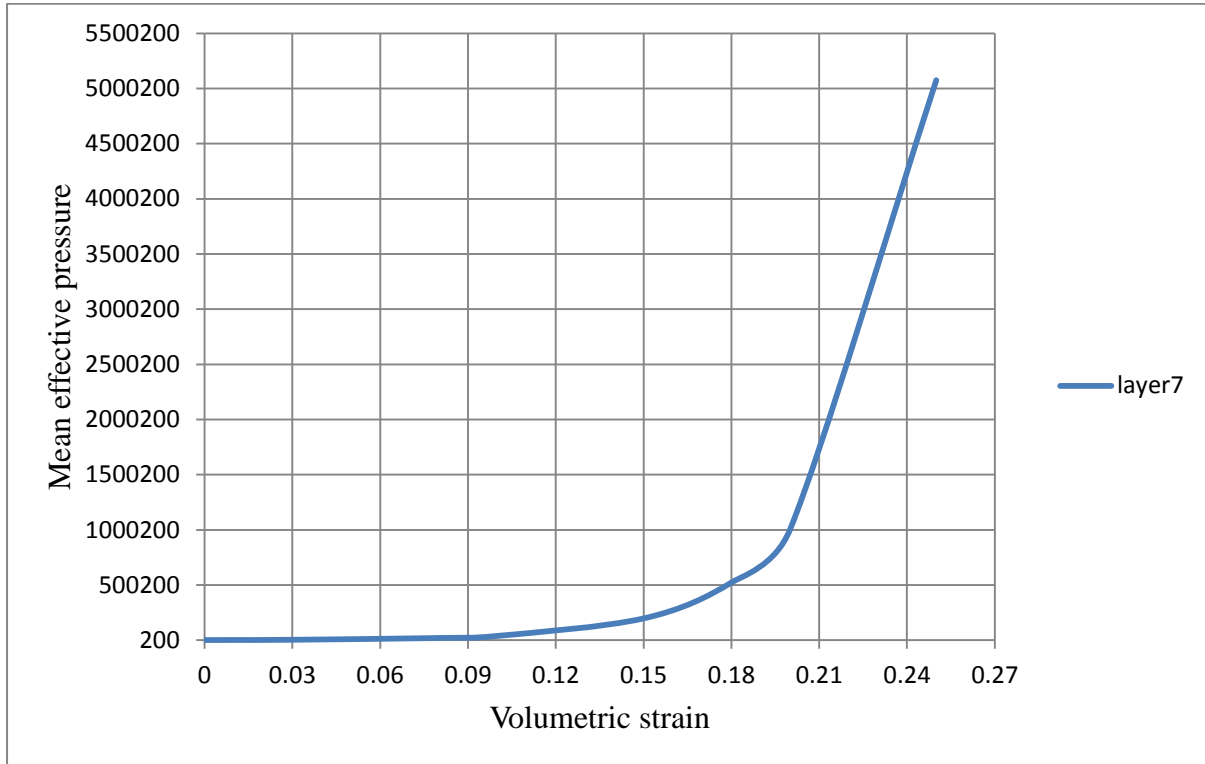
INVESTIGATING THE LOAD SHARING BEHAVIOUR OF PILED RAFT WITH PILE TIPS ON HARD STRATA USING FINITE ELEMENT METHOD



Cap hardening curve for soil layer 5



Cap hardening curve for soil layer 6



Cap hardening curve for soil layer 7

