# Assessment of "Piled Raft Foundation" for High-Rise Buildings Under Vertical Load in Addis Ababa: The case of Nib, Zemen, and United Bank Headquarter Buildings

**By**
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# Assessment of Piled Raft Foundation for High-Rise Buildings Under Vertical Load in Addis Ababa: The case of Nib, Zemen, and United Bank Headquarter Buildings

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1|Page

# Declaration

I, undersigned declare that this thesis is my original work, has not been presented for a degree in any other universities and that all sources of materials used for this thesis have been duly acknowledged.

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## Table of Content

## List of Symbols/Acronyms

A: Global displacement vector

$B_{r:}$ Width of the raft

CPT: Cone penetration test

CPRF: Combined piled raft footing

$E_s$: Stress-strain modulus

$E_{sh}$: Stress-strain modulus horizontal

$E_{sv}$: Stress-strain modulus value

FEM: Finite element method

K: Global stiffness matrix

$K_r$: Stiffness of raft

$K_{pr}$: Stiffness of piled raft

$K_p$: Stiffness of the pile group

$K_{pr}$: Stiffness of piled raft

$K_p$: Stiffness of the pile group

Kr: Stiffness of the raft alone

$K_r$: Stiffness of the raft alone

L: Length of the raft

$N_{cor}$: Corrected N value

$p_a$: Average atmospheric pressure

$P_{up}$: Ultimate load capacity of the piles in the group

S`: Design settlement

$S_{pr:}$ Settlement of piled raft

$S_r$: Settlement of raft without piles

SPT: Standard penetration test

X: Proportion of load carried by the piles

γ: Unite weight

z: Depth

$\sigma_z$: Overburden pressure at a depth z below the foundation level

$\Delta\sigma_z$ : Additional vertical stress due to the loads from the superstructure

$\alpha_{cp}$: Raft – pile interaction factor

$\alpha_{cp:}$ Raft – pile interaction factor

3D: Three dimensional

3D: Three times diameter of the pile

6D: Six times diameter of the pile

## List of Figures

## List of Table

## Abstract

Piled Raft foundation is a newly emerged foundation system. Different researchers are working on it till now. The system is mainly used to respond to weak foundation materials.

The intent of this study is, to assess the piled-raft foundations of high-rise buildings under vertical load. These foundations are designed by local engineers to respond to high-rise building foundation requirement. The study is conducted using finite element method software package – ABAQUS.

The numbers of projects considered in this study are three. They have similar nature and are under construction. The designers use similar pile spacing for all projects and it is the only variable parameter in this study. The foundations are modeled with pile spacing of 3D (designers spacing) and 6D (two times designers spacing).

The site's geology is predominantly rocky but in different weathered condition. The assessment result shows maximum and differential settlement of the foundations are very insignificant and angular distortion is within permissible limit. Almost all the vertical load is taken by the pile and the load shared by the raft is very insignificant.

# INTRODUCTION

## 1.1 Background of the Study

One of the most important aspects of civil engineering projects is a foundation system. Carefully and properly designed foundations surely lead to a safe, efficient and economic project.

Foundation systems are divided into two: shallow and deep foundations. Selection of the foundation system mainly depends on the nature of the structure and type of earth material.

In piled-raft foundations, the piles are used as elements to enhancing the behavior of the raft to satisfy the design requirements, and they are typically not considered as carriers for the total structural load. The design requirements may be related to the settlement control or increase the ultimate bearing capacity of the foundation. Since the main purpose of the piles in the majority of piled foundations is to limit settlement, the piles in the piled raft will serve mainly as settlement reducers, (Adel Y. Akl, 2014).

According to Poulos, there are three broad classes of numerical analysis methods: Simplified Calculation Methods, Approximate Computer-Based Methods, and More Rigorous Computer-Based Methods. He also noted recently nonlinear 3D finite element and finite difference analyses methods. However, modeling problems related to the soil–structure interface still remain in the 3D finite element and finite difference analysis. The great challenge in the numerical methods is the choice of proper input parameters to give reasonable output results. The procedure of choosing right values for the input parameters can be adjusted by making back analysis for well-documented case histories, (Poulos, 2001).

In our country, piled-raft foundation system is started to be used for high-rise buildings, and different mega-structures. Having knowledge of piled raft foundation design, understanding the effect of vertical load, awareness of the parameters, importance of piled raft foundation in geotechnical engineering and other valuable points related to this system is very essential for economic and safe design. The local engineers design piled raft foundations as a pile foundation with the connected cap.

In the capital city, different high-rise buildings are constructed on ′′connected cap piled foundation′′. Even if these foundations are designed as a piled foundation, they can be designed as piled raft foundation system.

This paper, therefore focused on the 4B+G+30, 4B+G+30, and 4B+G+30 Headquarter Building projects that have piled-raft foundations. The owners of the project are United Bank Shared Company, Zemen Banks Shared Company, and Nib International Bank Share Company respectively. The sites are located in Addis Ababa, around Mexico, in front of the Headquarter of Awash International Bank. The study is conducted using finite element modeling software package Abacus.

## 1.2 Statement of the Problem

High-rise buildings and different civil structures are being constructed on piled raft foundations depending on the nature of the soil. The main driving force for the selection of this type of foundation is the poor bearing capacity of the foundation. But in our country, the selection is not based on this aspect. The knowledge of piled-raft foundation system is very important in all aspects.

The focus of this paper is, to assess the settlement and load sharing nature of piled raft foundation of the above-stated projects under vertical load. The major findings may be helpful to understand similar projects upcoming in the future.

## 1.3 Objectives

### 1.3.1 General Objective

The general objective of this study is to critically assess the piled raft foundation of three high-rise building settlement and load sharing nature under vertical load.

### 1.3.2Specific Objectives

The specific objectives of this study are:

- To assess piled raft designs settlement under vertical load;
- To carry out and compare settlement analysis of piled raft foundation with 3D (actually designed pile spacing) and 6D pile spacing(optimization);
- To investigate the pile capacity of piled raft foundations under vertical load; and ;
- To determine the load sharing between the pile and the raft;

## 1.4 Methodology

The study approach is quantitative and method is mixed study methods. (i.e. Case study and numerical analysis). The study sampling focus on active projects found in the city. This is only to avoid data unavailability.

Primarily, a brief literature review is made from related documents. The primary data of soil investigation, structural geometry, site condition, other project documents, and information are collected from the site, the consulting offices, and the owners. The soil-pile-raft system is simulated using finite element software ABAQUS. Basically, the material property of the soil and some geometric properties like pile diameters, pile length, raft thickness and raft dimension are kept constant. The only variable parameter is the pile spacing. The projects foundation system is modeled as designers decision (i.e. pile spacing equal to 3D) and then modeled with pile spacing equal to 6D (doubled design spacing). Only structural vertical load is considered in the study and it is applied on the top of the raft in the form of uniformly distributed load. The pile top and the raft are connected rigidly.

## 1.5 Scope of the Study

This study is conducted on construction projects of United Bank Share Company, Zemen Banks Share Company and Nib International Bank Share Company Headquarter projects that are under construction and found in Addis Ababa, Kirkos Sub-City, in front of Awash International Bank Headquarter Building.

## 1.6 Significance of the Study

The results gained from this study would help to:-

- ✓ Investigate settlement of high-rise buildings that have piled raft foundation under vertical load;
- ✓ To understand the load sharing nature of piled-raft under vertical load;
- ✓ Assess the design condition of the buildings under vertical loading;
- ✓ To contribute its part to the area.

## 1.7. Organization of the Thesis

This paper contains five chapters. The first chapter is an introductory part and it contains the background of the study, objective of the study, the scope of the study and significance of the study. The second chapter is a literature review. Under this part; design philosophy, design consideration, settlement of piled raft foundation, and tolerable settlement of a building is addressed. The third chapter deals with field and laboratory results. Some soil parameters are determined using correlation formulas in this chapter. The fourth chapter is about FEM of piled raft foundation. (i.e. piled raft and material modeling). Here also, modeling of piled raft foundation on Abaqus addressed. The last chapter contains a summary of the thesis, major findings, and recommendations made.

# 2 LITERATURE REVIEW

## 2.1 Introduction

According to Wang, Shenzhi (2004), piled raft foundation is a composite geotechnical structure consisting of three parts: piles, raft, and soil. It is widely recognized as an economically effective design because the use of piles is to reduce raft settlements and differential settlements without compromising the safety and performance of the foundation.

Piled Raft foundation is different from a pile cap on a group of piles; the piled-raft foundation is designed with a completely different philosophy. The former type is a fully piled foundation in which all the superstructure loading is taken by the piles. While in the piled-raft foundation, the piles are designed as only settlement reducers ;( Burland et al., 1977).

Piled rafts foundations are grouped into two broad categories on the basis of the design requirements:

**Small piled rafts:** - those in which the bearing capacity of the unpiled-raft is insufficient, and thus the primary reason to add the piles is to achieve a suitable safety factor. The width of the raft $B_r$, belonging to this category, is generally small in comparison to the length L of the piles ($B_r/L<1$) and amounts to a few meters. The flexural stiffness of the raft is usually high and the differential settlement does not signify a problem; (Mohammed Y., 2013).

**Large piled rafts**: - those in which the bearing capacity is sufficient to carry the total load with a reasonable margin so that the addition of piles is usually intended to reduce settlement. In general, the width $B_r$ of the raft is relatively large in comparison with the length of the piles ($B_r/L>1$); (Mohammed Y., 2013).

According to M.R Mahmood (2017), piles are arranged to minimize differential deflection in the raft. The piled raft is economical compared to pile foundation as the piles are not penetrated to full depth of the clay. Regarding settlement, piled raft settle more than piled foundation but has less settlement than raft foundation.

## 2.2 Design Concepts

Anup S. (2013) reviewed the design philosophy and design consideration of piled raft foundation as follows.

### 2.2.1 Design Philosophy

Numerous works have been done and still in progress to develop a definite design strategy for piled raft foundation. Researchers use their own design philosophy to formulate the design process for piled raft foundation. Randolph (1994) categorized the various design philosophy into three different design approaches.

a) **The conventional approach**, in which, the foundation is designed as a pile group with regular spacing over the entire foundation area, to carry the major portion of the load (60 – 75% of the total structural load) and allowance is made for the raft to transmit some load directly to the ground.

The conventional approach has the limitation that any design by this philosophy will remain inevitably in elastic regime, where the piles are loaded below their shaft capacity and piles are evenly distributed over the whole foundation area. The shaft capacity of the group pile is "extremely difficult and has not resolved yet" (Das 2007). Moreover, the allowance for the raft to transmit the load is not defined in this approach, which they let it for engineering judgment.

b) **Creep piling approach**, proposed by Hansbo and Kalstrom (1983), in which, piles are designed to operate at a working load, at which significant creep starts to occur (typically at about 70 – 80% of its ultimate load bearing capacity). To reduce the net contact pressure between the raft and soil, adequate piles are added in order to reduce the pre-consolidation pressure of the clay.

The creep piling approach again sets the limitation of operating the pile below the creep load, which is, as mentioned above, as 70% to 80% of ultimate load-bearing capacity. However, the ultimate capacity is based on conventional group theory, which is under investigation. Moreover, the reduction of net contact pressure between raft and soil by means of pile addition will refrain the raft from transmitting the load to its full capacity directly to the soil.

c) **Differential settlement control approach**, in which, pile supports are designed tactfully in order to minimize differential settlement rather than reducing average settlement significantly.

The differential settlement control approach could be the economic one, as piles are located strategically to reduce the differential settlement. It will require less number of piles, in comparison to the other two approaches. However, to cause the differential settlement to occur, the utilization of the ultimate bearing capacity of each of the single individual pile in the group is required, which is not yet established and cannot be captured by any analytical method developed so far. The goal, to use the ultimate bearing capacity according to the requirement of raft-pile-soil interaction, can be achieved by the numero-geotechnical methods by simulating the complex nature of piled raft foundation.

#### 2.2.2 Design Considerations

Katzenbach et al. (2005) termed piled raft foundation as a Combined Piled Raft Foundations (CPRF), which consists of three bearing elements, piles, raft, and subsoil. The stiffness of raft and pile, the soil properties, the dimension and strategy of pile location, play a significant role in the design of a piled raft foundation system. Figure 2.1 illustrates general piled raft foundation with its overall forces and moments.

The following design issues must be taken into consideration, in order to design a successful piled raft foundation.

1. Ultimate geotechnical capacity under vertical, lateral and moment loadings,
2. Maximum and total settlements,
3. Differential settlement and angular rotation,
4. Lateral movement and stiffness,
5. Load sharing between the piles and raft,
6. Raft moment and shear for the structural design of raft and its stiffness,
7. Pile loads and moments for the structural design of the piles and its stiffness.

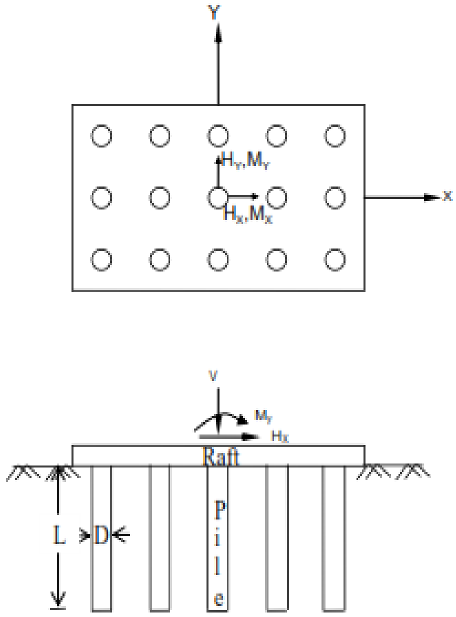



**Figure 2-1** Piled Raft foundation system

A sound prediction of a piled raft foundation behavior implies a full interaction between piles, raft, and soil. Hence, the following factors must be considered.

1. The raft characteristics (its relative stiffness – flexibility, rigidity, shape - dimension).

2. Pile characteristics (number, layout, length, diameter, stiffness)

3. Applied load characteristics (concentrated or distributed load and its level related to the ultimate capacity)

4. Soil characteristics (soil profile, layers and their stiffness, the ultimate soil bearing capacity)

Therefore, it is essential to develop a design method for pile raft foundation, which includes the following capabilities

1. Piles-raft-soil interaction in a logical manner, including the interaction of pile-pile, raft-pile, pile-raft, raft-soil, and pile-soil,

2. Variation of number, location, and characteristics of the pile,

3. Load sharing of pile and raft and,

4. Full utilization of the ultimate pile load capacity in compression and tension as well as the nonlinear load-deflection behavior of the foundation.

H.G. Poulos (2001) also states the same Design Philosophies for piled raft foundation as that of Randolhp (2001).

#### 2.2.3 Method of Analysis

According to the report made by H.G. Poulos on behalf TC18 International Society of Soil Mechanics and Geological Engineering, July 2001, several methods of analyzing piled rafts have been developed, and some of these have been summarized by Poulos et al (1997). Three broad classes of analysis method have been identified:

- Simplified calculation methods,
- Approximate computer-based methods,
- More rigorous computer-based methods.

Simplified methods include those of Poulos and Davis (1980), Randolph (1983, 1994), Van Impe and Clerq (1995), and Burland (1995). All involve a number of simplifications in relation to the modeling of the soil profile and the loading conditions on the raft.

The approximate computer-based methods include the following broad approaches:

- Methods employing a "strip on springs" approach, in which the raft is represented by a series of strip footings and the piles are represented by springs of appropriate stiffness (e.g. Poulos, 1991)

- Methods employing a "plate on springs" approach, in which the raft is represented by a plate and the piles as springs (e.g. Clancy and Randolph, 1993; Poulos, 1994; Viggiani, 1998; Anagnastopoulos and Georgiadis, 1998).

The more rigorous methods include:

• Boundary element methods, in which both the raft and the piles within the system are discretized, and use is made of elastic theory (e.g. Butterfield and Banerjee, 1971; Brown and Wiesner, 1975; Kuwabara, 1989; Sinha, 1997)

• Methods combining boundary element for the piles and finite element analysis for the raft (e.g. Hain and Lee, 1978; Ta and Small, 1996; Franke et al, 1994; Russo and Viggiani, 1998)

•Simplified finite element analyses, usually involving the representation of the foundation system as a plane strain problem (Desai, 1974) or an axisymmetric problem (Hooper, 1974), and corresponding finite difference analyses via the commercial program FLAC (e.g. Hewitt and Gue, 1994)

• Three-dimensional finite element analyses (e.g. Zhuang et al, 1991; Lee, 1993; Wang, 1995; Katzenbach et al, 1998) and finite difference analyses via the commercial program FLAC 3D.

In the following section, a more detailed description will be given of a limited number of the above methods, and these will then be used to analyze a relatively simple hypothetical problem.

#### 2.2.3.1 Simplified Analysis Methods

##### 2.2.3.1.1 Poulos-Davis-Randolph (PDR) Method

For assessing the 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:

- The sum of the ultimate capacities of the raft plus all the piles;
- 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.

For estimating the load-settlement behavior, an approach similar to that described by Poulos and Davis (1980) can be adopted. However, a useful extension to this method can be made by using the simple method of estimating the load sharing between the raft and the piles, as outlined by Randolph (1994). The definition of the pile problem considered by Randolph is shown in Figure 2.2. Using his approach, the stiffness of the piled raft foundation can be estimated as follows:

$$K_{pr} = \frac{(K_p + Kr(1-\alpha_{cp})}{(1-\alpha_{cp}\dfrac{2K_r}{K_p})} \qquad \text{(Eq.2.1)}$$

Where:-$K_{pr}$= stiffness of piled raft

$K_p$ = stiffness of the pile group

$K_r$ = stiffness of the raft alone

$\alpha_{cp}$ = raft – pile interaction factor.

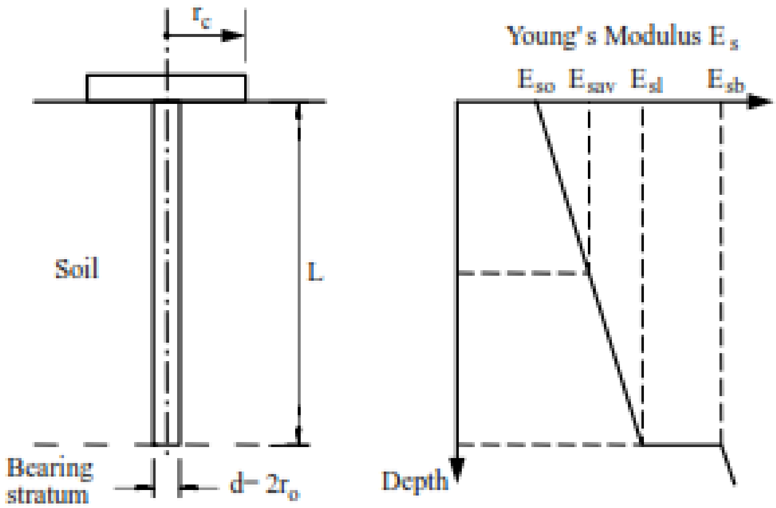



**Figure 2-2** Simplified representation of Piled-raft unit ( H.G. Poulos 2001)

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

$$\frac{p_r}{P_t} = \frac{K_r(1-\alpha_{cp})}{(K_p + K_r(1-\alpha_{cp})} = X \qquad \text{(Eq. 2.2)}$$

Where: -$P_r$ = load carried by the raft

$P_t$ = total applied load.

The raft – pile interaction factor $\alpha_{cp}$ can be estimated as follows:

$$\alpha_{cp} = 1 - \ln(\frac{r_c}{r_0})\,\zeta \qquad \text{(Eq. 2.3)}$$

Where: -$r_c$ = average radius of pile cap, (corresponding to an area equal to the raft area divided by number of piles)

$r_0$ = radius of pile

Φιγυρε 12 = ln ($r_m$ / $r_0$)

$r_m$= {0.25+ξ [2.5 ρ (1-v) – 0.25) * L

$$\zeta = \frac{E_{sl}}{E_{sb}}$$

$$\rho = \frac{E_{sav}}{E_{sl}}$$

v = Poissons ratio of soil

L = pile length

$E_{sl}$ = soil Young's modulus at level of pile tip

$E_{sb}$ = soil Young's modulus of bearing stratum below pile tip

$E_{sav}$= average soil Young's modulus along pile shaft.

The above equations can be used to develop a tri-linear load-settlement curve as shown in Figure 2.3. First, the stiffness of the piled raft is computed from equation (2.1) for the number of piles being considered. This stiffness will remain operative until the pile capacity is fully mobilized. Making the simplifying assumption that the pile load mobilization occurs simultaneously, the total applied load, $P_1$, at which the pile capacity is reached is given by:

$$P_1 = \frac{P_{up}}{(1-X)} \qquad \text{(Eq.2.4)}$$

Where:-$P_{up}$ = ultimate load capacity of the piles in the group

X = proportion of load carried by the piles (Equation 2.2).

Beyond the point (Point A in Figure 2.3), the stiffness of the foundation system is that of the raft alone ($K_r$), and this holds until the ultimate load capacity of the piled raft foundation system is reached (Point B in Figure 2.3). At that stage, the load-settlement relationship becomes horizontal.

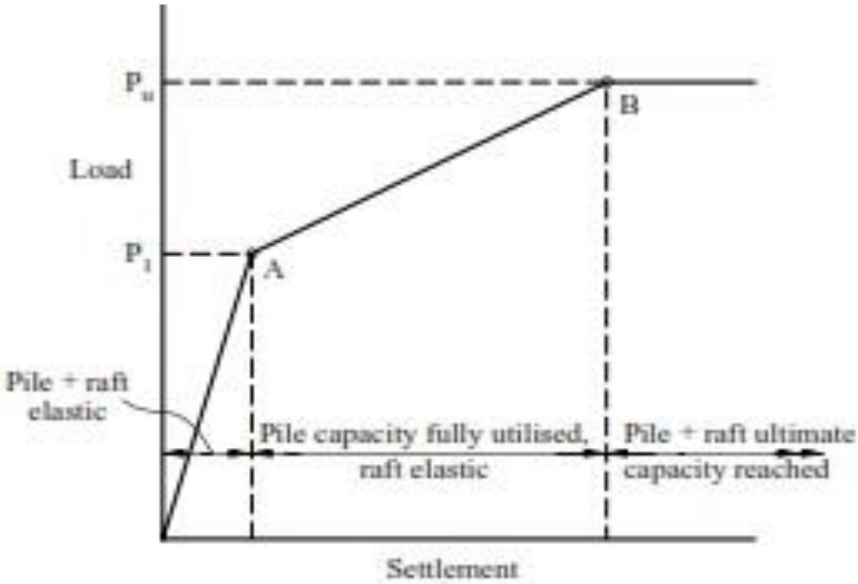



**Figure 2-3** Simplified load-settlement curve for preliminary analysis ( H.G. Poulos, 2001)

#### *2.2.3.1.2 Burland's Approach*

When the piles are designed to act as settlement reducers and to develop their full geotechnical capacity at the design load, Burland (1995) has developed the following simplified process of design:

• Estimate the total long-term load-settlement relationship for the raft without piles (see Figure 2.4). The design load $P_0$ gives a total settlement $S_0$.

• Assess an acceptable design settlement $S_d$, which should include a margin of safety.

• $P_1$ is the load carried by the raft corresponding to $S_d$.

• The load excess $P_0 - P_1$ 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, $P_{su}$.

• If the piles are located below columns which carry a load in excess of $P_{su}$, the piled raft may be analyzed as a raft on which reduced column loads act. In such columns, the reduced load $Q_r$ is:

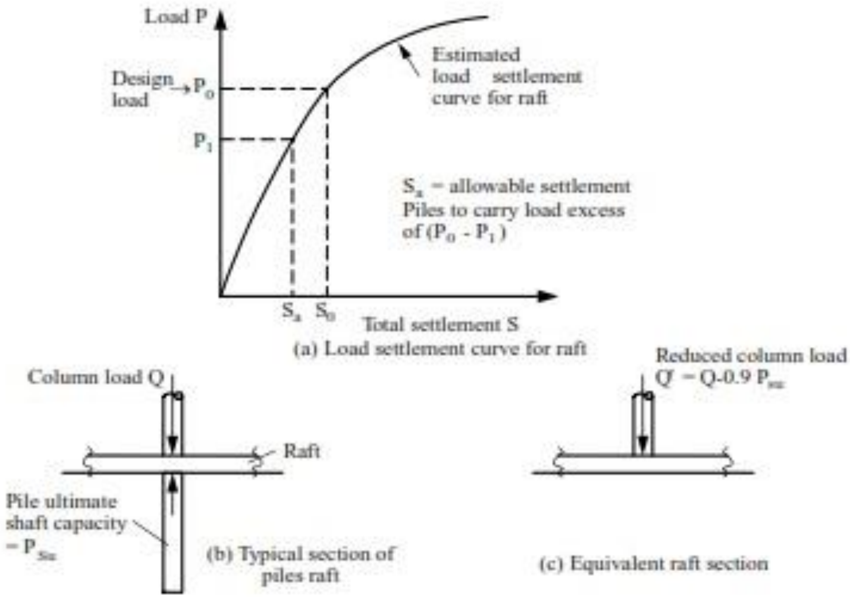



**Figure 2-4** Burland's Simplified Design Concept (Poulos, 2001)

$$Q_r = (Q - 0.9P_{su}) \quad \text{(Eq.2.5)}$$

• The bending moments in the raft can then be obtained by analyzing the piled raft as a raft subjected to the reduced loads $Q_r$.

• The process for estimating the settlement of the piled raft is not explicitly set out by Burland, but it would appear reasonable to adopt the approximate approach of Randolph (1994) in which:

$$S_{pr} = \frac{S_r * K_r}{K_{pr}} \quad \text{(Eq. 2.6)}$$

Where:-$S_{pr}$ = settlement of piled raft;

$S_r$ = settlement of raft without piles subjected to the total applied load;

$K_r$ = stiffness of raft;

$K_{pr}$= stiffness of piled raft;

$K_{pr}$ =can be estimated using.

$$K_{pr} = \frac{(K_p + Kr(1-\alpha_{cp}))}{(1-\alpha_{cp}\dfrac{2K_r}{K_p})} \quad \text{(Eq.2.7)}$$

Where:-$K_{pr}$ = stiffness of piled raft ;

$K_p$ = stiffness of the pile group;

$K_r$ = stiffness of the raft alone ;

$\alpha_{cp}$= raft – pile interaction factor.

### 2.3.2 Approximate Computer Methods

#### 2.3.2.1 Strip on Springs Approach (GASP)

An example of a method in this category is that presented by Poulos (1991) and illustrated in Figure 2.5. A section of the raft is represented by a strip and the supporting piles by springs. Approximate allowance is made for all four components of interaction (raft-raft elements, pile-pile, raft-pile, pile-raft), and the effects of the parts of the raft outside the strip section being analyzed are taken into account by computing the free-field soil settlements due to these parts. These settlements are then incorporated into the analysis, and the strip section is analyzed to obtain the settlements and moments due to the applied loading on that strip section and the soil settlements due to the sections outside the raft.

The method has been implemented via a computer program GASP (Geotechnical Analysis of Strip with Piles) and has been shown to give settlements which are in reasonable agreement with more complete methods of analysis.

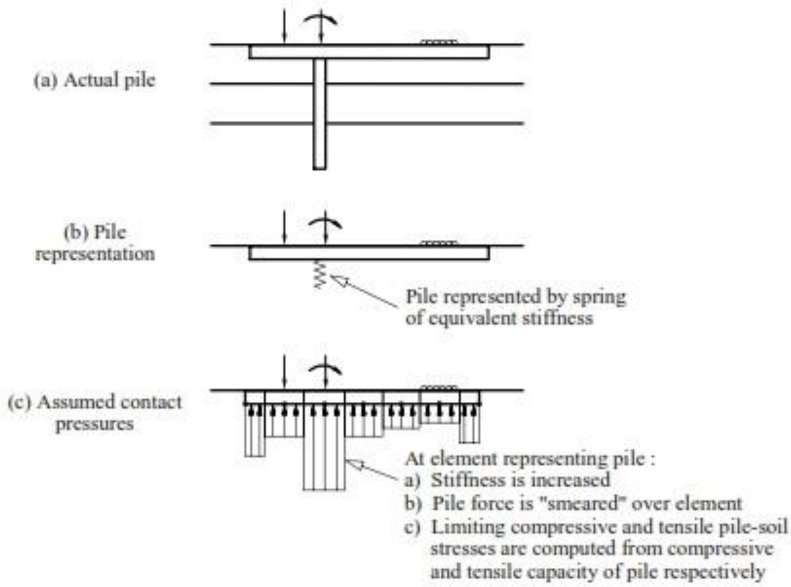



**Figure 2-5** Representation of piled strip problem via GASP analysis (Poulos, 1991)

However, it does have some significant limitations, especially as it cannot consider tensional moments within the raft, and also because it may not give consistent settlements at a point if strips in two directions through that point are analyzed.

In GASP, the soil non-linearity is taken to account in an approximate manner by limiting the strip-soil contact pressures not to exceed the bearing capacity (in compression) or the raft uplift capacity in tension. In carrying out a nonlinear analysis in which strips in two directions are analyzed, it has been found desirable to only consider nonlinearity in one direction (the longer direction) and to consider the pile and raft behavior in the other (shorter) direction to be linear. Such a procedure avoids unrealistic yielding of the soil beneath the strip and hence unrealistic settlement predictions.

#### 2.3.2.2 Plate on Springs Approach (GARP)

In this type of analysis, the raft is represented by an elastic plate. The soil is represented by an elastic continuum and the piles are modeled as interacting springs. Some of the early approaches in this category (e.g. Hongladaromp et al, 1973) neglected some of the components of interaction and gave piled-raft stiffnesses which were too large.

Russo (1998) and Russo and Viggiani (1997) have described a similar approach to the above methods, in which the various interactions are obtained from elastic theory, and non-linear behavior of the piles is considered via the assumption of a hyperbolic load-settlement curve for single piles. Pile-pile interaction is applied only to the elastic component of pile settlement, while the non-linear component of the settlement of a pile is assumed to arise only from loading on that particular pile.

Most analyses of piled rafts are based on the raft being treated as a thin plate, and it is of interest to see what the effect of using thick plate theory is on the numerical predictions. Poulos et al (2001) have examined the effect of the method of modeling the raft as a thin plate who analyzed a typical problem using: firstly, a three dimensional finite element program where the raft was first modeled using thin shell theory, and then secondly, by making the raft 0.3m thick, and assigning the raft modulus to that part of the finite element mesh representing the raft. It was assumed in the analysis that there was no slip between the raft and the soil or between the piles and the soil. It was found that there was not a great deal of difference in the computed deflections for the raft, for both a stiff raft and a flexible raft.

It was concluded that the use of thin shell elements to represent the raft will lead to reasonable estimates of deflections, and therefore moments, as long as the raft is not extremely thick. Stresses in the soil will be higher for the thin shell analysis, and this effect may become important if the yield of the soil due to concentrated loads is of concern.

### 2.3.3 More Rigorous Computer Methods

#### 2.3.3.1 Two – Dimensional Numerical Analysis (FLAC)

Methods in this category are exemplified by the analyses described by Desai (1974), Hewitt and Gue (1994) and Pradoso and Kulhawy (2001). In the former case, the commercially available program FLAC has been employed to model the piled raft, assuming the foundation to be a two-dimensional (plane strain) problem or an axially symmetric three-dimensional problem. In both cases, significant approximations need to be made, especially with respect to the piles, which must be "smeared" to a wall and given an equivalent stiffness equal to the total stiffness of the piles being represented. Problems are also encountered in representing concentrated loadings in such an analysis since these must also be smeared. Unless the problem involves uniform loading on a symmetrical raft, it may be necessary to carry out analyses for each of the directions in order to obtain estimates of the settlement profile and the raft moments. As with the plate on springs approach, this analysis cannot give torsional moments in the raft.

#### 2.3.3.2 Three – Dimensional Numerical Analysis

A complete three-dimensional analysis of a piled raft foundation system can be carried out by finite element analysis (e.g. Katzenbach et al, 1998) or by use of the commercially available computer program FLAC 3D. In principle, the use of such a program removes the need for the approximate assumptions inherent in all of the above analyses. Some problems still remain, however, in relation to the modeling of the pile-soil interfaces, and whether interface element should be used. If they are, then approximations are usually involved in the assignment of joint stiffness properties. Apart from this difficulty, the main problem is the time involved in obtaining a solution, in that a non-linear analysis of a piled raft foundation can take several days, even on a modern computer running at 450 MHz. Such analyses are therefore more suited to obtaining benchmark solutions against which to compare simpler analysis methods, rather than routine design tools.

But according to Ripunjoy Deka (2014) review on different analysis methods of Piled Rafts, we found more rigorous computer methods summarized as follows.

**2.3.3.3 Hybrid Approach**

Clancy and Randolph (1993) employed a hybrid method which combined finite elements and analytical solutions. The raft was modeled by two-dimensional thin plate-bending finite elements. Piles were modeled by one-dimensional rod finite elements and the soil response was calculated by using an analytical solution. The pile was attached to a raft element at a common node, such that the vertical freedoms are common at the connected nodes. Interaction effects between all pairs of nodes are calculated using the elastic solution of Mindlin (1936) a point-to-point fashion. The major features of this hybrid finite element-elastic continuum-load transfer approach are presented in Figure 2.6. As shown on the figure (1) one-dimensional pile element; (2) lumped soil response at each pile-node-load transfer spring; (3) two-dimensional plate-bending finite element raft mesh; (4) ground resistance at each raft node represented by an equivalent spring; (5) pile-soil-pile interaction effects calculated between pairs of nodes-Mindlin's equation; (6) raft-soil-raft interaction; and (7) pile-soil-pile interaction.

The hybrid method provides a relatively rigorous and considerably more efficient method of analysis for pile rafts. Fewer equations need to be solved than for the finite element method and the time-consuming numerical integrations of boundary element method are not necessary. The hybrid method allows for variable geometry, pile stiffness, soil stiffness and raft stiffness. Although only vertical applied loading and linear elastic soil conditions are considered here, it is relatively straightforward to allow for non-linear response at the pile-soil interface or to extend the analysis to include horizontal or inclined loading. However, the method is limited to homogeneous soil.

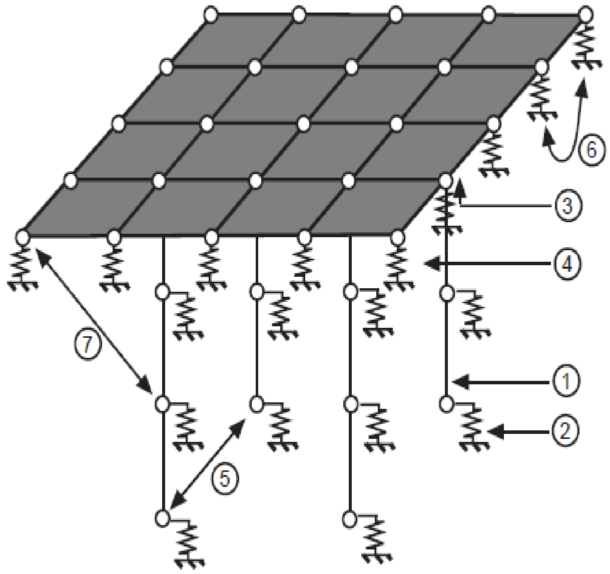



**Figure 2-6** Numerical representation of piled raft foundation (Clancy et al., 1993)

#### 2.3.3.4 Mixed Technique Approach

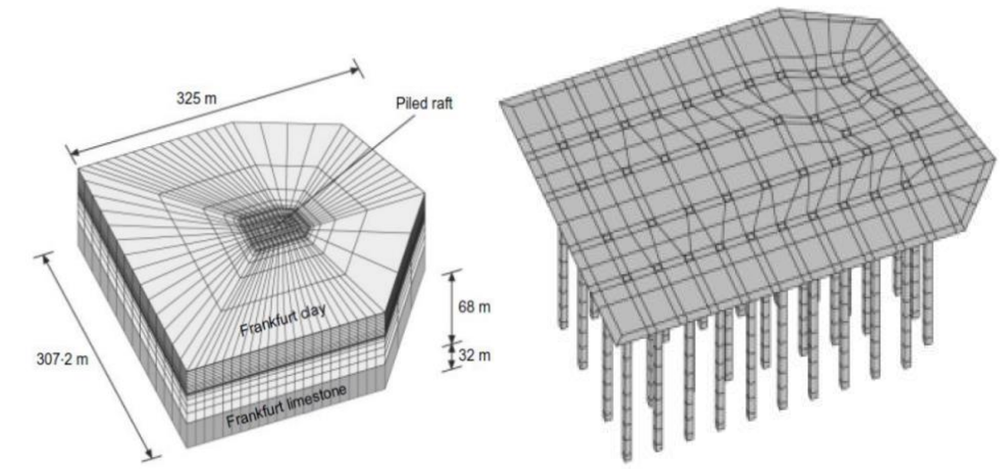
A mixed technique based on finite element method and boundary element method was developed by Franke et al. (2000) to model the three-dimensional nature of a piled raft. Plate-bending finite elements were used to model the stiffness of the raft, whereas the piles and soil were modeled by non-linear elastic springs and were attached to each node of the finite element mesh of the raft. Boundary elements at the raft-soil and pile-soil interfaces were used to model the contact pressure between the raft and soil and between the piles and soil respectively. The non-linear pile response was described by (i) a hyperbolic shear stress-shear strain relationship for the soil adjacent to the piled raft; (ii) a boundary element solution to obtain the skin friction distribution; (iii) a hyperbolic load-deformation relationship for the soil near the pile base. In the analysis, the effects of construction sequence and pile installation on the non-linear pile response were also taken into account.

A coupled boundary element and finite element formulation has been described by Mendonca and de Paiva (2003). In this approach, the bending plate is assumed to have linear elastic properties and is modeled by Finite Element Method (FEM) while the soil is considered as an elastic half-space in the Boundary Element Method (BEM). The pile is represented by a single element and the shear force along the shaft is interpolated by a quadratic function. The plate-soil interface is divided into triangular boundary element (soil) and finite elements (plate) and the sub-grade reaction is linearly interpolated across each element. The sub-grade reactions are eliminated from the FEM and BEM algebraic systems of equations, resulting in the governing system of equation for plate-pile-soil interaction problem. The system of equations from the BEM (soil and pile) and FEM (bending plate) are coupled through matrix operations. The analysis dealt with a raft subjected to vertical load only.

#### 2.3.3.5 Software Based Analysis

Reul and Randolph (2003, 2004) presented a three-dimensional elasto plastic finite element method for the analyses of piled-raft foundations. The analysis was implemented by finite-element based software code ABAQUS. The structural model based on the finite element method for this analysis is presented in Figure 2.7.

The soil and the foundation are modeled with finite elements, which allow the most rigorous treatment of the soil-structure interaction. The soil and the piles are represented by first-order solid brick elements and wedge element respectively. For the modeling of the raft, first order shell element of square and triangular shape with reduced integration has been used. Only the soil below the foundation level is modeled with finite element. The soil above the foundation level is considered through its weight. The circular piles have been replaced by square piles with the same shaft circumference. The soil is simplified to a one-phase medium instead of multiphase by considering drained shear parameters. The soil is modeled by a cap model to simulate the non-linear behavior. The cap model consists of three yield surface segments: the pressure-dependent, perfectly plastic shear failure surface; the compression cap yield surface and the transition yield surface, whereas changes of stress on the yield surfaces cause plastic deformation. The shear failure surface is perfectly plastic, whereas volumetric plastic strains cause hardening or softening of the cap. Plastic flow is defined by the non-associated flow potential of the shear surface and the associated flow potential of the cap. The cap model with yield surfaces in principal stress space and p-t plane is shown in Figure 2.8.




(a) (b)

**Figure 2-7** Problem definition. (a) Finite element mesh of the system; (b) Finite element mesh of piled raft. (Reul et al., 2003)

The interface between the raft and the soil and between the pile and the soil were modeled by thin solid continuum elements and were assumed to be perfectly rough.

Maharaja and Gandhi (2004) presented the results of three-dimensional non-linear finite element analysis of raft and piled raft foundations that have been loaded until failure.

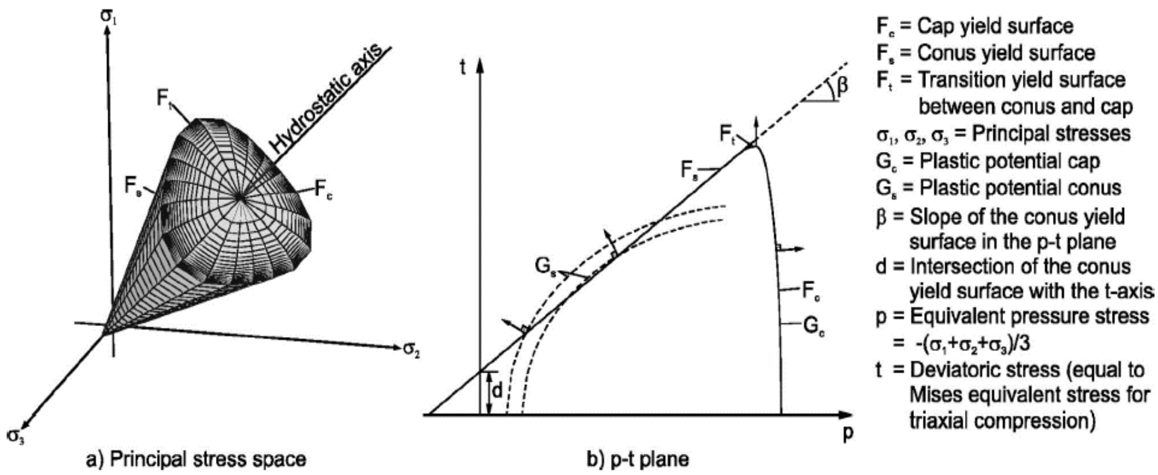



**Figure 2-8** Cap model: Yield surfaces in principal stress space and p-t plane (Reul et al., 2004)

This method combined an incremental iterative procedure with a Newton-Raphson method to solve the non-linear equations involved in a plasticity analysis. The raft, pile, and soil are discredited into eight nodded brick elements, which are shown in Figure 2.9. In this study, the raft and piles are assumed to be linearly elastic and the non-linear behavior of the soil was modeled by the Mohr-Coulomb criterion.

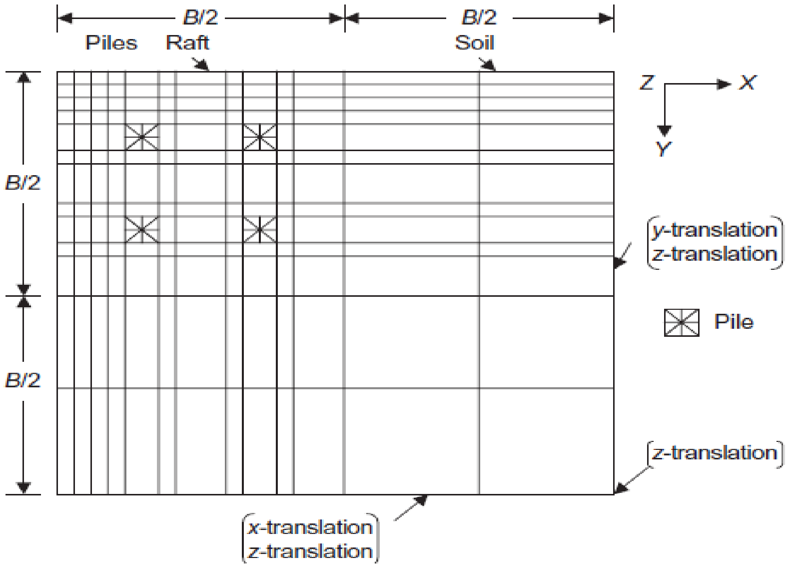



**Figure 2-9** Finite element discretization for piled raft foundation (Maharaj and Gandhi, 2004)

Some of the available methods for analysis of piled raft behavior presented by the different researcher are reviewed. All are involved numbers of simplifications in relation to modeling of the soil profile and the loading conditions. The simple methods can be implemented with minimal computer requirements with some limitation. Three-dimensional using a numerical method based software like ANSYS; ABAQUS etc. can give a more realistic solution. Analysis of foundations embedded in the non-homogeneous soil can be done by using finite element based commercial software package. However, those solutions given by Poulos(2001), Randolph (1994) are mostly used in the design analysis in piled raft foundation.

## 2.4. Settlement of Piled Raft Foundation

Settlement of piled raft foundation reviewed by Gebregziabher (2011) as follows. The settlement of piled raft foundation is dependent on the total resistance of the foundation unit, $R_{tot}$, k(s) is the sum of the resistances of all the individual piles and the raft, or equivalently:

$$R_{tot,k}(s)=\sum_{j=1}^{m}R_{pile,k,j}(s)+R_{raft,k}(s) \quad \text{(Eq.2.8)}$$

The resistance of the individual piles $R_{pile}$,k,j(s)is computed as the sum of the base and skin friction resistances:

$$R_{pile,k,j}(s) = R_{b,k,j}(s) + R_{s,k,j}(s) \quad \text{(Eq. 2.9)}$$

The resistance of the raft $R_{raft}$, k(s) can be determined by integrating the settlement dependent pressure σ(x, y) over the (raft-soil) contact area:

$$R_{raft,k}(s) = \iint \sigma(s,x,y)\,dxdy \quad \text{(Eq.2.10)}$$

The total external load $F_{tot}$, k is carried partly by the piles and partly by the contact pressure developed between the raft and the soil. The proportion of the load carried by the pile and raft is usually expressed using the CPRF coefficient $\alpha_{pr}$.

The value of $\alpha_{pr}$ varies between 0 and 1, the lowest and highest values representing the case of the unpiled raft and free-standing pile groups respectively.

The primary purpose of the piles in most CPRF designs is the reduction of settlements. In the case of large pile groups or CPRF supporting heavy structures, the optimization criteria of design are the reduction of differential settlements (Leung 2010). Hence the design of CPRF shall be in such a way that the settlements are in compliance with specifications of local and international building standards. While handling serviceability of a building, it would be necessary to consider the following settlement measures:

Maximum settlements Max: usually occurs at the midpoint of the raft of a CPRF with piles of uniform dimensions acted upon by uniform loads. In that case, its magnitude will be equal to the midpoint settlements M

$$S_{Max} = S_M$$

- Minimum settlement $S_{Min}$: usually occurs at the corner of the raft of a CPRF with piles of uniform dimensions acted upon by uniform loads, in which case its magnitude will be equal to the corner settlement $S_C$

$$S_{Min} = S_C$$

- Differential settlement ΔS is defined as the difference between the maximum and minimum settlements

$$\Delta S = S_{Max} - S_M$$

Maximum angular distortion δ/L is defined as the differential settlement between two points divided by the distance between them.

#### 2.4.1 Tolerable Raft Settlement of Buildings

Foundation settlements must be estimated with great care for buildings, bridges, towers, power plants, and similar high-cost structures. For structures such as fills, earth dams, levees, braced sheeting, and retaining walls, a greater margin of error in the settlements can usually be tolerated.

The principal components of $\Delta H$ are particle rolling and sliding, which produce a change in the void ratio, and grain crushing, which alters the material slightly. Only a very small fraction of $\Delta H$ is due to elastic deformation of the soil grains.

Settlements are usually classified as follows:

1. **Immediate**, or those that take place as the load is applied or within a time period of about seven days.

2. **Consolidation**, or those that are time-dependent and take months to years to develop.

After MacDonald and Skempton (1955) recommended maximum values in parentheses tolerable differential settlement and angular distortion of buildings.

**Table 2-1** Angular and differential settlement max limit (Bowles, 1997)

| Criterion | Rafts Max. Differential settlement, mm |
|---|---|
| Angular distortion (cracking) | 1/300 |
| Greatest differential settlement | |
| Clay | 45 |
| Sand | 32 |

### 2.4.2 Design of Settlement Reducing Piles

While designing the settlement reducing piles, some concepts should be decided to pay special attention (Poulos, 2002). These issues are listed by Poulos as follows:

- Maximum settlement
- Differential settlement
- load capacities for vertical, lateral and moment loading
- Pile loads and moments
- Raft moments and shears

In the design process, for the settlement reducing piles, special attention should be given to avoid the overdesign since it can cause as much trouble as designing under capacity (Love, 2003).

However, since piles are only there to reduce settlements, they do not contribute to the bearing capacity. Hence, the unpiled raft should be sufficient from the bearing capacity point of view and it should provide a factor of safety of at least three (De Sanctis et al., 2002).

Unfortunately, engineers have been designing the systems based on the principle that an adequate number of piles should be placed in order to carry the structural weight.

However, they should be thinking in terms of settlements and the placement should be done according to the acceptable settlement limits (De Sanctis et al., 2002). While designing the settlement reducing piles, the load sharing between raft and piles should be taken into account since the piles are not placed to provide bearing capacity. However, the settlement behavior of the raft directly changes when the piles are present so, this load sharing becomes more important. This load sharing is based on the stiffnesses of the soil and raft and the pile settlements (Love, 2003). The load sharing behavior of the system highly depends on the working conditions of the piles, i.e. the factor of safety values applied to the piles. There is some detailed analysis of case histories in the literature which give reasonable explanations about the relation between load sharing and the load carrying performances of the piles and rafts. In the paper presented by O'Neill (2005), some case histories were investigated. It has been seen that when piles are loaded to 50 percent of their ultimate capacities, the load carried by the raft drops to almost one-half of the total load whereas when they are loaded to 80 percent or above of their ultimate capacities, the piles carry a lower proportion of the load (O'Neill, 2005).

### 2.5 Advantage of Piled Raft Foundation

H.G. Poulo,(2011) in piled raft foundations piles are used as settlement control element with most of the stiffness at serviceability loads and the raft element providing additional capacity at ultimate loading. Consequently, it is generally possible to reduce the required number of piles when the raft provides this additional capacity. In addition, if there are one or more defective or weaker piles, in the subsoil the presence of the raft allows some measure of re-distribution of the load from the affected piles to those that are not affected. Another feature of piled rafts is that the pressure applied from the raft on to the soil can increase the lateral stress between the underlying piles and the soil, and thus can increase the ultimate load capacity of a pile as compared to free-standing piles (Katzenbach et al., 1998).

### 2.6 Mobilized Friction and Load Carried by Friction

The friction load transfer curve can be represented by an elastic, perfectly plastic curve. The slope of the elastic part of the friction transfer curve (Frank, Zhao 1982) is given by Eq(2.11)

$$\boxed{f = \frac{Es}{(1+v)(1+\ln(L/D))*D} * Sfriction} \qquad \text{Eq(2.11)}$$

where f is the friction stress at the interface between the pile and the soil, Es and $\nu$ are the soil modulus and the Poisson's ratio at depth z where the friction is generated, L is the embedded pile length, D is the pile diameter, and friction is the downward movement of the pile at depth z. and to Calculate the load carried in friction in the element is given by Eqn(2.12) Jean L ( 2013)

$$\boxed{Qfriction = fi\Pi D\Delta L} \qquad \text{Eq(2.12)}$$

# 3. SITE CHARACTERIZATION

## 3.1 Introduction

Site characterization is an essential part of any civil structure construction. Without adequate investigation and site characterization, any type of project can't meet its desired functions and goals. In this section, the most important material parameters of the soil and rock are determined to be used in the numerical analysis.

The soil investigation of the project sites was performed by Construction Design Share Company (2013), Best Consulting Engineering Private Limited Company (2014) and Transport Construction Share Company (2012) for United International Bank, Zemen International Bank, and Nib International Bank respectively. All of them use similar investigation techniques. Regarding the geology of the site, they describe that the areas are predominantly covered by a rock at the different weathered condition.

The sites are found around Mexico, in front of the AAU Commerce College very near to each other. Area of the piled raft foundation of Zemen Bank, United Bank, and Nib Bank are 1378.60m$^2$, 1469.79 m$^2$, and 1954.88m$^2$ respectively.

## 3.2 Field and laboratory test results

### 3.2.1 Field tests

The field test is a valuable test method that can take place on the actual field under investigation. The fieldwork and data interpretation provide information about site topography, the thickness of fill or soil layer, groundwater level, etc. The most important field tests conducted for the projects are summarized in Table 3.1.

a) Water Table Level

Piezometer is one of the major techniques that help to determine or study the groundwater level and its movement. Piezometer installation is performed with the help of core drilling. The installed piezometer has a slot at the middle portion of the pipe and blind at both ends of the pipe wall. The pipe diameter is 50 cm uPVC surrounding with filter gravel pack. The water table measurement results are shown in Appendix C.

**Table 3.1** Conducted Major Field Tests

| Field Investigation | Zemen Bank | United Bank | Nib Bank |
|---|---|---|---|
| Core drilling | Yes | Yes | Yes |
| Number of boreholes | 4 | 8 | 4 |
| Length of boring ( m) | 50 | 30 to 50 | 30 to 40 |
| SPT test | Done | Done | Done |
| Water sample | Yes | No | No |
| Groundwater measurement ( m ) | 7.60 to 9.60 | 6 to 9.9 | 1 to 8 |
| Rock Sample | Taken | Taken | Taken |

b) Standard Penetration Test

A standard penetration test is one of the most frequently used field tests to determine the strength of fine or granular soil materials. From which the bearing capacity of the foundation can be determined. The N value is the number of blow required to penetrate 450mm to the test zone by standard test hammer 63.5kg; falling from 760mm.

The standard penetration test data enable us to determine the Modulus of Elasticity and shear parameters.

**3.2.2.1 Determination of Basic Material Parameters**

Some parameters like modulus of elasticity can be not directly determined directly from site or laboratory. Correlation technique helps us to determine such values form other known values. While determining the modulus of elasticity of the projects (using correlation techniques) it was investigated that the soil investigation reports are not dependable. In this study, the value of modulus of elasticity is taken from indicative values/ presumptive values.

1. Parameters for Cohesive Soils

a) Unit Weight ($\gamma$)

Unit weight is mass volume relationship and very important parameter in geotechnical engineering. This value depends on the chemical composition and structure of the soil mineral; (Appendix C).

b) Shear Strength Parameters

Shear strength parameters (C and Ø) are determined from the direct shear test, undrained triaxial test, and unconfined compressive test. Determination of undrained shear strength $C_u$ is very important for foundation design.

Bowles refers to the following SPT N-value correlations for angle of friction Ø. Eq. (3.1) is from Shioi and Fukui (1982), who obtained them from the Japanese Railway Standards, developed for a building. This equation is used for non-rocky materials shear parameter determination.

$$\varnothing = 0.36*N_{70} + 27 \qquad \text{(Eq.3.1)}$$

The shear strength of intact rocks leads to cohesion intercepts in the range of 5 to 40MPa and friction angles in the range of 30 to 50 degrees. (Jean L., 2013). In the model on most soil layers, C and Ø values are used $30^o$ & 5Mpa for decomposed rocks, $40^o$ & 10Mpa for fractured rocks, and $45^o$ & 30Mpa for basalt rocks are used. The summary of C and Ø are shown in Table 3.2 to 3.5.

c) Initial stress coefficient, $K_o$

According to Bowles (1997), any new foundation load—either an increase (+) from a foundation or a decrease (-) from an excavation—imposes new stresses on the existing state of "locked in" stresses in the foundation soil mass. Over geological time, the stresses in a soil mass at a particular level stabilize into a steady state and strains become zero. When this occurs the vertical and lateral stresses become principal stresses acting on principal planes. This effective stress state is termed the at-rest or $K_0$ condition with $K_0$ defined as Brooker and Ireland (1965) (for normally consolidated clay) suggest (Eq.3.2).

**Table 3.2** C and Ø Values for United International Bank

| Description | Depth (m) | Angle of internal friction (Ø′) | Cohesion ( c ) KPa |
|---|---|---|---|
| Layer 1:-Dense, grey to reddish brown, with some pinkish and whitish spot, slightly GRAVEL, derived from highly weather vascular rock | 15 - 20 | 30 | 5,000.00 |
| Layer 2:-Hard reddish-brown silty clay (paleo soil) | 20-27 | 27 | 14 |
| Layer 3: -Very dense light gray to yellowish gray, sandy Gravel derived from highly weather vascular rock | 27-40 | 30 | 5,000.00 |
| Layer 4:- Medium - strong, grey, slightly weathered fractured aphanitic BASALT | 40-45 | 45 | 40,000 |

**Table 3.3 C** and Ø Values of Zemen International Bank

| Description | Depth (m) | Angle of internal friction (Ø′) | Cohesion ( c ) KPa |
|---|---|---|---|
| Layer 1:- Stiff to very stiff, brown to yellowish gray to gray, Sandy SILT with gravel ( Decomposed Rock) | 16 - 23 | 30 | 5,000.00 |
| Layer 2:- stiff to very stiff, brownish to yellowish gray to gray sand Clayey SILT with gravel ( Decomposed Rock) | 23 - 38 | 30 | 5,000.00 |
| Layer 3:- Fine-grained, gray to dark gray, slightly weathered and moderately fractured BASALT | 38- 44 | 40 | 10,000.00 |
| Layer 4: -stiff to very stiff, brownish to yellowish gray to gray sandy clayey SILT ( decomposed rock) | 44 - 50 | 30 | 5,000.00 |

**Table 3.4** C and Ø of Nib International Bank

| Description | Depth (m) | Angle of internal friction (Ø′) | Cohesion ( c ) KPa |
|---|---|---|---|
| Layer 1: -Grayish black, slightly - moderately wreathed, moderat2ly - highly fractured weak to very weak rock with some of the rock portion ( at the rim )changed to clay | 15-27 | 40 | 5,000.00 |
| Layer 2:- Light red, highly decomposed, very weak rock. Greyish black strong to moderately strong, moderately weather rock (with cavities) and highly fractured rock. | 27-30 | 45 | 40,000.00 |
| Layer3:- Dark gray, highly wreathed strong to moderately strong basalt | 30-40 | 45 | 40,000.00 |

$$K_o = 0.95\text{-}\mathrm{Sin}Ø' \qquad \text{(Eq.3.2)}$$

An equation similar to Eq. (3.2) is given by Holtz and Kovacs (1981) as

$$K_0 = 0.44 + 0.0042Ip \qquad \text{(Eq. 3.3)}$$

Where:-$I_p$ is in percent

The tabulated Initial stress coefficient values are attached in Appendix C.

d) Poisson's Ratio

Poisson's ratio is the transverse deformation characteristic of a soil which is resulted from the application of a load along the longitudinal direction. This value is very helpful in design and analysis. It is a property that describes the volume change of a material in a direction perpendicular to the application of a load. It is defined as the ratio of the axial compression to the lateral expansion of soils. Bowles (1997) recommends a range of values of Poison's ratio shown in Table 3.5. It is used for this study.

### 3.2.2.2 Major material parameters for the non-cohesive soil layers

a) Stiffness Modulus

There are several methods which are helpful for determining the stress-strain modulus of soil. These are unconfined compression tests, triaxial compression tests, and In-situ tests (SPT, CPT, Plate,-load tests) the in situ tests of SPT and CPT tend to use empirical correlations to obtain $E_s$.

The value of stress-strain modulus $E_s$ obtained from these tests is generally the horizontal value—but the vertical value is usually needed for settlements. Most soils are anisotropic, so the horizontal $E_{sh}$ value may be considerably different from the vertical value $E_{sv}$. Over-consolidation may also alter the vertical and horizontal values of the stress-strain modulus.

The standard penetration test (SPT) and cone penetration test (CPT) have been widely used to obtain the stress-strain modulus $E_s$ resulting from empirical equations and/or correlations.

The $E_s$ obtained using N values from the SPT may be more reliable than those from the CPT. Some of the Empirical correlation suggested using N value are listed in Table 3.6.

**Table 3.5** Initial Stress Coefficient for Nib Bank Project (Source Bowles 1997)

| Type of soil | μ | used | Description |
|---|---|---|---|
| Clay, unsaturated | 0.4-0.5 | 0.45 | |
| Clay, saturated | 0.1-0.3 | 0.2 | |
| Sandy clay | 0.2-0.3 | 0.25 | |
| Silt | 0.3-0.35 | 0.35 | |
| Sand, gravelly sand | - 0.1-1.00 | 0.6 | |
| commonly used | 0.3-0.4 | 0.35 | |
| Rock | 0.1-0.4 | 0.25 | depends on rock type |
| Loess | 0.1-0.3 | 0.2 | |
| Concrete | 0.15 | 0.15 | |

The German code DIN 4094-1 (2002) and DIN 4094-2 (2003)

$$E_s = \mu x P_a (\frac{\sigma_z + 0.5\Delta\sigma_z}{P_a}) w \qquad \text{(Eq.3.7)}$$

Where: -

μ = stiffness coefficient depending on values of N-SPT

w = stiffness exponent, which has a value of 0.5 for non-cohesive soil & 0.6 for cohesive soils

$\sigma_z$= overburden pressure at a depth z below the foundation level

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

$p_a$= average atmospheric pressure, taken as 101.4 kN/m², according to Hayward and Oguntoyinbo (1987)

Using the formula based on EVB (1996) E determined from Es using Eq. (3.8).

$$E = E_s \frac{(1-2v^2)}{(1-v)} \qquad \text{(Eq. 3.8)}$$

Where:-v = poison's ratio

Stiffness moduli of the projects are computed using the above correlations, but it does not fall within the range of indicative/presumptive values. Values of E for intact rock or rock substance are in the range of 2,000 MPa to 100,000 MPa (Jean L. 2013). **Table 3.6** Modules of Elasticity Empirical correlations (Bowles, 1997)

| **Type of soil** | **Es** |
|---|---|
| Sand (normally consolidated) | Es = 500($N_{cor}$ +15) |
| Sand (saturated) | Es = 250($N_{cor}$ r +15) |
| | Es = 1200($N_{cor}$ + 6) |
| Gravelly sand | Es = 600($N_{cor}$ + 6) , for N< 15 |
| | Es = 600($N_{cor}$ + 6) + 2000, for N> 15 |
| Clayey sand | Es = 320($N_{cor}$ +15) |
| Silts, sandy silt, or clayey silt | Es = 300($N_{cor}$ + 6) |

Accordingly, in the developed model, the Modulus of elasticity for the majority of decomposed rocks, fractured rocks, and basalt rocks are used 5,000Mpa, 10,000MPa, and 30,000Mpa respectively. The rest of the Martial property and modulus of elasticity are taken from the same source. (Jean L. 2013, Table 14.1-14.7). The values of E are summarized in Tables 3.7 to 3.9.

**Table 3.7.** Stiffness modulus of United bank project

| Depth(m) | Description | E (KPa) |
|---|---|---|
| 15 20 | Layer 1:-Dense, grey to reddish brown, with some pinkish and whitish spot, slightly GRAVEL, derived from highly weather vascular rock | 5,000,000.00 |
| 20-27 | Layer 2:-Hard reddish-brown silty clay (paleo soil) | 120,000.00 |
| 27-40 | Layer 3:- Dense to very dense, brownish to grayish, sandy gravel derived from highly weathered Basalt | 5,000,000.00 |
| 40-87 | Layer 4:- Medium - strong, grey, slightly weathered fractured aphanitic BASALT | 30,000,000.00 |

**Table 3.8** Stiffness Modulus of Zemen Bank Project

| Depth(m) | Description | E (KPa) |
|---|---|---|
| 15 - 22 | Layer 1:- Stiff to very stiff, brown to yellowish gray to gray, Sandy SILT with gravel ( Decomposed Rock) | 5,000,000 |
| 22 - 40 | Layer 2:- stiff to very stiff, brownish to yellowish gray to gray sand Clayey SILT with gravel ( Decomposed Rock) | 5,000,000 |
| 40 -44 | Layer 3:- Fine-grained, gray to dark gray, slightly weathered and moderately fractured BASALT | 30,000,000 |
| 44 - 87 | Layer 4 -stiff to very stiff, brownish to yellowish gray to gray sandy clayey SILT ( decomposed rock) | 5,000,000 |

**Table 3.9** Stiffness Modulus of Nib Bank Project

| Depth (m) | Description | E (KPa) |
|---|---|---|
| | Layer 7;Highly fractured rock | 18,000,000 |
| 15 24 | Layer 8: Grayish black moderately strong -strong basalt with sub-vertical and subhorizontal joints | 18,000,000 |
| | Layer 9: Grayish black, highly fractured, moderately strong to week basalt and reddish moderately to highly decomposed weak to very weak basalt | 18,000,000 |
| 24 -30 | Layer 10; Greyish black, fractured, strong rock, with cavities and highly weathered, slightly to modernly decomposed weak to strong rick with K- feldspars | 30,000,000 |
| 30 - 85 | Layer 10;Reddish,highly fractured ,slightly weathered ,strong basalt with sub vertical and sub horizontal fractures | 30,000,000 |

The above first - three layers of Nib Bank soil strata is considered as one homogeneous layer having averaged modulus of elasticity as shown. Note that the soil strata shown on each project borehole are different within a site. The soil stratifications shown on the above tables are done by combining much resembled consecutive layers.

## 3.3 General Comment on the soil investigation report

- ✓ The visual description is so confusing. One can't understand the exact nature of the material readily. Here are some of exemplifying narrations:-' red to reddish, stiff to very stiff, brownish gray', 'Fine-grained, gray to dark gray, slightly weathered and moderately fractured BASALT', 'Dense, grey to reddish brown, with some pinkish and whitish spot , slightly GRAVEL, derived from highly weather vascular rock', 'Grayish black, slightly - moderately wreathed, moderately - highly fractured weak to very weak rock with some of the rock portion ( at the rim )changed to clay.' These descriptions are very confusing. For the better understanding and proper use, it shall be specific and precisely indicative.

- ✓ Groundwater level determination is done for all the projects. But it needs some elaboration. One cannot understand whether the water is purched water or aquifer. Moreover, the water table rang of Nib bank is very wide. As the projects are big, piezometric test and installation shall be done too. It is done for Zemen bank only. The projects initial water level and the stabilized level are not indicated.
- ✓ No justification is made for the determination of the depth of boring. According to Jean L.B (2013) the depth of the total boring is typically equal to twice the foundation width below the foundation depth. Shallower borings may be accepted if a hard layer is found and confirmed to be thick enough for the project. Depths of boring commonly vary from 5m to 30 m.

  The zone of influence below the tip of a pile may be a few meters for single pile but for a group of piles, it becomes very larger. Table 3.11 shows that the depth of soil investigated below the pile tip. There is uninvestigated soil portion below the pile's tip of United and Nib banks.

The Number of boreholes according to Jean L.B (2013) a common rule for a building is to perform 1 borehole per 250 m2 of foundation surface area. The comparison of the actual number of boreholes and Jean L. and ES EN 1997: 2015 Part 2 recommendations are shown in Table 3.10.

- ✓ Some of the cohesion parameters are not in the range of indicative values.

**Table 3.10** Number of Boring based on Jean L. (2013)

| Description | Nib Bank | Zemen Bank | UnitedBank |
|---|---|---|---|
| Total are of the piled- raft (m2) | 1954.88 | 1378.6 | 1469.79 |
| Number of BH as per Jean L. | 8 | 6 | 6 |
| Actual Number of BH | 4 | 4 | 8 |

**Table 3.11** Depth of boring based on Jean L. (2031) and ES EN 1997: 2015 Part 2

| Description | Nib Bank | Zemen Bank | United Bank |
|---|---|---|---|
| Total length of basement floor 3.3m per floor | 13.2 | 13.2 | 13.2 |
| Thickness of raft,(m ) | 2 | 2.78 | 3.9 |
| total bulk excavation | 15.2 | 15.98 | 17.1 |
| Length of piles,( m ) | 24 | 28 and 20 | 28 |
| Depth of boring (avg ) | 50 | 40 | 35 |
| Investigate soil below the pile tip | 10.8 | -3.98 | -10.1 |
| Expecting depth of boring based on Jean L. recommendation 2*Wr | 86 | 84 | 70 |
| Expecting depth of boring treated as a high rise building foundation based on ES EN 1997:2015 Part 2, the max of Za≥3*bF or Za≥ 6m | | | |
| Za≥3*bF | 129 | 126 | 105 |
| Note the negative sine is uninvestigated soil layer. | | | |

# 4. FINITE ELEMENT OF PILED RAFT FOUNDATION

## 4.1 General Steps of the Finite Element Method

Analysis of simple structure by using closed form solution is simple. When the structure is complex, it is very difficult to have an algebraic equation for the entire body. But using the numerical method with the help of digital computer we can get an approximate solution. The numerical method uses discretization of the continuum structure into finite or simple elements. FEM uses the concept of polynomial interpolation. This method can readily handle complex geometries and complex loadings like point loads, element load (pressure, thermal, inertial forces) and time or frequency dependent loading.

The general step for finite element analysis is as follows:

Step 1 Discretization

Discretization is the process by which the solution region will divide into finite elements. A computer program processor plays the greater role to create the fine element mesh.

The engineering judgment determines a total number of elements used and their variation in size. The elements must be made small enough to give usable results and yet large enough to reduce computational effort; (Montaser M., 2012).

Most of the high-rise buildings often have irregular foundation geometry. The sophisticated software available for the simplification of irregular geometry of the foundation is analyzed and designed with reasonable approximations for irregular geometries. The common practice in the design and construction of deep foundations is to use a regular arrangement of piles with piles of equal diameter and length, (Gebregziabher, 2011).

Step 2 Select a Displacement Function

This step enables us to select the displacement function within each element. This is achieved by using the nodal values of the elements. Frequently used functions are a Linear function, a quadratic function, cubic polynomials function and trigonometric series, (Montaser M., 2012).

For geotechnical problems, displacements are usually taken as primary variables while stresses and strains are taken as secondary variables. The displacement components are then expressed in terms of their values at nodes. This large system of simultaneous equations has to be solved to give nodal displacements ($u_i$) using different mathematical techniques, (Gebregziabher2011).

Step 3 Define the Strain/Displacement and Stress/Strain Relationships

Strain/displacement and stress/strain relationships are necessary for deriving the equations for each finite element. In the case of one-dimensional deformation, say, in the x-direction, the strain is related to displacement u by

$$dx = \frac{du}{dx} \qquad \text{(Eq. 4.1)}$$

for small strains. In addition, the stresses must be related to the strains through the stress/strain law—generally called the constitutive law. The ability to define the material behavior accurately is most important in obtaining acceptable results. The simplest of stress/strain laws, Hooke's law, which is often used in stress analysis, is given by $\sigma_x = E\varepsilon_x$ where $\sigma_x$ = stress in the x-direction and E= modulus of elasticity; (Montaser M., 2012).

Step 4 Derive the Element Stiffness Matrix and Equations

Initially, the development of element stiffness matrices and element equations was based on the concept of stiffness influence coefficients, which presupposes a background in structural analysis. There are other alternative methods which do not require this special background, namely direct equilibrium method, work or energy methods, and methods of weighted residuals (MontaserM. 2012).

In Direct Equilibrium Method, the stiffness matrix and element equations relating nodal forces to nodal displacements are obtained using force equilibrium conditions for a basic element, along with force/deformation relationships.

In Work or Energy Methods, the stiffness matrix and equations for two- and three-dimensional elements, it is much easier to apply a work or energy method. The principle of virtual work (using virtual displacements), the principle of minimum potential energy, and Castigliano's theorem are methods frequently used for the purpose of the derivation of element equations.

Methods of Weighted Residuals are useful for developing the element equations; particularly popular is Galerkin's method. These methods yield the same results as the energy methods wherever the energy methods are applicable. They are especially useful when a functional such as potential energy is not readily available. The weighted residual methods allow the finite element method to be applied directly to any differential equation.

Step 5 Assemble the Element Equations to Obtain the Global

According to Gebregziabher (2011), assembly creates a set of global equations from separate equilibrium equations. This process is done by direct stiffness method. The resulting global equation will form

$$K. a=P \qquad (\text{Eq.4.2})$$

Where: - K= Global stiffness matrix

a=Global displacement vector

P= External force member

Assembling all the element equations is used to find the global equation system for the whole solution region. In other words, we must combine local element equations for all elements used for discretization. Element connectivities are used for the assembly process. Before solution, boundary conditions (which are not accounted in element equations) should be imposed.

This step merges the individual element nodal equilibrium equations into the global nodal equilibrium equations. Another more direct method of superposition (called the direct stiffness method), whose basis is nodal force equilibrium, can be used to obtain the global equations for the whole structure. Implicit in the direct stiffness method is the concept of continuity, or compatibility, which requires that the structure remains together and that no tears occur anywhere within the structure.

The separate element equilibrium equations are finally assembled to create a set of global equations. The assembly process is done by the direct stiffness method, where the terms of the global stiffness matrix are obtained by summing the individual element contributions taking into account the degrees of freedom which are common between elements (Gebregziabher, 2011).

Step 6 Solve for the Unknown Degrees of Freedom

The finite element global equation system is typically sparse, symmetric and positive definite. Direct and iterative methods can be used for the solution. The nodal values of the sought function are produced as a result of the solution.

Applying mathematical procedures based on solutions of governing differential equations for pile and soil displacements obtained using the principle of minimum potential energy and comparing the results with FE analysis using ABAQUS, (Seo et al. ,2009; Gebregziabher,2011)

Step 7 Solving for the Element Strains and Stresses

For the structural stress-analysis problem, important secondary quantities of strain and stress (or moment and shear force) can be obtained because they can be directly expressed in terms of the displacements determined by solving the element equations in a global direction.

Step 8 Interpreting the Results (Interpretation of results)

The final goal is to reach a conclusion based on the analysis output. This is done by settlement and stress output.

## 4.2 Finite Element Modeling of Piled Raft Foundation

Modeling of piled-raft foundations are consists of three basic material modeling. Pile, raft, and soil are the elements which should be modeled as the real elements. To simulate the whole system this section will present the modeling criteria of each element in finite element method.

#### 4.2.1. Modeling of Piles and Raft

The concrete element is similar to the 8-node (3-D Structural Solid) element with the addition of special cracking and crushing capabilities. The most important aspect of this element is the treatment of nonlinear material properties. The concrete is capable of cracking (in three orthogonal directions), crushing, plastic deformation, and creep. (Fei-ran Li, 2009).

#### 4.2.2. Modeling of Soil

The soil is considered as a solid an isotropic material, homogenous element and has elastic half-space medium. In this thesis depth of soil was defined 2.5 times pile length and its width is 3 times the width of the raft. For the analysis, all inputs for Mohr-Coulomb model are considered. The soil medium below the raft was modeled using the eight-node brick element having three degrees of freedom of translation in the x, y and z directions at each node. It is found that for the width and the thickness of the soil medium more than 2.5 times the least width of the raft foundations shows a negligible influence on the settlement and the contact pressure. The vertical translation is arrested at the bottom boundary while the lateral translation is arrested at the vertical boundary (FeiranLi, 2009).

#### 4.2.3. Soil-Pile Interaction

In 3D analysis the nature of the soil pile interaction is complicated. Therefore, to find a closed form solution is extremely difficult. According to YasserK. et al. (2014), Soil structure interaction modeled best using finite element method. This method has several advantages over other methods, (1) it allows to model different pile and soil geometries; (2) capability of using different boundary and combined loading conditions; (3) the feature of discretization allows to find solutions at each element and node in the mesh; (4) modeling of different types of soil and various material behavior for piles; and (5) ability to account for the continuity of the soil behavior. Several researchers have used the FEM to model pile-soil interaction.

## 4.3. Material Modeling

Material modeling is a governing factor for the type of structural behavior. The behavior of the material is either linear or nonlinear. Changing status, Geometric nonlinearities and/or Material nonlinearities are a source of nonlinear structural behavior.

**Changing Status** (Including Contact): many common structural features exhibition linear behaviors, that is, they are status-dependent. For example, a tension-only cable is either slack or taut; a roller support is either in contact or not in contact. Status changes might be directly related to load (as in the case of the cable), or they might be determined by some external causes. Situations in which contact occurs are common to many different nonlinear applications. Contact forms distinctive and important subset to the category of changing status nonlinearities.

**Geometric Nonlinearities:** If a structure experiences large deformations, its changing geometric configuration can causeth structure to respond nonlinearly. Geometric nonlinearity is characterized by "large" displacements and/or rotations.

**Material Nonlinearities:** Nonlinear stress-strain relationships are common causes of nonlinear structural behavior. Many factors can influence a material's stress-strain properties, including load history (as in elastoplastic response), environmental conditions (such as temperature), and the amount of time that a load is applied (as in creep response), (ANSYS, 2011).

### 4.3.1. Material Modeling of Pile and Raft

The concrete material model predicts the failure of brittle materials. Both cracking and crushing failure modes are accounted for. The criterion for failure of concrete due to a multiaxial stress state can be expressed in the form (Willam and Warnke).

$$\frac{F}{f_c} - S \geq 0 \qquad \text{(Eq. 4.3)}$$

Where: F = a function (to be discussed) of the principal stress state ($\sigma_{xp}$, $\sigma_{yp}$, $\sigma_{zp}$);

S = failure surface expressed in terms of principal stresses and five input parameters $f_t$, $f_c$, $f_{cb}$, $f_1$ ,$f_2$;

$f_c$= uniaxial crushing strength;

( $\sigma_{xp}$, $\sigma_{yp}$, $\sigma_{zp}$ )= principal stresses in principal directions.

If previous Equation is satisfied, the material will crack or crush.

#### 4.3.2. Material Modeling of Soil (Mohr-Coulomb Method)

Mohr-Coulomb is the most known law for modeling a soil mass and it is one of the constitutive models found in ABAQUS material modeling library. More engineering calculations are done using this method as it is a well-developed method.

Mohr-Coulomb model is a perfectly elasto-plastic model. Elastic behavior and plastic behavior of material defined this law. The model needs four imputes parameters in ABAQUS: friction angle, dilatation angle, cohesion yield stress, and absolute plastic strain and sometimes use temperature for temperature dependent materials.

#### 4.3.3. Material Modeling of Contact Element (surface to surface)

Thin Layer Method (TLM) allows replacing the two semi-infinite media, on the left and right (top and bottom) of a central domain of interest; by equivalent nodal forces simulating their effect. Those are deduced from an eigenvalue problem formulated in the two semi-infinite lateral media.

The Thin Layer Method (TLM) or the consistent transmitting boundary condition method, developed by Waas, is adopted in this work to simulate wave propagation in unbounded soil media. The problem is formulated in the frequency domain under plane strain conditions. The domain is divided into three regions. The region of interest usually contains foundations, tunnels, wave barriers or surface structures, which is the focus of analysis. It is called the irregular soil domain. This zone will be treated by the finite element method and it will be discretized into structured or unstructured mesh grids with two degrees of freedom per node. The base of the model is assumed in the current analysis to be horizontal and resting on rigid bedrock; (Nawras Hamdan, 2013).

According to Potts and Zdravkovic (2001), it is preferable not to use interface elements for the case of vertical loading. Experience at the Institute of Geotechnics of TU Darmstadt has shown that the use of very thin continuum elements for modeling soil-structure interaction, which is also known as ideal contact, is more suitable for deep foundations with bored piles, due to the roughness of the surfaces of the soil and concrete. Owing to its wide application for similar problems (Reul 2004, Katzenbach et al. 2009, and Katzenbach et al. 2009) the ideal contact concept has been made use of in this study too. Since the width of the shaft element affects the resistance - settlement relationships of the foundation, it shall be adjusted using numerical analyses (Katzenbach et al. 2007). Continuum elements of thickness 0.1D at the pile-soil interface have been applied throughout the numerical studies as recommended by Reul (2000) and de Sanctis and Mandolini (2003, 2006); cited by Gebregziabher,( 2011).

### 4.4. Modeling of Piled Raft Foundations Using Abaqus

#### 4.4.1. Introduction

This chapter explains how to model piled raft foundations using ABAQUS, which is one of the powerful finite element program software. 3D numerical modeling involves considerably more elements and therefore more nodes and interaction points than 2D modeling. The modeling process is started by defining the element materials for soil and concrete.

#### 4.4.2. Modeling of Piled-Raft Foundations

The next sub-topics cover the procedure of modeling of all elements of pile-raft foundation in ABAQUS. This covers; modeling of piles, raft, and soil layers, pile-soil contact, raft-soil contact, mesh generation, constraints definition, and load application.

#### 4.4.2.1. Modeling of RC Piles

Modeling of a pile is a two-parameter output; the length and the diameter. ABAQUS has 8-node linear brick material which is used for the 3-D modeling of solids materials. In addition, its feature is solid deformable type.

In a preprocessor such as ABAQUS, we define the model geometry rather than the nodes and elements; when we mesh the geometry, the preprocessor automatically creates the nodes and elements needed for the analysis.

#### 4.4.2.2. Modeling of RC Raft

The RC raft in this study is the same element as pile modeled using ABAQUS node linear brick material which is used for the 3-D modeling of solids. The raft can be modeled by defining its coordinate and at last, it becomes a three parameter output; the length, width, and thickness.

#### 4.4.2.3. Modeling of Soil

The soil element in the current study is modeled using in ABAQUS as a deformable solid element. Model of the soil contains the different soil layers of the projects.

For 3D modeling of the soil 8-node brick material is used. This eight nodded brick element has three degrees of freedom at each node. That is translations in the nodal x, y, and z directions, Mohr-Coulomb method is used to define the material properties of soil in this study.

#### 4.4.2.4. Modeling of Pile-Soil Contact

The substructure and superstructure load transferred to the soil is predominantly by skin friction of the piles. Investigation of soil-pile interaction has been studied in the past four decades through physical and numerical modeling ;( Gebregziabher, 2011).

Due to the roughness of the surfaces of the soil and concrete use of very thin continuum elements for modeling soil-structure interaction is more suitable for deep foundations with bored piles. Continuum elements of thickness 0.1D at the pile-soil interface have been applied throughout this study.

In ABAQUS a contact surface was modeled using thin layer method. It is used to model the interaction contact surface between the pile shaft external surfaces and the surrounding soil. These elements have only displacement degrees of freedom at their nodes. The contact surface thickness is 0.1*D (D= diameter of the pile)

#### 4.4.2.5. Modeling of Raft-Soil Contact

"Loads from superstructures are transferred to the soil predominantly through skin friction of the piles. Thus the contact between the piles and soil shall be handled carefully. The pile-soil contact has been investigated in the past four decades by physical and numerical modeling.

Kulhawy and Peterson (1979), Potynody (1961), Uesugi et al. (1990), Katzenbach et al. (2000) carried out extensive experiments to study the contact behavior between concrete and soil. Based on such in-situ, field, laboratory and numerical experiments, the friction laws for the contact between soil and concrete were formulated by different researchers. Most of the idealizations were based on simple Mohr-Coulomb linear models which cannot represent the hardening and softening behaviors. Wriggers and Haraldsson (2003) formulated mathematical elasto-plastic models for the concrete-soil contact based on the real shear test experiments of Katzenbach et al. (2000), which was later checked experimentally by Katzenbach and Turek (2003). Haraldsson (2004) extended these works by performing sensitivity analysis to propose a better contact formulation and simulated with FEM, (Arslan 1994).

There are different methods of idealizing the contact between the soil and concrete foundation unit. Some of these idealizations were summarized by Reul (2000) as follows:

According to Potts and Zdravkovic (2001), it is preferable not to use interface elements for the case of vertical loading. Experience at the Institute of Geotechnics of TU Darmstadt has shown that the use of very thin continuum elements for modeling soil-structure interaction, which is also known as ideal contact, is more suitable for deep foundations with bored piles, due to the roughness of the surfaces of the soil and concrete. Owing to its wide application for similar problems (Reul 2004, Katzenbach et al. 2009, and

Katzenbach et al. 2009) the ideal contact concept has been made use of in this study too. Since the width of the shaft element affects the resistance - settlement relationships of the foundation, it shall be adjusted using numerical analyses (Katzenbach et al. 2007). Continuum elements of thickness 0.1D at the pile-soil interface have been applied throughout the numerical studies as recommended by Reul (2000) and de Sanctis and Mandolini (2003, 2006) cited by Gebregziabher, (2011).

The contact surface between raft and soil was modeled using a thin layer method.

### 4.5. Mesh Generation

Generating a mesh is the process of dividing the domain into many subdomains (Finite elements). In FE the accuracy of the domain depends on the nature of the mesh. The finer mesh is the more accurate the result will be, but it consumes longer period of analysis.

The raft, pile, and soil system assembled before the orphan mesh is applied. This mesh is applied based on the principle of symmetry. Based on symmetry principle, the foundation systems are modeled as bi-symmetrical and uni-symmetrical systems before applying boundary condition.

### 4.6. Soil Mass Boundaries

Boundary condition used values of basic solution variables like displacement. The infinite soil mass outside the modeled soil mass is not considered.

The boundary condition is a proper constraint on a mesh. The nodes on the periphery of the symmetrical mesh are fixed against displacement in the horizontal direction but remain free to displace in the vertical direction. Moreover, the bottom of the mesh is fixed against displacement in all directions. The boundary of a model is placed far from the reign of interaction in order not to affect the displacement.

Abaqus has boundary condition to solve unknowns. Due to symmetry, the models are one-fourth of the model and one half of the model simulated and then boundary condition is applied on the mid-axis.

The boundary conditions in this study are (a) the horizontal boundary of the soil was placed at least 3 times the raft width. (2) The vertical boundary was placed 70 to 110m from the bottom of the raft.

### 4.7. Model Implementation

For the interest of time, the developed models are bidirectional symmetry (Quarter of the system is modeled) and unidirectional symmetry (Half of the system is modeled). Symmetric modeling also reduces the modeling time and the result can be interpreted in a simple way. Advantages of symmetry modeling are; simplify result readings, and save time and effort at modeling.

**Table 4.1** Actual Designed Dimension of the Piled Raft Foundation

| Description of the actual piled raft foundation | Nib Bank (Unidirectional symmetry) | Zemen Bank (Bidirectional symmetry) | United Ban (Unidirectional symmetry) |
|---|---|---|---|
| Breadth of raft, B ( m ) | 34.79 | 49.96 | 42.21 |
| Length of raft ,L ( m) | 43.23 | 41.95 | 39.50 and 35.72 |
| Thickness of raft, tr ( m ) | 2 | 2.78 | 3.9 |
| Length of piles ,Lp( m ) | 24 | 28 and 20 | 28 |
| Diameter of piles ( m ) | 0.8 | 0.8 and 0.6 | 0.8 |
| Pile spacing | 3*D = 2.4 | 3*D = 2.4 | 3*D = 2.4 |
| Number of piles ,n | 240 | 358 and 36=394 | 300 |

### 4.8. Application of Loading

The static structural loads in this study are applied as a uniformly distributed load. The loads are determined from the structural load of the buildings, imposed loads and transient/live loads. As shown in Table 4.3 it is very large to accept and to use. (detail calculation is shown in the appendix C.7). The unfactord structural load is between 16.35KN/m$^2$ to 19.91 KN/m$^2$.

For this study, 12KN/m$^2$ uniformly distributed load is used. On this base unfactored and factored loads become 420KN/m$^2$ and 582 KN/m$^2$ respectively.

**Table 4.2** Modeled Dimension of the Soil-Piled- Raft System

| Description piled raft foundation model | Nib Bank (Unidirectional symmetry) | Zemen Bank (Bidirectional symmetry) | United Bank (Unidirectional symmetry) |
|---|---|---|---|
| Breadth of raft, B ( m ) | 35 | 27 | 35.72 |
| Length of raft ,L ( m) | 21.6 | 21.5 | 21 |
| Thickness of raft, $t_r$ ( m ) | 2 | 2.78 | 4.03 |
| Length of piles ,$L_p$( m ) | 24 | 28 and 20 | 28 |
| Diameter of piles,D ( m ) | 0.8 | 0.8 and 0.6 | 0.8 |
| Pile spacing | 3*D = 2.4 and 6*D = 4.8 | 3*D = 2.4 and 6*D = 4.8 | 3*D = 2.4 and 6*D = 4.8 |
| Number of piles ,n | 126 for 3D and 35 for 6D | 99 for 3D and 31 for 6D | 135 for 3D and 40 for 6D |
| No of pile Difference from actual | 92 | 68 | 95 |
| Model type | Unidirectional symmetry | Bidirectional symmetry | Unidirectional symmetry |
| Breadth of soil, B ( m ) | 105 | 85 | 108 |
| Length of soil ,L ( m) | 65 | 65 | 63 |
| Thickness of soil, $t_S$ ( m ) | 70 | 110 | 72 |

**Table 4.3** Applied Unfactored and Factored Loads (hand calculation)

| Project | Total Unfactord Load applied on the Raft (KN/m2) | Total Factored Load applied on the Raft (KN/m2) |
|---|---|---|
| Zemen Bank | 595.04 | 810.45 |
| United Bank | 611.07 | 813.29 |
| Nib Bank | 615.85 | 837.5 |

### 4.9. Step-By-Step Analysis

Simulating the actual construction process for finite element method is very important to attain reasonable results. This is achieved by step-by-step analysis. These are:

1. Initial Stress Condition:

Initial conditions are specified for particular nodes or elements as appropriate. The data can be provided directly, in an external input file or in some cases, by a user subroutine or by the results or output database file from a previous ABAQUS analysis. The in-situ stress state of the soil mass is simulated in this first step. (it helps to define the equilibrium condition of internal stress )

2. Primary Load

This step is used to consider the primary existed soil mass that was excavated to create the basement stories.

3. Excavation:

Excavation of the soil for pile installation is modeled by removing the soil from the bottom of the raft to the bottom of the pile. In this step, the soil below the raft is excavated to full depth of the pile (up to 20m, 24 m and 28m below the bottom of the raft).

4. Pile Installation:

Installation of the foundation piles is simulated here.

5. Raft installation:

The simulation of the application of weight and stiffness of the raft are handled in this step.

6. Loading:

At last, factored or unfactord loads of the substructure (except the raft load) and superstructure loads are applied as uniformly distributed load acting on top of the raft.

Note 1) Pressure relief due to excavation is considered in the analysis. As modeling is started -19m below the natural ground level, the soil above it is imposed in the form of uniformly distributed load. Then it is excavated (deactivated) in the excavation step.

2) The pile head and the raft are rigidly connected.

# 5. 0 ANALYSIS OF RESULTS

## 5.1 FEM Analysis Result

### a) Settlement Analysis Result

Settlement analysis result for unfactord Load condition is showed in Figures 5.1 to 5.6. It is the vertical displacement of the system on the region where the load is applied. Applied load magnitude is 12KN/m$^2$ and the total load applied on the raft is 420 KN/m$^2$. The maximum displacement results are summarized in Table 5.1- 5.3.

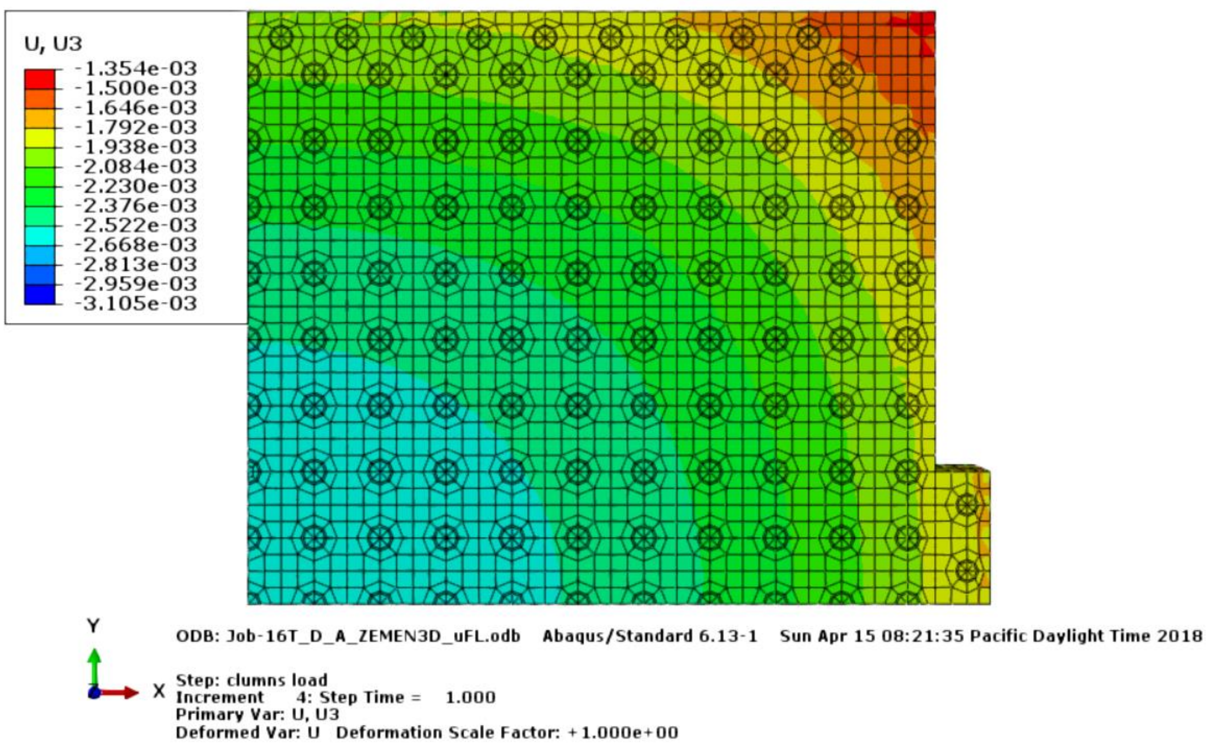



**Figure 5.1.** Settlement Analysis Result of Zemen Bank -Pile Spacing 3D

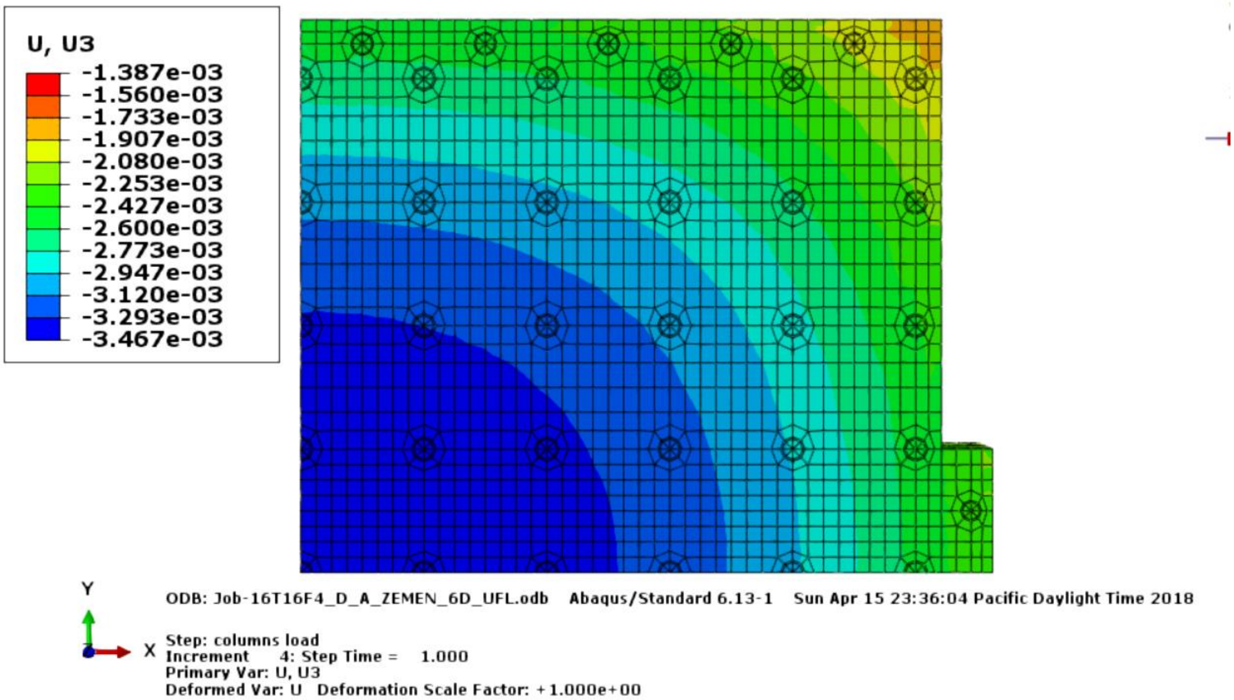



**Figure 5.2** Settlement Analysis Result of Zemen Bank -Pile Spacing 6D

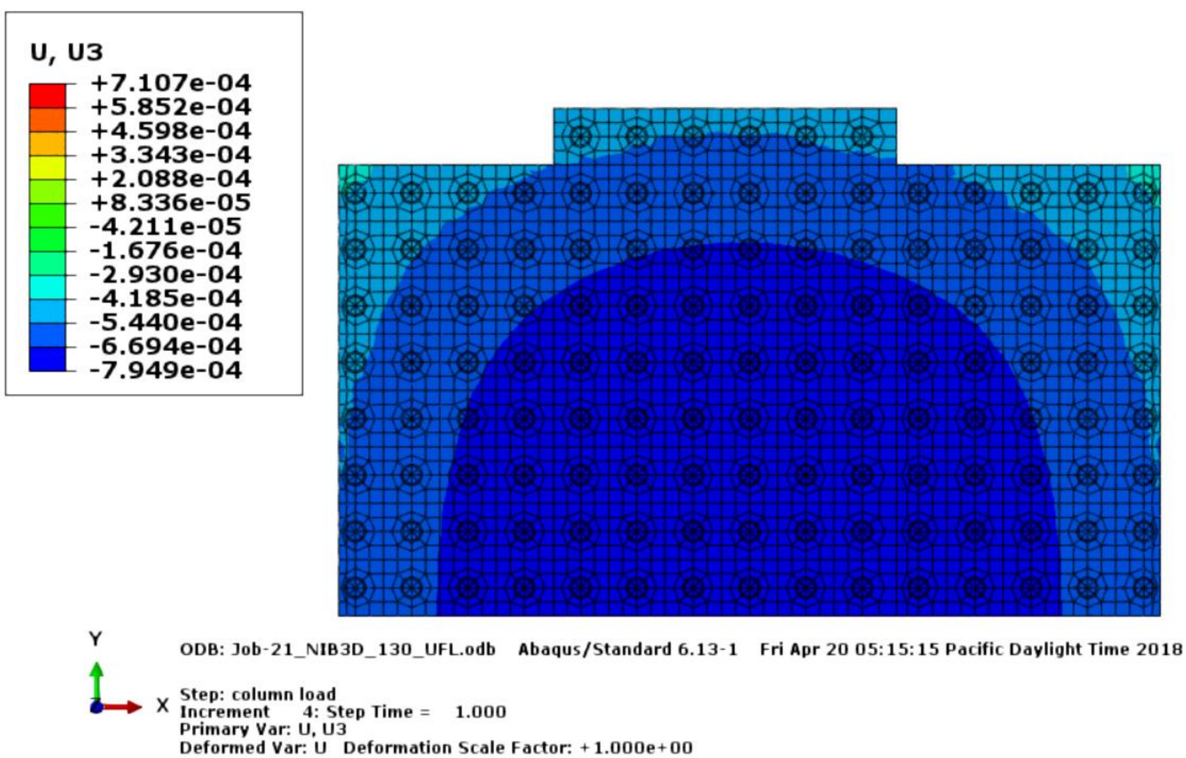



**Figure 5.3.** Settlement Analysis Result of Nib Bank- Pile Spacing 3D

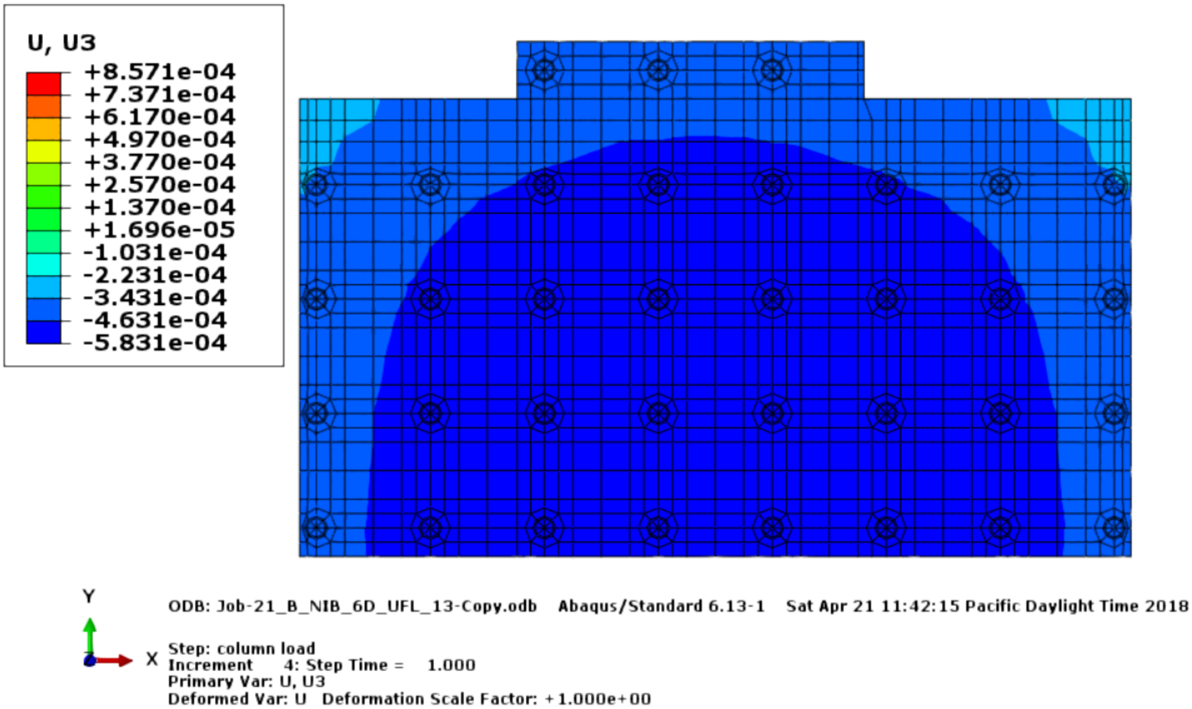



**Figure 5.4** Settlement Analysis Result of Nib Bank - Pile Spacing 6D

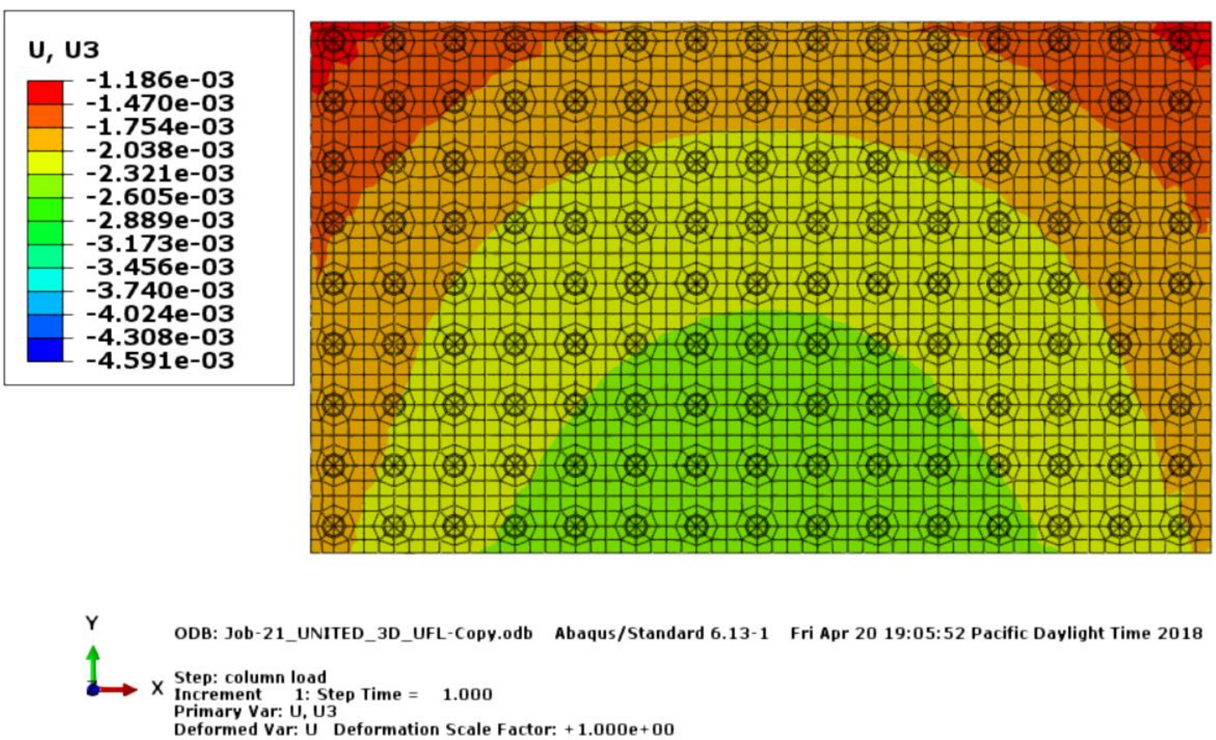



**Figure 5.5** Settlement Analysis Result of United Bank - Pile Spacing 3D

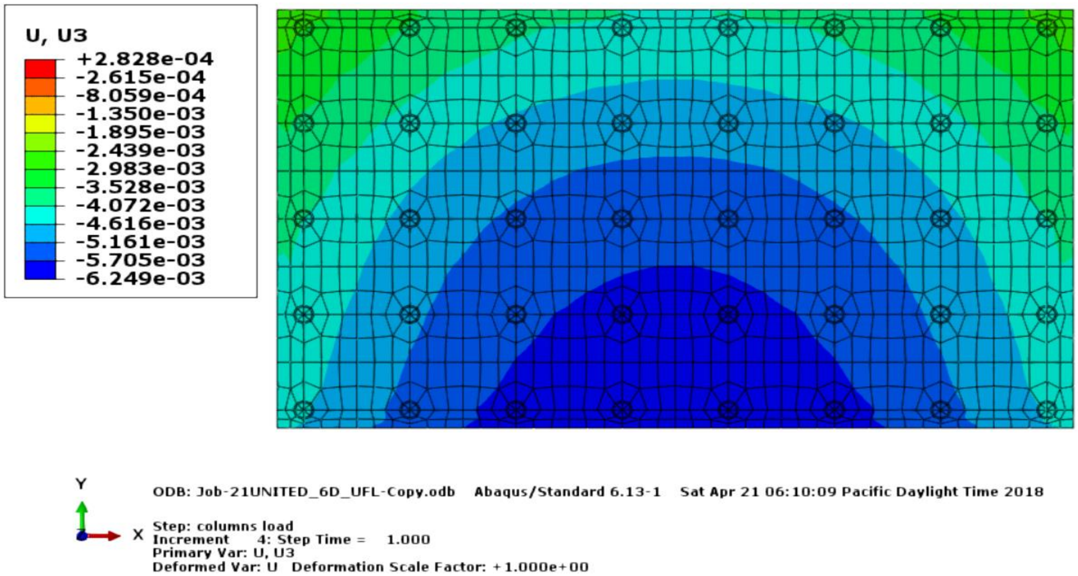



**Figure 5.6** Settlement Analysis Result of United Bank- Pile Spacing 6D

### 5.1.1. Differential Settlement and Angular Distortion

After the analyzing the piled raft foundations, it is found that the differential settlement varied from 0.05mm to 4.61 mm. Differential settlement and angular distortion results for 3D and 6D pile spacing are shown below in Table 5.1. As per MacDonald and Skempton (1955) recommendation, all the projects differential settlement and angular distortion are within tolerable limit.

**Table 5.1** Differential settlement and angular distortion

| Project | Pile spacing | Differential Settlement (mm) | Difference of Differential Settlement (mm) | Angular Distortion | Remark |
|---|---|---|---|---|---|
| Zemenk Bank | 3D | 2.25 | 0.11 | 7.88E-04 | L= 32.97m |
| | 6D | 2.6 | | 7.55E-04 | |
| Nib Bank | 3D | 0.05 | 0.46 | 0.00196 | L=25.74 |
| | 6D | 0.51 | | 0.0198 | |
| United Bank | 3D | 3.41 | 1.2 | 0.001703 | L=25.43M |
| | 6D | 4.61 | | 1.77E-05 | |

### 5.1.2. Settlement of Piled Raft and Raft Alone

The settlements of the piled raft foundation for both spacing and settlement the raft alone is between 0.079 to 4.67mm and 3.02 to 9.71mm respectively. (See Table 5.2.) The settlement response of piled raft foundation for both pile spacing is almost the same. The settlement of raft alone is insignificant too.

**Table 5.2** Maximum Settlement of Piled Raft Foundation

| Project | Pile spacing | Max. Settlement of piled raft (mm) | Max. Settlement of raft alone (mm) |
|---|---|---|---|
| Zemen Bank | 3D | 3.05 | 8.97 |
| | 6D | 3.47 | |
| Nib Bank | 3D | 0.079 | 3.024 |
| | 6D | 0.583 | |
| United Bank | 3D | 4.59 | 9.71 |
| | 6D | 4.67 | |

### 5.1.3. Load Sharing Between the Pile and the Raft

The uniformly distributed structural load is carried by the raft and the piles together. From the stress output and area of piles; the load carried by the piles is easily determined. Table 5.3. depicts that the load carried by raft is 3% to 15% and 24% to 39% for 3D and 6D pile spacing respectively.

**Table 5.3** Load Sharing Between the Pile and the Raft (%)

| Part | Zeme Bank 3D | Zeme Bank 6D | United Bank 3D | United Bank 6D | Nib Bank 3D | Nib Bank 6D |
|---|---|---|---|---|---|---|
| Raft | 10% | 39% | 9% | 24% | 3% | 25% |
| Pile | 90% | 61% | 91% | 76% | 97% | 75% |

**B) Normal Stress on the Pile**

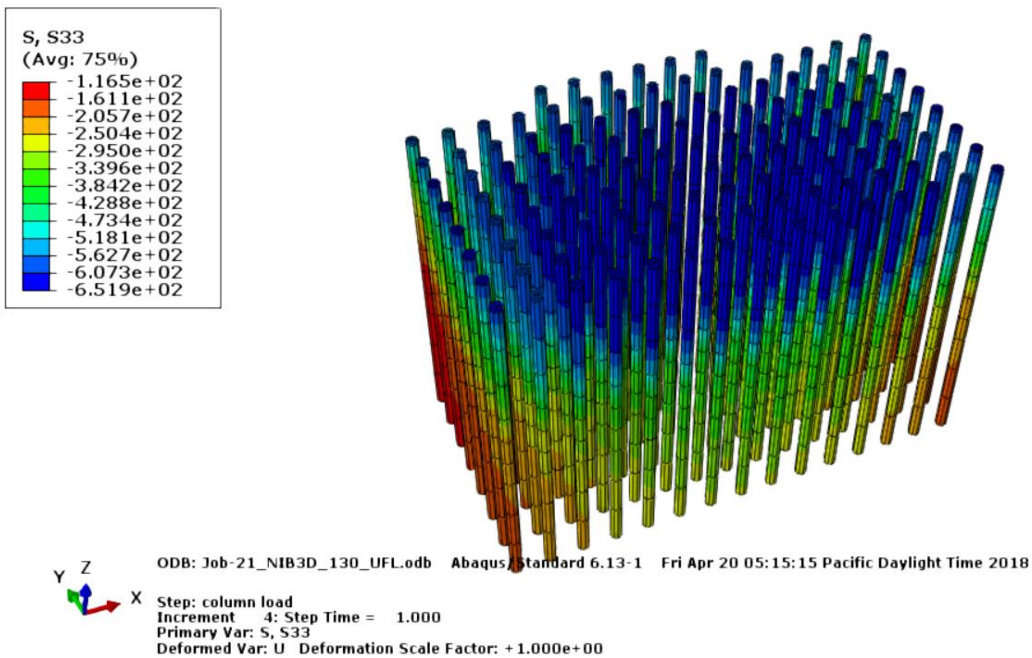



**Figure 5.7** Normal Stress Result of Nib Bank- Pile Spacing 3D

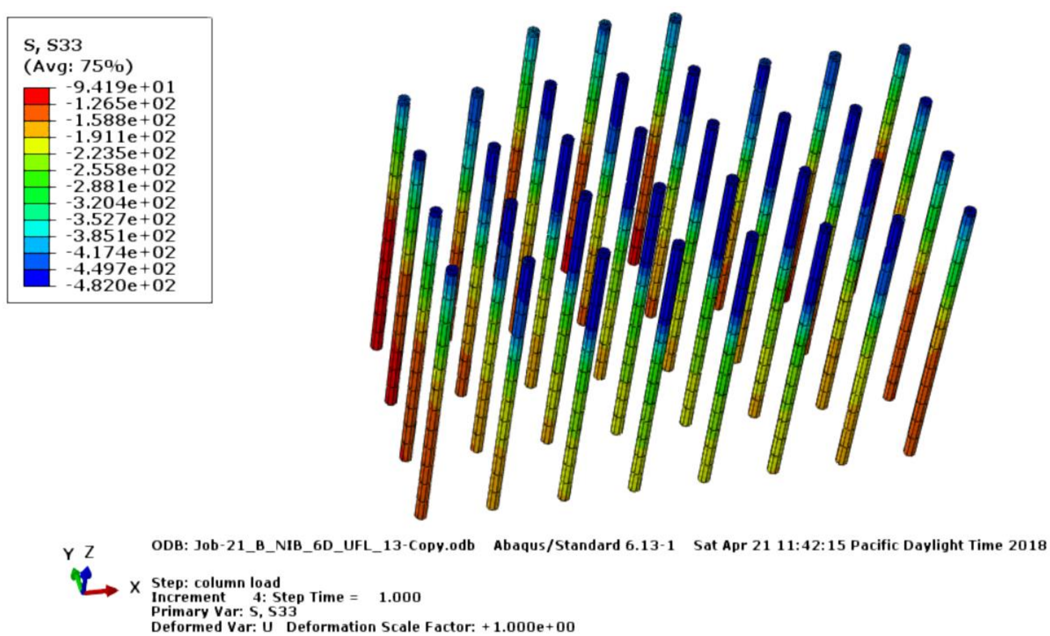



**Figure 5.8** Normal Stress Result of Nib Bank- Pile Spacing 6D

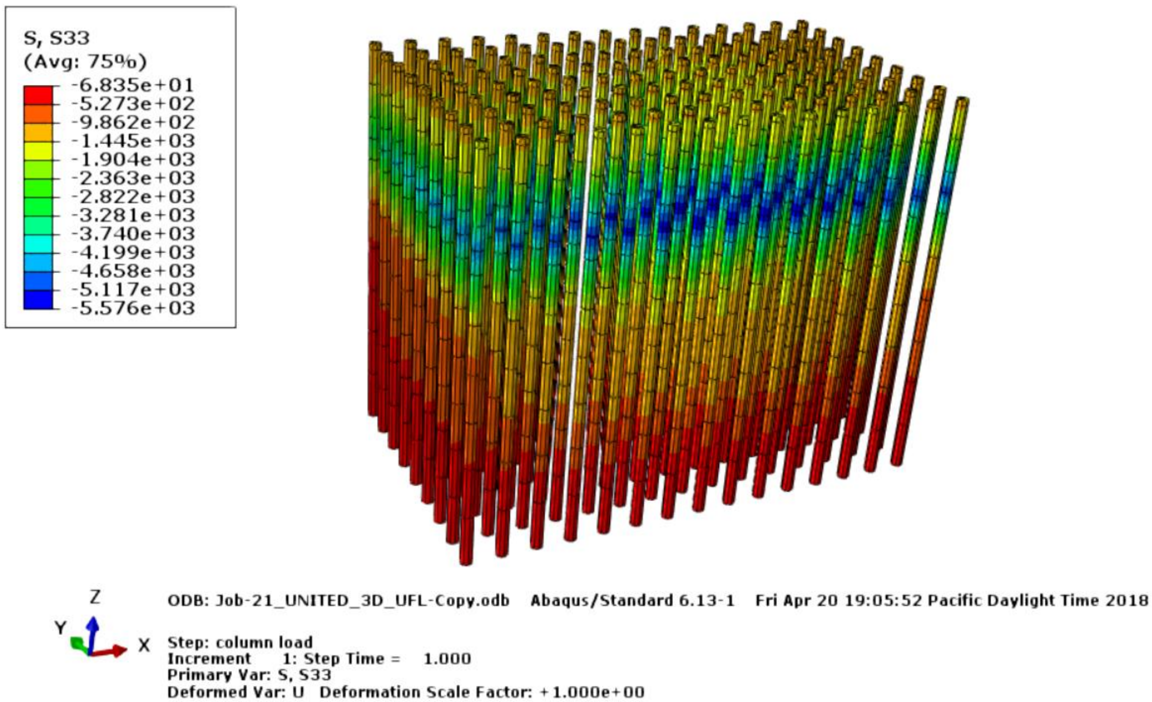



**Figure 5.9** Normal Stress Result of United Bank- Pile Spacing 3D

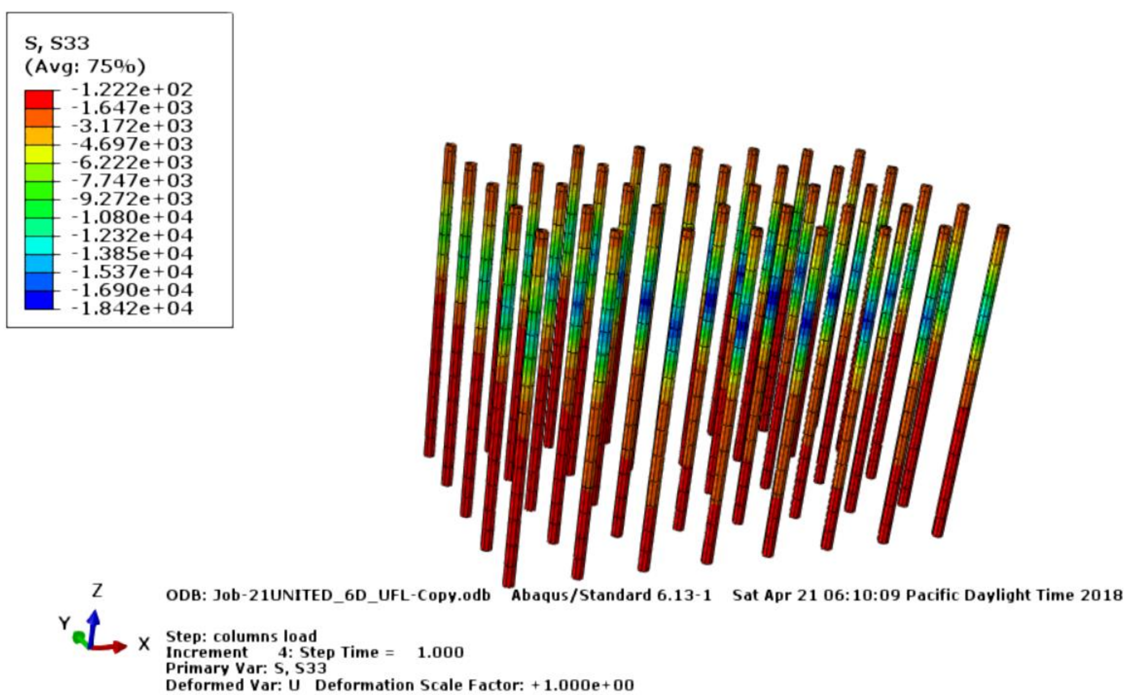



**Figure 5.10** Normal Stress Result of United Bank- Pile Spacing 6D

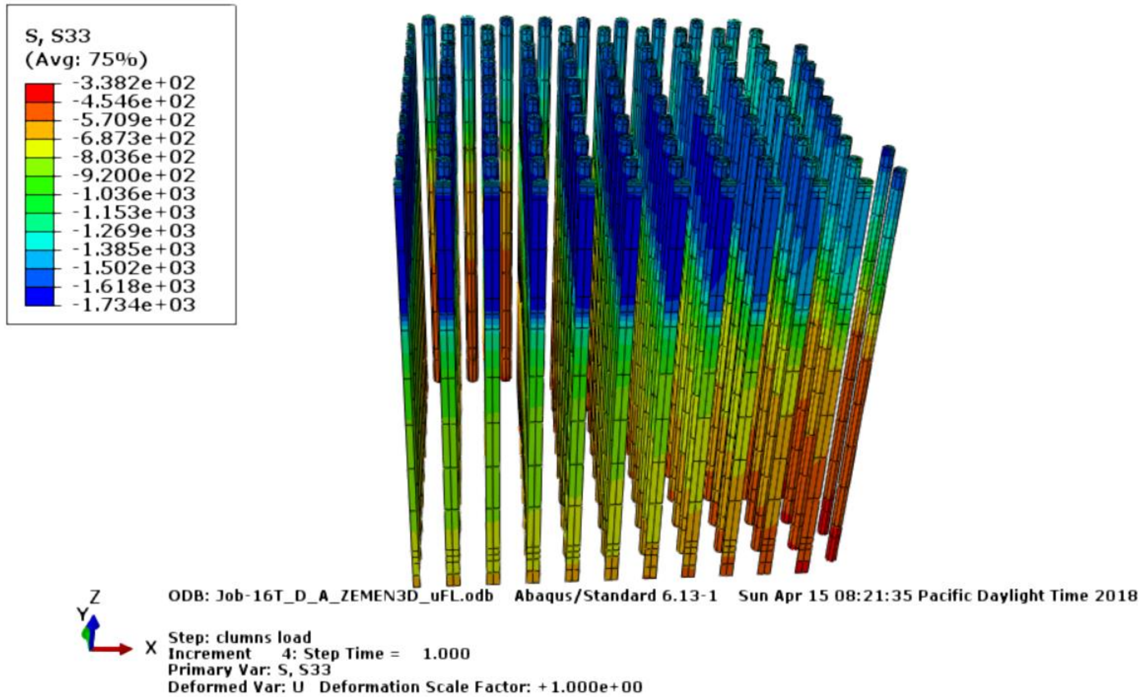



**Figure 5.11** Normal Stress Result of Zemen Bank- Pile Spacing 3D

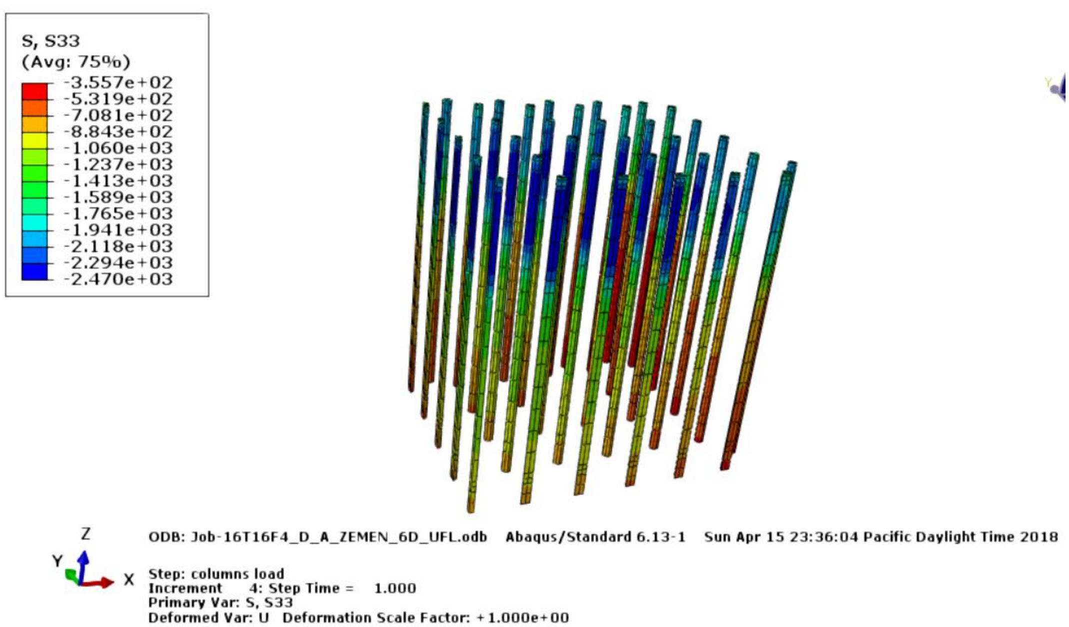



**Figure 5.12 Normal Stress Result of Zemen Bank- Pile Spacing 6D**

#### 5.2.3 Load Settlement Graph (for factored load)

To study the structural load (Vertical load) settlement nature; factored load is applied on respected foundations having 3D and 6D pile spacing. After the analysis; the output data is extracted from the modes using the unique nodal command. For each increment load respective settlement is collected and the graphs are drawn as showmen on figure 5.1 to 5.6. Location of the settlement measurement is taken at the center of the foundation.

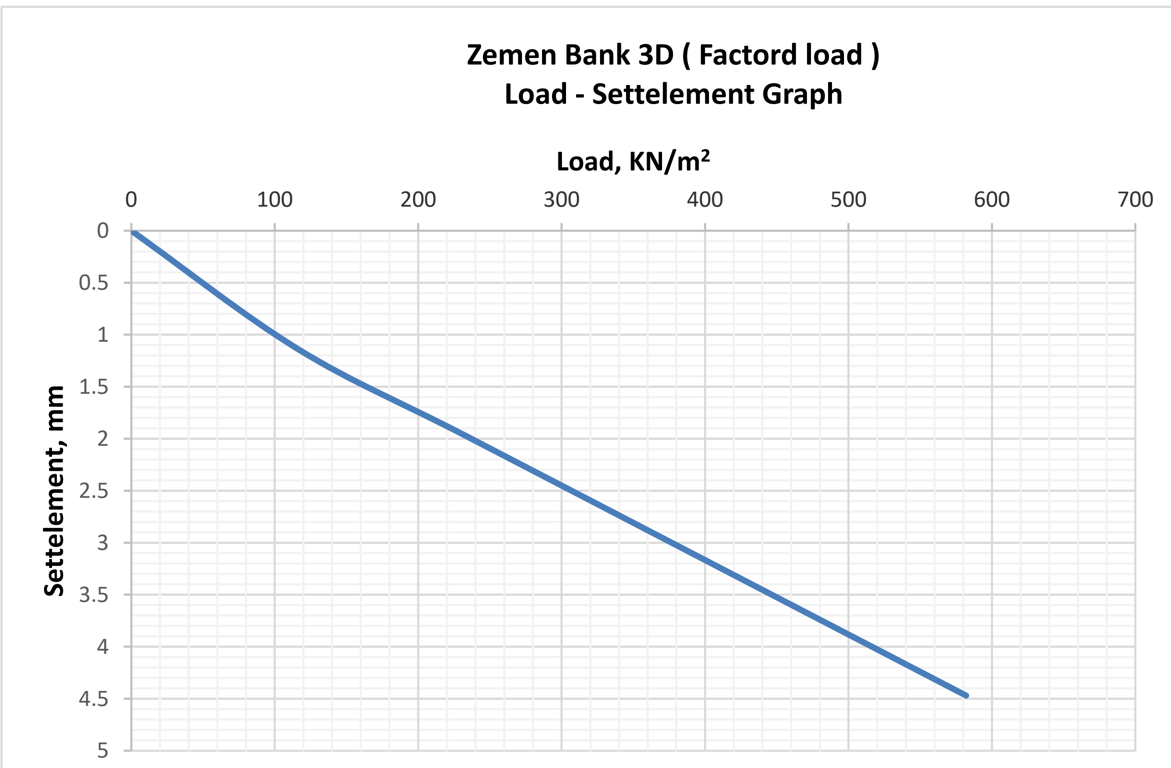



**Fig.5.13.** Zemen Bank 3D Load settlement Curve

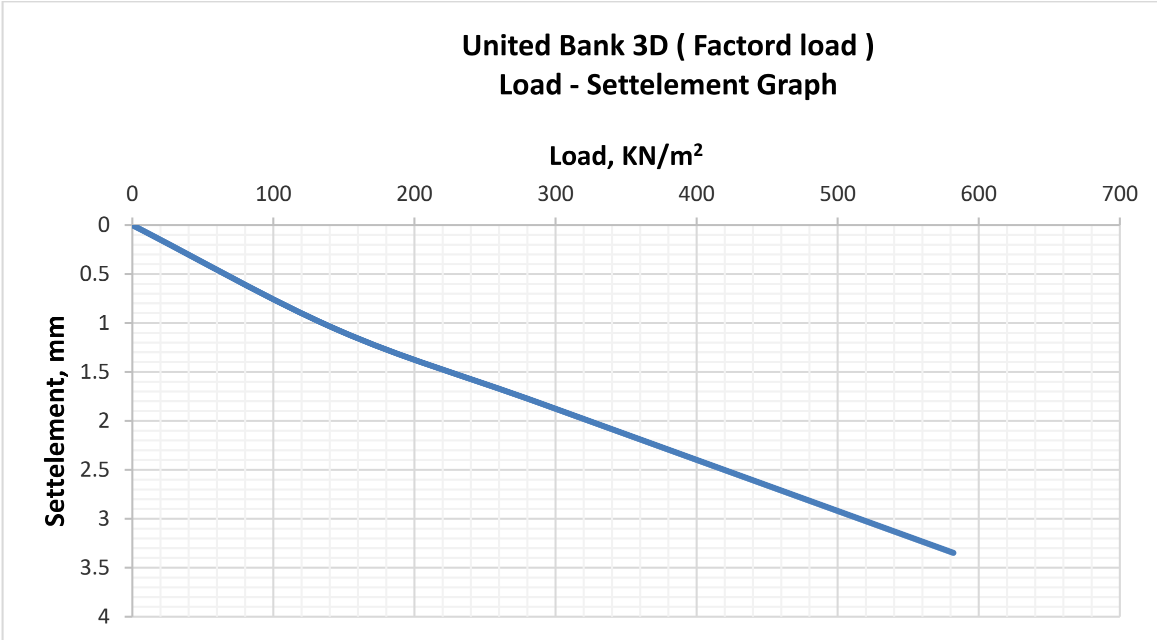



**Fig.5.14.** United Bank 3D Load settlement Curve

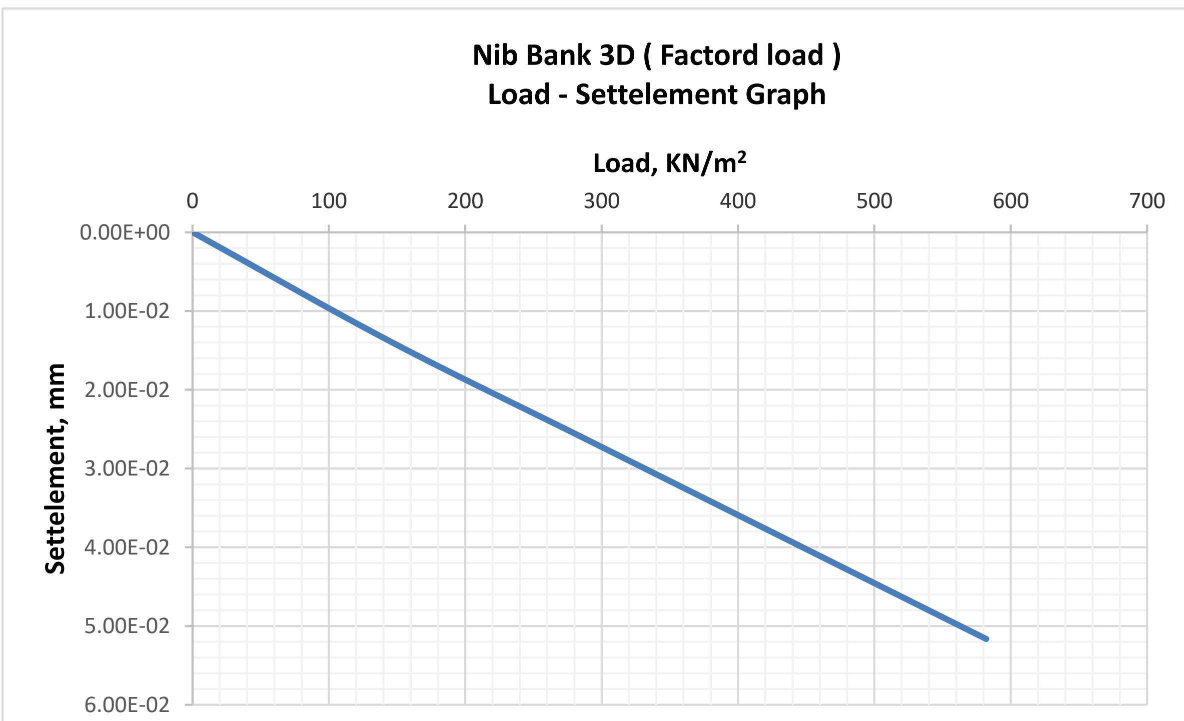



**Fig.5.15.** Nib Bank 3D Load settlement Curve

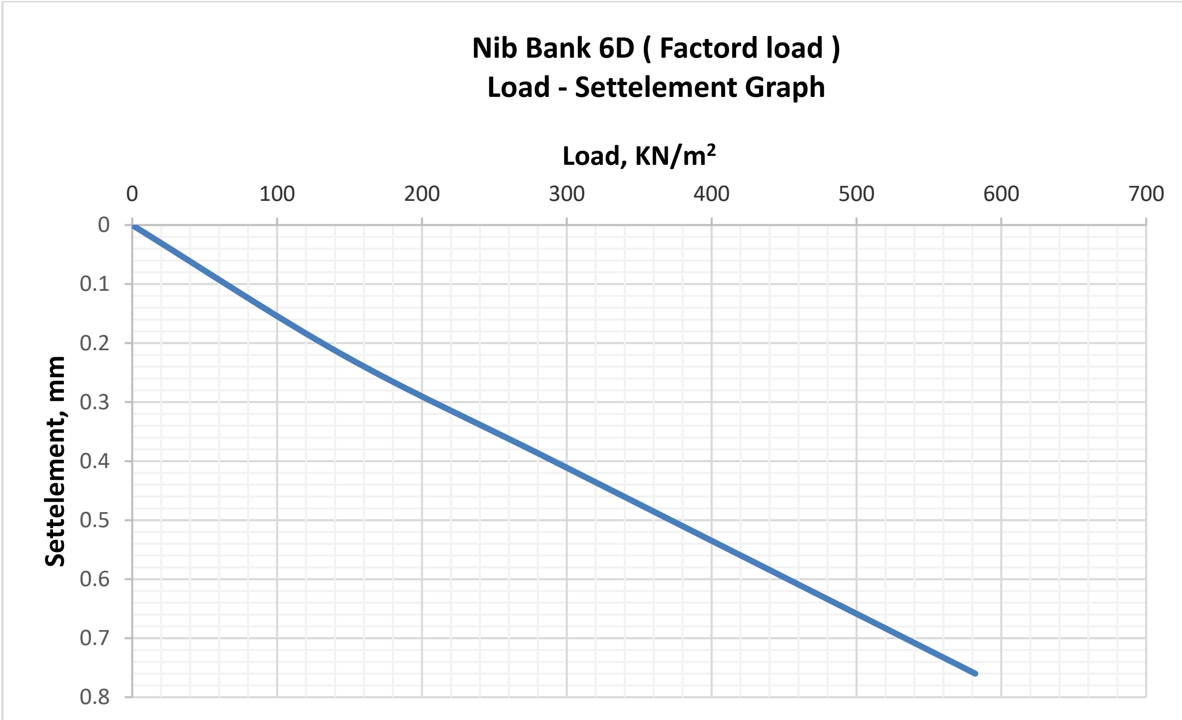



**Fig.5.16.** Nib Bank 6D Load settlement Curve

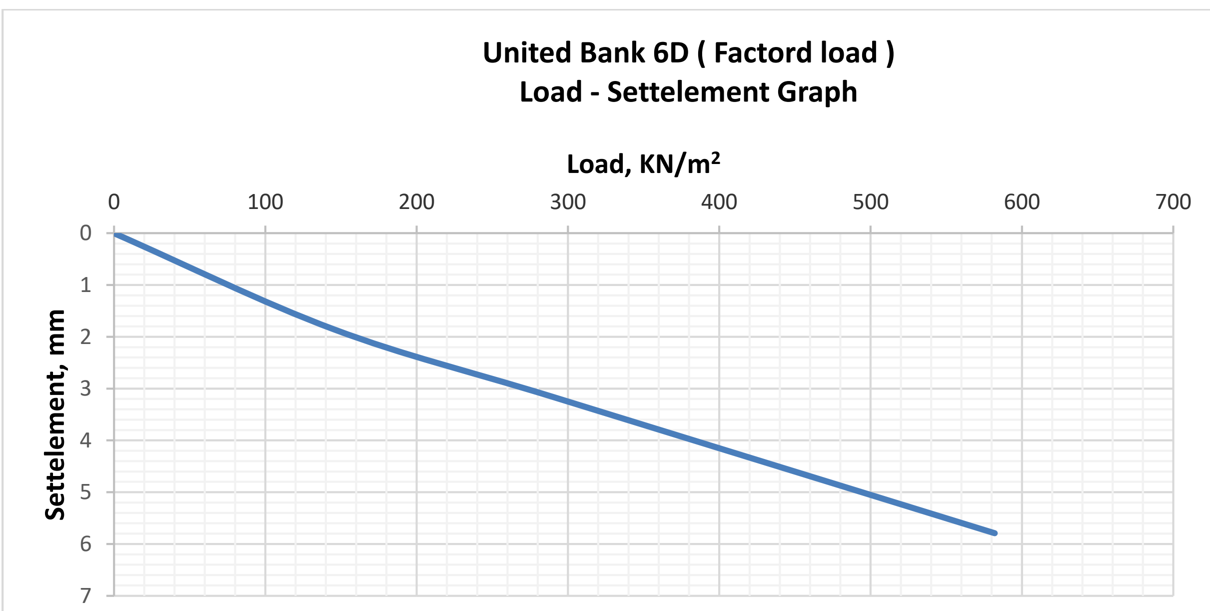



**Fig.5.17.** United Bank 6D Load settlement Curve

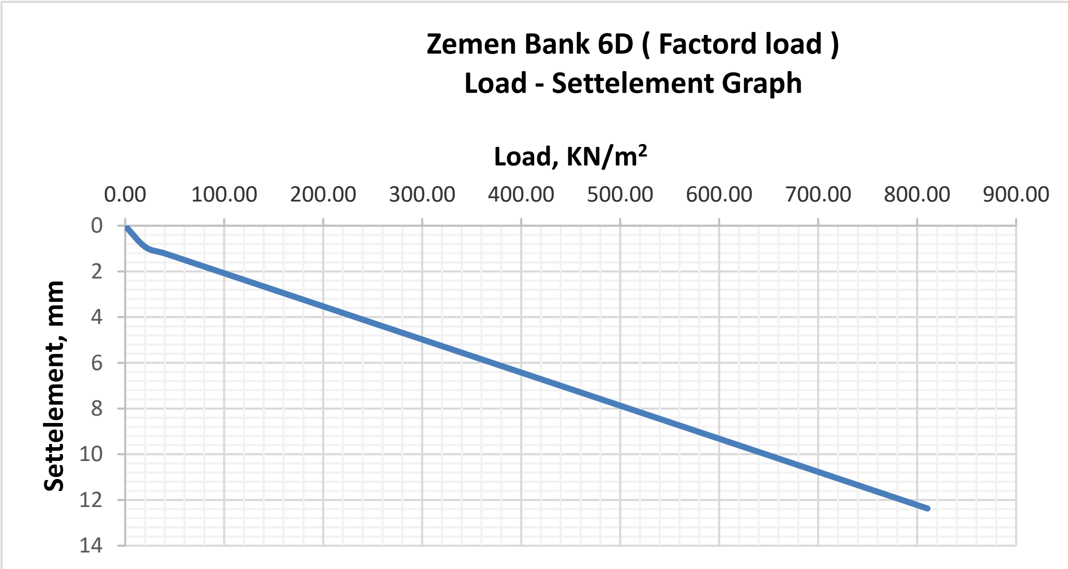



**Fig.5.18**. Zemen Bank 6D Load settlement Curve

### 5.2.4 Friction Mobilized and Load Carried In Friction

Mobilized friction and load carried by friction are summarized in table 5.4.

**Table 5.4** Mobilized friction and load carried by friction

| | | United Bank | | | Nib Bank | | | Zemen Bank | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Top portion | Middle portion | Bottom portion | Top portion | Middle portion | Bottom portion | Top portion | Middle portion | Bottom portion |
| Friction Mobilized (Kpa) | 3D | 70,580.95 | 553.58 | 5,843.30 | 8,930.30 | 3,572.12 | 4,822.36 | 47,822.82 | 31,830.63 | 119,018.86 |
| | 6D | 47,822.82 | 859.89 | 23,834.53 | 52,063.64 | 22,236.44 | 7,983.69 | 53,358.58 | 35,521.13 | 13,009,038.27 |
| Load Carried In Friction (KN) | 3D | 2,836,789.64 | 93,169.07 | 73,391.89 | 201,896.20 | 53,838.99 | 109,023.95 | 961,047.47 | 1,439,253.57 | 1,195,901.51 |
| | 6D | 1,922,094.94 | 144,722.62 | 299,361.65 | 1,177,054.84 | 335,147.69 | 180,495.20 | 1,072,294.12 | 1,606,123.54 | 130,714,816.56 |
| % Increased/ Decreased | | -32% | 55% | 308% | 483% | 523% | 66% | 12% | 12% | 10830% |

Note the thickness of top, middle, and bottom portions of the piles are shown in Table 5.5 to 5.7.

Table 5.4 shows that mobilized friction and load carried by friction are increased as the pile spacing changed from 3D to 6D, except united bank on top portion the pile. The increment for the top, middle, and bottom portion are 12%-480%, 12%-523%, and 66%-10,8030% respectively. The percent increment depends on the length of the thickness of the soil.

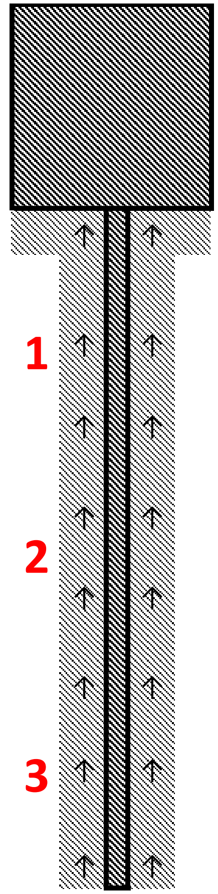
**Table 5.5** Mobilized friction at different loading

| Smmary of skin friction at different loading stage | | | | | | |
|---|---|---|---|---|---|---|
| | NIB BANK | | UNITED BANK | | ZEMEN BANK | |
| | NIB 3D | NIB 6D | UNITED 3D | UNITED 6D | ZEMEN 3D | ZEMEN 6D |
| 1st load Increment | (6,241.20) | (1,513.38) | (4,731.69) | (10,343.85) | (9,792.65) | (4,921.41) |
| 2nd load Incremer | (12,022.13) | (2,729.09) | (10,731.69) | (21,274.46) | (17,826.20) | (9,241.73) |
| 3rd load Incremen | (20,700.07) | (8,416.39) | (14,731.69) | (43,176.93) | (29,354.99) | (15,556.17) |
| Last load Incremer | (22,687.85) | (11,672.81) | (24,731.69) | (51,551.60) | (43,033.01) | (39,976.76) |

Results of mobilized friction at different loading are shown in table 5.5. The result indicates that the foundation system is not typically piling foundation under vertical load. If the frictional force starts declining, it implied it became end bearing foundation.

**Table 5.5 Friction Mobilized** and **Load Carried In Friction Q Friction Zemen Bank, 3D and 6D**

| Layer | Material Property | | Friction Mobilized, Fi (Kpa) | Load Carried In Friction Q Friction I (KN) |
|---|---|---|---|---|
| | $E_s$ | 30,000,000 | | |
| | v | 0.25 | | |
| L=1, (15-24m) | L | 24.00 | 8,930.30 | 201,896.20 |
| | D | 0.80 | | |
| | $S_{friction}$ | 1.00E-04 | | |
| | $E_s$ | 30,000,000 | | |
| | v | 0.25 | | |
| L=2,(24-30m) | L | 24.00 | 3,572.12 | 53,838.99 |
| | D | 0.80 | | |
| | $S_{friction}$ | 4.00E-05 | | |
| | $E_s$ | 18,000,000 | | |
| | Vs | 0.25 | | |
| L=3,(30-39m) | L | 24.00 | 4,822.36 | 109,023.95 |
| | D | 0.80 | | |
| | $S_{friction}$ | 9.00E-05 | | |




| Layer | Material Property | | Friction Mobilized, Fi (Kpa) | Load Carried In Friction Q Friction (KN) |
|---|---|---|---|---|
| | $E_s$ | 30,000,000 | | |
| | v | 0.25 | | |
| 1 | L | 24.00 | 52,063.64 | 1,177,054.84 |
| | D | 0.80 | | |
| | $S_{friction}$ | 5.83E-04 | | |
| | $E_s$ | 30,000,000 | | |
| | v | 0.25 | | |
| 2 | | 24.00 | 22,236.44 | 335,147.69 |
| | | 0.80 | | |
| | $S_{friction}$ | 2.49E-04 | | |
| | $E_s$ | 18,000,000 | | |
| | Vs | 0.25 | | |
| 3 | L | 24.00 | 7,983.69 | 180,495.20 |
| | D | 0.80 | | |
| | $S_{friction}$ | 1.49E-04 | | |

**Table 5.6 Friction Mobilized** and **Load Carried In Friction Q Friction UNITED Bank, 3D and 6D**

| UNITED Bank, 3D | | | | | UNITED Bank, 6D | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Layer | Material Property | | Friction Mobilized, Fi (Kpa) | Load Carried In Friction Q Friction I (KN) | Layer | Material Property | | Friction Mobilized, Fi (Kpa) | Load Carried In Friction Q Friction I (KN) |
| | $E_s$ | 5,000,000 | | | | $E_s$ | 5,000,000 | | |
| | v | 0.25 | | | | v | 0.25 | | |
| L=1,(15-20m) | L | 28.00 | 70,580.95 | 2,836,789.64 | 1 | L | 28.00 | 47,822.82 | 1,922,094.94 |
| | D | 0.80 | | | | D | 0.80 | | |
| | $S_{friction}$ | 4.59E-03 | | | | $S_{friction}$ | 3.11E-03 | | |
| | $E_s$ | 120,000 | | | | $E_s$ | 120,000 | | |
| | v | 0.25 | | | | v | 0.25 | | |
| L=2,(20-27m) | L | 28.00 | 553.58 | 93,169.07 | 2 | L | 28.00 | 859.89 | 144,722.62 |
| | D | 0.80 | | | | D | 0.80 | | |
| | $S_{friction}$ | 1.50E-03 | | | | $S_{friction}$ | 2.33E-03 | | |
| | $E_s$ | 5,000,000 | | | | $E_s$ | 5,000,000 | | |
| | Vs | 0.25 | | | | Vs | 0.25 | | |
| L=3,(27-42m) | L | 28.00 | 5,843.30 | 73,391.89 | 3 | L | 28.00 | 23,834.53 | 299,361.65 |
| | D | 0.80 | | | | D | 0.80 | | |
| | $S_{friction}$ | 3.80E-04 | | | | $S_{friction}$ | 1.55E-03 | | |

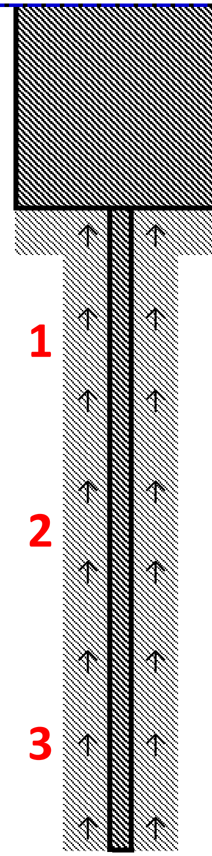
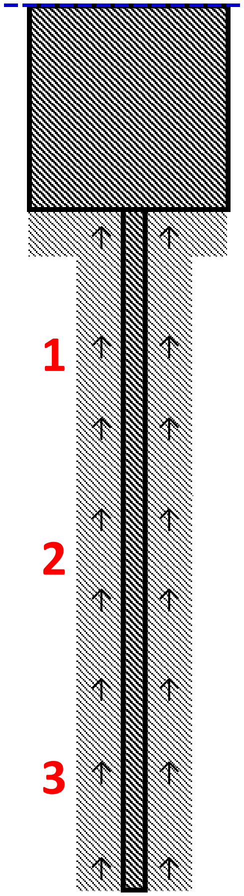


**Table 5.7 Friction Mobilized** and **Load Carried In Friction Q Friction NIB Bank,3D and 6D**




| NIB Bank, 3D | | | | | NIB Bank, 6D | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Layer | Material Property | | Friction Mobilized, Fi (Kpa) | Load Carried In Friction Q Friction I (KN) | Layer | Material Property | | Friction Mobilized, Fi (Kpa) | Load Carried In Friction Q Friction (KN) |
| L=1, (15-24m) | $E_s$ | 30,000,000 | 8,930.30 | 201,896.20 | 1 | $E_s$ | 30,000,000 | 52,063.64 | 1,177,054.84 |
| | v | 0.25 | | | | v | 0.25 | | |
| | L | 24.00 | | | | L | 24.00 | | |
| | D | 0.80 | | | | D | 0.80 | | |
| | $S_{friction}$ | 1.00E-04 | | | | $S_{friction}$ | 5.83E-04 | | |
| L=2,(24-30m) | $E_s$ | 30,000,000 | 3,572.12 | 53,838.99 | 2 | $E_s$ | 30,000,000 | 22,236.44 | 335,147.69 |
| | v | 0.25 | | | | v | 0.25 | | |
| | L | 24.00 | | | | | 24.00 | | |
| | D | 0.80 | | | | | 0.80 | | |
| | $S_{friction}$ | 4.00E-05 | | | | $S_{friction}$ | 2.49E-04 | | |
| L=3,(30-39m) | $E_s$ | 18,000,000 | 4,822.36 | 109,023.95 | 3 | $E_s$ | 18,000,000 | 7,983.69 | 180,495.20 |
| | Vs | 0.25 | | | | Vs | 0.25 | | |
| | L | 24.00 | | | | L | 24.00 | | |
| | D | 0.80 | | | | D | 0.80 | | |
| | $S_{friction}$ | 9.00E-05 | | | | $S_{friction}$ | 1.49E-04 | | |

**Summary**

This study uses FEM software ABAQUS to assess the piled raft foundation under vertical load. This is done using three projects having similar natures. On modeling; the pile length, pile diameter, raft thickness, width & length and all the material property were constant. But the spacing of the pile was variable. Two types of the load (factored and unfactored loads in the form of uniformly distributed load) applied separately on each model. (i.e. 3D, 6D and raft alone). The following conclusion is drawn from the 3D FEM analysis.

## 5.2. Conclusion and Recommendation

### 5.2.2 Conclusion

- Piled raft foundation with actually designed spacing (3D) and doubled pile spacing (6D) controlled differential settlement almost equally.
- Angular distortion results are within the tolerable limit in all projects for both pile spacing.
- Doubling pile spacing doesn't have significant importance on settlement reduction and but play a role in load sharing under vertical load.
- The load settlement graph shows that the piled raft has not reached to the state of failure under vertical load. There is reserved bearing capacity.
- Under the vertically applied load, all the projects are safe.
- Generally, for similar projects maximum settlement, differential settlement, and angular distortion of "piled raft foundation" under vertical load is within a tolerable limit. Moreover, the maximum load is taken by the piles. Load on the raft is almost zero.

### 5.2.3 Recommendation

- This study is conducted on the bases of statical load only. Earth quack load and/ or wind action shall be studied in the future separately. The other study directions may focus on varying pile and raft parameters together or separately for optimization.

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# Appendix A

## Soil Investigation Data and Data Used in the Study

**Table A.1.**Water Table Measurement Result

| Investigation | Zemen Bank | United Bank | Nib Bank |
|---|---|---|---|
| Ground Water measurement ( m ) | 7.60 to 9.60 | 6 to 9.9 | 1 to 8 |

**Table A.2.**Natural Unit Weight and Effective Unit Weight

i) Unit weight (γ) of United Bank

| **Geological Layer Description (only foundation bearing layers of united bank )** | **Depth of Occurrence, m** | **Mass density(kg/m3) γ** | **γ Mass density(effective) (kg/m3)** |
|---|---|---|---|
| Layer 1:-Dense, grey to reddish brown, with some pinkish and whitish spot, slightly GRAVEL, derived from highly weather vascular rock | 15 - 20 | 22.99 | 1343.53 |
| Layer 2:-Hard reddish-brown silty clay (paleo soil) | 20-27 | 20.28 | 1067.28 |
| Layer 3:- Dense to very dense, brownish to grayish, sandy gravel derived from highly weathered Basalt | 27-40 | 18.75 | 911.31 |
| Layer 4:- Medium - strong, grey, slightly weathered fractured aphanitic BASALT | 40-45 | 20.84 | 1124.36 |

ii) Unit weight (γ) of Zemen Bank

| **Geological Layer Description (only foundation bearing layers of Zemen Bank)** | **Depth of Occurrence, (m)** | **Mass density(kg/m3) γ** | **Mass density(effective) (kg/m3)** |
|---|---|---|---|
| **Layer 1**:- Stiff to very stiff, brown to yellowish gray to gray, Sandy SILT with gravel ( Decomposed Rock) | 16 - 22 | 28 | 1854.23 |
| **Layer 2**:- stiff to very stiff, brownish to yellowish gray to gray sand Clayey SILT with gravel ( Decomposed Rock) | 24 - 40 | 28 | 1854.23 |
| **Layer 3**:- Fine grained, gray to dark gray, slightly weathered and moderately fractured BASALT | 42 -44 | 28 | 1854.23 |
| **Layer 4** -stiff to very stiff, brownish to yellowish gray to gray sandy clayey SILT ( decomposed rock) | 45 - 50 | 28 | 1854.23 |

iii) Unit weight (γ)Nib Bank

| Geological Layer Description (only foundation bearing layers of Nib bank ) | Depth of Occurrence, m | Mass density(kg/m3) γ | Mass density(effective) (kg/m3) |
|---|---|---|---|
| Layer 7; Highly fractured rock<br><br>Layer 8 : Greyish black moderately strong - **strong basalt** with sub-vertical and sub-horizontal joints | 15- 24 | 17.10 | 1762.49 |
| Layer 9: Grayish black, highly fractured, moderately strong to week basalt and reddish moderately to highly decomposed weak to very weak basalt<br>Layer 10; Greyish black, fractured, **strong rock**, with cavities and highly weathered, slightly to modernly decomposed weak to strong rick with K- feldspars | 24 -30 | 27.1 | 1762.49 |
| Layer 10;Reddish,highly fractured ,slightly weathered ,**strong basalt** with sub vertical and sub horizontal fractures | 30 - 40 | 27.1 | 1762.49 |

As per the data collected by Rechard E. Goodman (1989), the dry density of a basaltic rock is 27.1 KN/m3

# Appendix B

## Load Determination

## **Table B.1**. Unfactored Load

| Zemen Bank | | | | | | United Bank | | | | | | Nib Bank | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Unfactored super Structure Load | | | Unfactored Sub Structure Load | | | Unfactored super Structure Load | | | Unfactored Sub Structure Load | | | Unfactored super Structure Load | | | Unfactored Sub Structure Load | | |
| a) Structural Load | | | a) Structural Load | | | a) Structural Load | | | a) Structural Load | | | a) Structural Load | | | a) Structural Load | | |
| Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 |
| Column | 1.65 | KN/m2 | Column | 1.05 | KN/m2 | Column | 1.13 | KN/m2 | Column | 1.13 | KN/m2 | Column | 1.44 | KN/m2 | Column | 1.24 | KN/m2 |
| Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 |
| Stair Case+Shear Wall | 1.93 | KN/m2 | Stair Case+Shear Wall | 1.93 | KN/m2 | Stair Case+Shear Wall | 3.03 | KN/m2 | Stair Case+Shear Wall | 3.03 | KN/m2 | Stair Case+Shear Wall | 2.90 | KN/m2 | Stair Case+Shear Wall | 2.90 | KN/m2 |
| | | | Shear Wall (Sub Structure) | 3.46 | | | | | Shear Wall (Sub Structure) | 1.46 | | | | | Shear Wall (Sub Structure) | 1.22 | |
| b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | |
| marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 |
| cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 |
| ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 |
| Partion Wall ( Al+HCB CW) | 1.3 | KN/m2 | | | KN/m2 | Partion Wall ( Al+HCB) | 1.3 | KN/m2 | | | KN/m2 | Partion Wall ( Al+HCB) | 1.3 | KN/m2 | | | KN/m2 |
| C. Transient loads | | | C. Transient loads | | | C. Transient loads | | | C. Transient loads | | | C. Transient loads | | | C. Transient loads | | |
| Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 |
| Live loads | 3 | KN/m2 | | | KN/m2 | Live loads | 3 | KN/m2 | | | KN/m2 | Live loads | 3 | KN/m2 | | | KN/m2 |
| **Total Unfactored Load (1st -6th)** | **18.35** | **KN/m2** | **Total Unfactored Load (B-G)** | **19.91** | **KN/m2** | **Total Unfactored Load (1st -6th)** | **18.93** | **KN/m2** | **Total Unfactored Load (B-G)** | **19.09** | **KN/m2** | **Total Unfactored Load (1st -6th)** | **19.11** | **KN/m2** | **Total Unfactored Load (B-G)** | **18.83** | **KN/m2** |
| **Total UnfactoredLoad (7th-31)** | **16.35** | **KN/m2** | | | **KN/m2** | **Total Unfactored Load (7th-31)** | **16.93** | **KN/m2** | | | **KN/m2** | **Total Unfactored Load (7th-31)** | **17.11** | **KN/m2** | | | **KN/m2** |

## Table B. 2. Unfactored Load Summary

| Project | No of Substructure | No of Superstructure | Substructure Load (KN/m$^2$) | Superstructure Load (KN/m$^2$)1-6th | Superstructure Load (KN/m$^2$) 7th-31+roof | Total Load applied on the mat (KN/m$^2$) | Remark |
|---|---|---|---|---|---|---|---|
| Zemen Bank | 4 | 31 | 59.73 | 110.12 | 425.19 | 595.04 | 90%of the load is used in the model (535.53kn/m2) |
| United Bank | 4 | 31 | 57.28 | 113.59 | 440.21 | 611.07 | 90%of the load is used in the model (549.39kn/m2) |
| Nib Bank | 4 | 13 | 56.48 | 114.63 | 444.73 | 615.85 | 90%of the load is used in the model (554.26 KN/m2) |

## **Table B. 3.**Factored Load

| Zemen Bank | | | | | | United Bank | | | | | | Nib Bank | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Factored super Structure Load | | | Factored Sub Structure Load | | | Factored super Structure Load | | | Factored Sub Structure Load | | | Factored super Structure Load | | | Factored Sub Structure Load | | |
| a) Structural Load | | | a) Structural Load | | | a) Structural Load | | | a) Structural Load | | | a) Structural Load | | | a) Structural Load | | |
| Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 | Slab | 6.25 | KN/m2 |
| Column | 1.65 | KN/m2 | Column | 1.05 | KN/m2 | Column | 1.13 | KN/m2 | Column | 1.13 | KN/m2 | Column | 1.44 | KN/m2 | Column | 1.24 | KN/m2 |
| Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 | Stair | 0.03 | KN/m2 |
| Stair Case+Shear Wall | 1.93 | KN/m2 | Stair Case+Shear Wall | 1.93 | KN/m2 | Stair Case+Shear Wall | 3.03 | KN/m2 | Stair Case+Shear Wall | 3.03 | KN/m2 | Stair Case+Shear Wall | 2.90 | KN/m2 | Stair Case+Shear Wall | 2.90 | KN/m2 |
| | | | Shear Wall (Sub Structure) | 3.46 | | | | | Shear Wall (Sub Structure) | 1.46 | | | | | Shear Wall (Sub Structure) | 1.22 | |
| b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | | b. Imposed Loads | | |
| marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 | marble finish | 0.81 | KN/m2 |
| cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 | cement screed | 0.92 | KN/m2 |
| ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 | ceiling finish | 0.46 | KN/m2 |
| Partion Wall ( Al+HCB CW) | 1.3 | KN/m2 | | | KN/m2 | Partion Wall ( Al+HCB) | 1.3 | KN/m2 | | | KN/m2 | Partion Wall ( Al+HCB) | 1.3 | KN/m2 | | | KN/m2 |
| C. Transient loads | | | C. Transient loads | | | C. Transient loads | | | C. Transient loads | | | C. Transient loads | | | C. Transient loads | | |
| Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 | Live loads | 5 | KN/m2 |
| Live loads | 3 | KN/m2 | | | KN/m2 | Live loads | 3 | KN/m2 | | | KN/m2 | Live loads | 3 | KN/m2 | | | KN/m2 |
| **Total Factored Load (1st -6th)** | **25.36** | **KN/m2** | **Total Factored Load (B-G)** | **27.38** | **KN/m2** | **Total Factored Load (1st -6th)** | **26.11** | **KN/m2** | **Total Factored Load (B-G)** | **26.32** | **KN/m2** | **Total Factored Load (1st -6th)** | **26.34** | **KN/m2** | **Total Factored Load (B-G)** | **25.98** | **KN/m2** |
| **Total Factored Load (7th-31)** | **22.16** | **KN/m2** | | | **KN/m2** | **Total Factored Load (7th-31)** | **22.91** | **KN/m2** | | | **KN/m2** | **Total Factored Load (7th-31)** | **23.14** | **KN/m2** | | | **KN/m2** |

## **Table B. 4.**Factored Load Summary

| Project | No of Substructure | No of Superstructure | Substructure Load ($KN/m^2$) | Superstructure Load ($KN/m^2$)1-6th | Superstructure Load ($KN/m^2$) 7th-31+roof | Total Load applied on the mat ($KN/m^2$) | Remark |
|---|---|---|---|---|---|---|---|
| Zemen Bank | 4 | 31 | 82.15 | 152.16 | 576.14 | 810.45 | 90%of the load is used in the model (729.40kn/m2) |
| United Bank | 4 | 31 | 78.96 | 156.66 | 595.67 | 831.29 | 90%of the load is used in the model (748.17kn/m2) |
| Nib Bank | 4 | 13 | 77.93 | 158.02 | 601.55 | 837.50 | 90%of the load is used in the model (753.75 KN/m2) |

# Appendix C
## Drill log Diagrams

**Table C.1.a.** Zemen Bank Project BH-1

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

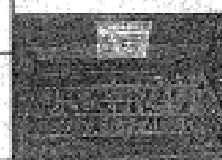
## Drilling Log

| | |
|---|---|
| Drilling Log | Sheet: 1 of Sheet:4 |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472386 Northing: 0996250 | Type of Rig: Wire line rottary rig SH4500 |
| Ground water Elevation(m): 16.60 | BH-ID:BH-01 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 16-02-2013 / Date Completed : 24-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

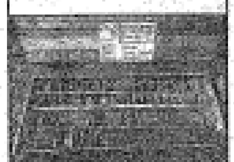
| core run (m) | Hole dia. (mm) | Core Recovery TCR% | Core Recovery RQD% | Core Recovery AFS(cm) | Sampling | SPT No. Blows |
|---|---|---|---|---|---|---|
| 0.00 | 110 | 100 | | | | |
| 1.10 | | 100 | | | | |
| 1.50 | | 100 | | | | |
| 2.00 | | 100 | 0 | HF | | |
| 2.50 | | 100 | | | | |
| 2.90 | | 100 | | | | |
| 3.55 | | 100 | | | | |
| 4.00 | | 100 | | | | |
| 4.20 | | 100 | | | | |
| 5.00 | | 100 | | | | |
| 5.40 | | 100 | | | | |
| 6.00 | | 100 | | | | |
| 6.70 | 101 | 100 | 0 | | | |
| 7.00 | | 100 | | | | |
| 8.25 | | 100 | | | | |
| 9.75 | | 100 | | | | |
| 10.50 | | 100 | | | 10.50–10.75 | |
| 11.55 | | 100 | | | | |
| 12.00 | | 100 | | | | |
| 13.00 | | 100 | | | | |
| 13.80 | | 100 | | | | |
| 14.40 | | 100 | | | | |
| 15.00 | | | | | | |

| Depth (m) | Field Description |
|---|---|
| 0.60 | Fill material |
| 1.50 | Medium stiff,brownish,gravelly clayey SILT |
| 3.60 | Strong,dark gray, fine grained,highly weatherd and moderately to highly fractured BASALT with some fine materials. |
| 8.50 | Dense to very dense, brownish gray, gravelly silty SAND with some Highly weatherd core stone BASALT (Highly weatherd vesicular basalt) |
| 15.00 | Stiff to very stiff, red to reddish brown Highly plastic, clayey SILT. |

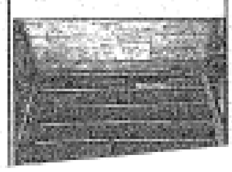
Core photography



0.00-5.00m



5.00-9.85m



Contractor:
Subcontractor:
Consultant:JDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Guteme Megersa

soil Sample
Rock Sample
Undisturbed Sample

Bit
Bit

**Table C.1.b** Zemen Bank ProjectBH-1

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

Sheet: 2 of Sheet:4

| | |
|---|---|
| Project : 48+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472386 Northing: 0996250 | Type of Rig: Wire line rottary rig SH1000 |
| Ground water Elevation(m): 16.60 | BH-ID:BH-01 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 16-02-2013 / Date Completed : 24-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

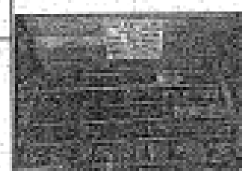
Hole dia. (mm): 101

| core run (m) | Core Recovery TCR% | RQD% | AFS(cm) |
|---|---|---|---|
| 15.20 | 100 | | |
| 16.70 | 100 | | |
| 17.60 | 100 | | |
| 19.60 | 100 | | |
| 20.40 | 100 | | |
| 21.00 | 100 | | |
| 21.60 | 100 | | |
| 22.30 | 100 | | |
| 23.40 | 100 | | |
| 24.50 | 100 | | |
| 25.00 | 100 | | |
| 25.65 | 100 | | |
| 26.45 | 100 | | |
| 27.25 | 100 | | |
| 27.70 | 100 | | |
| 28.65 | 100 | | |
| 29.00 | 100 | | |
| 29.55 | 100 | | |
| 30.00 | 100 | | |

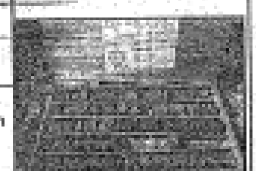
| Sampling | SPT No. Blows |
|---|---|
| 15.20 – 15.45 (Undisturbed) | 15.45 – 15.90: 8/15/20 |
| | 17.60 – 18.05: 6/8/10 |
| 19.40 – 20.10 (Disturbed) | 19.60 – 19.85: 22/R |
| | 21.60 – 21.67: R |
| 23.40 – 23.68 (Undisturbed) | 23.68 – 24.13: 7/16/20 |
| | 25.65 – 25.94: 35/R |
| 27.60 – 28.40 (Disturbed) | 27.60 – 28.14: 3/39/R |
| | 29.55 – 30.00: 28/32/33 |

| Depth (m) | Field Description |
|---|---|
| 15.20 – 16.00 | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. |
| 16.00 – 22.30 | stiff to very stiff, gray to yellowish gray,gravelly clay SILT with some core stone Highly weatherd Basalt (Decomposed rock) |
| 22.30 – 24.50 | Very stiff to hard,red to reddish brown Highly plastic, clayey SILT |
| 24.50 – 30.00 | Very stiff to Hard, brownish gray sandy clayey SILT with some gravel embeded to the clay matrix.The gravels are very weak ,weatherd , moist and vesicular basalt. |

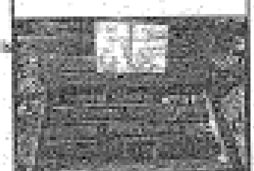
Core photography



15.00-19.85m



19.85-25.00m



25.00-30.00m

Contractor:
Subcontractor:
Consultant:IDAW Consulting Archibects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diametr: 110,101

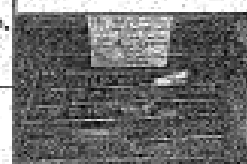
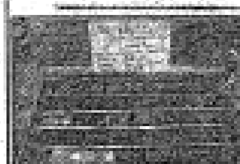
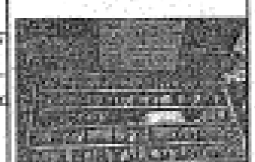
**Table C.1.c** Zemen Bank ProjectBH-1

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

| | |
|---|---|
| Sheet: 3 of Sheet:4 | |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 047286 Northing: 0986250 | Type of Rig: Wire line rottary rig SH1000 |
| Ground water Elevation(m): 16.60 | BH-ID:BH-01 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 16-02-2013 / Date Completed : 24-02-2013 | SPT: Authomatic Sampling: Split spoonand shelve Tube |

| core run (m) | Hole dia. (mm) | TCR% | RQD% | AFS(cm) | Sampling | SPT No. Blows | Graphic LOG | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 30.70–31.40 | 101 | 100 | | | 31.20–32.00 | 31.40 8/17/23 31.85 | | Very stiff to hard,red to reddish brown, Highly plastic,clayey SILT. 32.40 |  29.00-33.80m |
| 31.40–32.40 | | 100 | | | | | | | |
| 32.40–33.40 | | 100 | | | 32.20–32.40 | 33.40 R | | Very stiff to hard,brownish gray gravelly sandy SILT with occasional cobbles of weatherd basalt. | |
| 33.40–35.80 | | 100 | | | | | | | |
| 35.80–37.00 | | 100 | | | | 35.80 14/R 36.07 | | |  33.80-39.00m |
| 37.00–38.00 | | 100 | | | | | | | |
| 38.00–39.00 | | 100 | | | | 38.00 10/26/44 38.45 | | | |
| 39.00–40.00 | | 100 | | | | | | 40.20 | |
| 40.00–41.40 | | 100 | | | | 41.40 R 41.82 | | Moderately strong to strong,fine grained dark gray,slightly weatherd and Highly fractured BASALT. |  39.00-45.00m |
| 41.40–42.00 | | 70 | | | | | | Very stiff to Hard ,brownish gray,highly plastic,sandy clayey SILT | |
| 42.00–43.00 | | 80 | 0 | HF | | | | Moderately strong to strong,fine grained dark gray,slightly weatherd and Highly fractured BASALT. | |
| 43.00–43.70 | | 40 | 0 | HF | | | | 43.70 | |
| 43.70–45.00 | | 100 | | | | | | Very stiff ,red to reddish brown, Highly plastic,clayey SILT 45.00 | |

| | | |
|---|---|---|
| Contractor:<br>Subcontractor:<br>Consultant:IDAW Consulting Architects and Engineers<br>Logged By: Getachew Teresa<br>Approved By:Gutama Megersa | Disturbed soil Sample<br>Rock Sample<br>Undisturbed Sample | Bit Type: Diamond Bit<br>Bit diameter: 118,101 |

**Table C.1.d** Zemen Bank ProjectBH-1

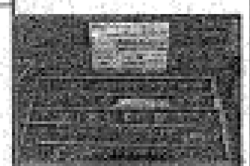
# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

Sheet: 4
of Sheet:4

| | |
|---|---|
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472386 Northing: 0996250 | Type of Rig: Wire line rottary rig SH1000 |
| Ground water Elevation(m): 16.60 | BH-ID:BH-01 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started.: 12-02-2013 / Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Split spoonand shelve Tube |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | AFS(cm) | Sampling | SPT No. Blows | Graphic LOG | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 45.50 | 101 | 100 | | | 45.00 – 45.95 | 45.50<br>8/14/48<br>45.95 | | Very stiff ,red to reddish brown Highly plastic,clayey SILT<br>45.50 | |
| 46.70 | | 100 | | | | | | very stiff to hard,brownish gray, gravelly sandy SILT with occasional slightly weatherd core stone basalt |  |
| 47.00 | | 100 | | | | | | | |
| 47.80 | | 100 | | | | 47.80<br>R<br>47.90 | | | |
| 48.40 | | 100 | | | | | | | |
| 50.00 | | 100 | | | | | | 50.00 | 44.00-50.00 |
| | | | | | END OF BORE HOLE | | | | |

Contractor:
Subcontracter:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Gebachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diameter: 110,101

**Table C.1.e** Zemen Bank ProjectBH-2

# BEST Consulting Engineers

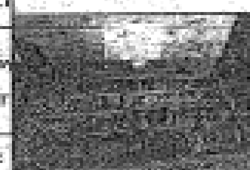
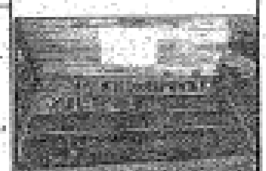
Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

Sheet: 1
of Sheet:4

| | |
|---|---|
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472392 Northing: 0996266 | Type of Rig: Wire line rotary rig SH4500 |
| Ground water Elevation(m): 10.20 | BH-ID:BH-02 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 12-02-2013 / Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

| core run (m) | Hole dia (mm) | Core Recovery TCR% | RQD% | APS(cm) | Sampling | SPT No. Blows | Graphic LOG |
|---|---|---|---|---|---|---|---|
| 0.50 | | 100 | | | | | |
| 1.00 | | 100 | 0 | HF | | | |
| 1.50 | | 100 | | | | | |
| 2.00 | | 100 | | | | | |
| 2.50 | | 100 | | | | | |
| 3.10 | | 100 | | | | | |
| 3.50 | | 100 | | | | | |
| 4.00 | | 100 | | | | | |
| 4.50 | | 100 | | | | | |
| 5.00 | | 100 | | | | | |
| 5.50 | | 100 | | | | | |
| 6.00 | | 100 | | | | | |
| 7.00 | 110 | 100 | | | | | |
| 8.00 | | 100 | | | | | |
| 8.50 | | 100 | | | | | |
| 9.00 | | 100 | | | | | |
| 9.60 | | 100 | | | | | |
| 10.00 | | 100 | | | 10.00 | | |
| 11.00 | | 100 | | | 10.30 | | |
| 12.00 | | 100 | | | | | |
| 13.00 | | 100 | | | | | |
| 13.50 | | 100 | | | | | |
| 14.00 | | 100 | | | | | |
| 14.70 | | 100 | | | | | |

| Field Description | Core photography |
|---|---|
| 0.30 Fill material | |
| Medium stiff,brown,gravelly clayey SILT 1.00 | |
| Fine grained,dark gray to brownish gray moderately to highly weatherd and fragmented to highly fractured BASALT 3.10 |  |
| Dense to very dense,brownish to dark gray,gravelly silty SAND interlayerd with highly weatherd to decomposed and fragmented BASALT. 5.50 | 0.00-4.00m |
| Stiff to very stiff, red to reddish brown, Highly plastic,clayey SILT. 11.00 |  4.00-8.00m |
| Stiff to very stiff, yellowish to gray ,sandy clayey SILT with gravel (Decomposed rock) 14.70 |  |
| Stiff to very stiff, red , clayey SILT | 8.00-12.00m |

Contractor:
Subcontracter:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Guteme Megersa

soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diametr: 110.89

**Table C.1.f** Zemen Bank ProjectBH-2

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

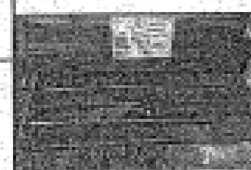
## Drilling Log

Sheet: 2 of Sheet:4

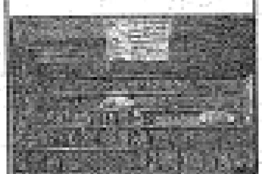
| | |
|---|---|
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472392 Northing: 0996266 | Type of Rig: Wire line rottary rig SH4500 |
| Ground water Elevation(m): 10.20 | BH-ID:BH-02 |
| CasingDepth(m): / Hole Elevation(m):440.36 | Total Depth drilled(m):50.00m |
| Date Started : 12-02-2013 / Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | Core Recovery RQD% | Core Recovery AFS(cm) | Sampling | SPT No. Blows | Field Description |
|---|---|---|---|---|---|---|---|
| | | 100 | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. 16.00 |
| 16.00 | | 100 | | | | | |
| | | 100 | | | | | |
| 17.00 | | 100 | 0 | HF | 17.00 – 17.35 | | Stiff to very stiff, gray to brownish gray gravelly clay SILT mixed with gravel to cobble size of highly weatherd BASALT(Decomposed rock) 27.00 |
| | | 100 | | | | | |
| | | 100 | | | | 18.00 6/8/9 18.45 | |
| 18.00 | | 100 | | | | | |
| 19.00 | | 100 | | | | | |
| 19.50 | | 100 | | | 19.50 – 19.80 | | |
| 20.00 | | 100 | | | | | |
| 20.50 | | 100 | | | | 20.50 5/7/13 20.95 | |
| | | 100 | | | | | |
| 22.00 | 101 | 100 | | | | 22.00 10/15/21 22.45 | |
| 23.00 | | 100 | | | | | |
| 23.50 | | 100 | | | | 23.50 13/29/R 23.90 | |
| 24.00 | | 100 | | | | | |
| 24.50 | | 100 | | | 24.70 – 25.60 | | |
| 25.00 | | 100 | | | | | |
| 25.50 | | 100 | | | | 25.50 R 25.62 | |
| 26.00 | | 100 | | | | | |
| 27.00 | | 100 | | | | | |
| 27.50 | | 100 | 0 | HF | | | Fine grained,yellowish gray, moderately to highly weatherd and highly fractured BASALT 29.50 |
| 28.00 | | 100 | 0 | HF | | | |
| 29.00 | | 100 | 0 | HF | | | |
| 30.00 | | 100 | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. |

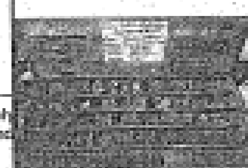
Core photography



16.00-21.00m



21.00-26.00m



26.00-31.00m

Contractor:
Subcontracter:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megerse

Disturbed soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diameter110 ,101,89

**Table C.1.g** Zemen Bank ProjectBH-2

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

| | | | |
|---|---|---|---|
| Drilling Log | | | Sheet: 2 of Sheet:4 |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | | Drilling Method: Rotary core drilling | |
| Location:Around A.A Commercial College<br>Easting: 0472392 Northing: 0996266 | | Type of Rig: Wire line rottary rig SH4500 | |
| Ground water Elevation(m): 10.20 | | BH-ID:BH-02 | |
| CasingDepth(m): | Hole Elevation(m):440.36 | Total Depth drilled(m):50.00m | |
| Date Started : 12-02-2013 | Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Split spoonand shelve Tub | |

| Depth | core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | AFR(cm) | Sampling | SPT No. Blows | Field Description |
|---|---|---|---|---|---|---|---|---|
| | | | 100 | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. |
| 16 | 16.00 | | 100 | | | | | |
| | | | 100 | | | | | 16.00 |
| 17 | 17.00 | | 100 | 0 | HF | 17.00 – 17.35 | | Stiff to very stiff, gray to brownish gray gravelly clay SILT mixed with gravel to cobble size of highly weatherd BASALT(Decomposed rock) |
| | | | 100 | | | | | |
| | | | 100 | | | | 18.00 | |
| 18 | 18.00 | | 100 | | | | 6/8/9 | |
| | | | | | | | 18.45 | |
| 19 | 19.00 | | 100 | | | | | |
| | 19.50 | | 100 | | | 19.50 – 19.80 | | |
| 20 | 20.00 | | 100 | | | | 20.50 | |
| | 20.50 | | 100 | | | | 5/7/13 | |
| 21 | | | 100 | | | | 20.95 | |
| | | 101 | 100 | | | | 22.00 | |
| 22 | 22.00 | | 100 | | | | 10/15/21 | |
| | | | | | | | 22.45 | |
| 23 | 23.00 | | 100 | | | | 23.50 | |
| | 23.50 | | 100 | | | | 3/29/R | |
| 24 | 24.00 | | 100 | | | | 23.90 | |
| | 24.50 | | 100 | | | 24.70 – 25.60 | | |
| 25 | 25.00 | | 100 | | | | 25.50 | |
| | 25.50 | | 100 | | | | R | |
| 26 | 26.00 | | 100 | | | | 25.62 | |
| 27 | 27.00 | | 100 | 0 | HF | | | 27.00 Fine grained,yellowish gray, moderately to highly weatherd and highly fractured BASALT |
| | 27.50 | | 100 | 0 | HF | | | |
| 28 | 28.00 | | 100 | 0 | HF | | | |
| 29 | 29.00 | | 100 | | | | | 29.50 Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. |
| 30 | 30.00 | | | | | | | |

Core photography

16.00-21.00m

21.00-26.00m

26.00-31.00m

| | | |
|---|---|---|
| Contractor:<br>Subcontractor:<br>Consultant:IDAW Consulting Architects and Engineers<br>Logged By: Getachew Teresa<br>Approved By:Gutema Megersa | Disturbed soil Sample<br>Rock Sample<br>Undisturbed Sample | Bit Type: Diamond Bit<br>Bit diameter 110 ,101,89 |

**Table C.1.h** Zemen Bank ProjectBH-2

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

Sheet: 2 of Sheet:4

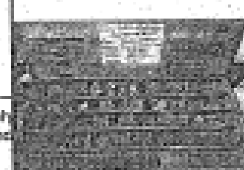
| | |
|---|---|
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472392 Northing: 0996266 | Type of Rig: Wire line rottary rig SH4500 |
| Ground water Elevation(m): 10.20 | BH-ID:BH-02 |
| CasingDepth(m): / Hole Elevation(m):440.36 | Total Depth drilled(m):50.00m |
| Date Started : 12-02-2013 / Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

| Depth | core run (m) | Hole dia. (mm) | TCR% | RQD% | AFR(cm) | Sampling | SPT No. Blows | Field Description |
|---|---|---|---|---|---|---|---|---|
| | | | 100 | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. |
| 16 | 16.00 | | 100 | | | | | |
| | | | 100 | | | | | 16.80 |
| 17 | 17.00 | | 100 | 0 | HF | 17.00 | | |
| | | | 100 | | | 17.35 | | |
| | | | 100 | | | | 18.00 | |
| 18 | 18.00 | | 100 | | | | 6/8/9 | |
| | | | 100 | | | | 18.45 | |
| 19 | 19.00 | | 100 | | | | | |
| | 19.50 | | 100 | | | 19.50 | | |
| 20 | 20.00 | | 100 | | | 19.80 | 20.50 | Stiff to very stiff, gray to brownish gray gravelly clay SILT mixed with gravel to cobble size of highly weatherd BASALT(Decomposed rock) |
| | 20.50 | | 100 | | | | 5/7/11 | |
| 21 | | | 100 | | | | 20.95 | |
| 22 | 22.00 | 101 | 100 | | | | 22.00 | |
| | | | | | | | 10/15/21 | |
| | | | 100 | | | | 22.45 | |
| 23 | 23.00 | | 100 | | | | 23.50 | |
| | 23.50 | | 100 | | | | 19/29/R | |
| 24 | 24.00 | | 100 | | | | 23.90 | |
| | 24.50 | | 100 | | | 24.70 | | |
| 25 | 25.00 | | 100 | | | 25.60 | 25.50 | |
| | 25.50 | | 100 | | | | R | |
| 26 | 26.00 | | | | | | 25.62 | |
| | | | 100 | | | | | 27.00 |
| 27 | 27.00 | | 100 | 0 | HF | | | Fine grained,yellowish gray, moderately to highly weatherd and highly fractured BASALT |
| | 27.50 | | 100 | 0 | HF | | | |
| 28 | 28.00 | | 100 | 0 | HF | | | |
| 29 | 29.00 | | | | | | | 29.50 |
| 30 | 30.00 | | 100 | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. |

Core photography

16.00-21.00m

21.00-26.00m



26.00-31.00m

Contractor:
Subcontracter:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diameter110 ,101,89

**Table C.1.i** Zemen Bank ProjectBH-2

# BEST Consulting Engineers

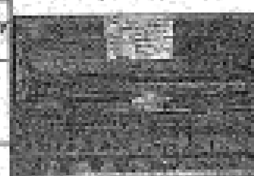
Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

| | |
|---|---|
| | Sheet: 3 of Sheet:4 |
| Project : 4B+G+30 Zemen Bank Head quarter Building | Drilling Method: Rotary core drilling |
| Client:Zemen Bank S.C | |
| Location:Around A.A Commercial College<br>Easting: 0472392 Northing: 0996266 | Type of Rig: Wire line rottary rig SH4500 |
| Ground water Elevation(m): 10.20 | BH-ID:BH-02 |
| CasingDepth(m): / Hole Elevation(m):440.36 | Total Depth drilled(m):50.00m |
| Date Started : 12-02-2013 / Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | APS(cm) | Sampling | SPT No. Blows | Field Description |
|---|---|---|---|---|---|---|---|
| 30.50 | 86 | 100 | | | 30.50 – 30.70 | | Very stiff to hard,red to reddish brown, Highly plastic,clayey SILT. 31.20 |
| 31.00 | | 100 | | | | | |
| 32.20 | | 100 | | | | | Medium stiff,whitish gray,low plastic, clay SILT(It is contact zone b/n red clay and residual soil of vesicular basalt) 32.20 |
| 33.00 | | 100 | | | 32.20 – 33.20 | 32.20 10/17/22 32.65 | Very stiff to hard,brownish gray, gravelly sandy SILT with occasional cobbles of weatherd basalt 33.80 |
| 33.40 | | 100 | | | | | |
| 33.80 | | 100 | | | | | |
| 34.00 | | 100 | 0 | HF | | | |
| 35.00 | 89 | 50 | 30 | 5 | | | Strong to very strong,fine grained dark gray,fresh to slightly weatherd and locally discolerd and moderately fractured BASALT. 37.70 |
| 36.00 | | 65 | 45 | 6 | | | |
| 36.50 | | 55 | 0 | HF | | | |
| 37.70 | | 60 | 0 | HF | | | |
| 38.50 | 86 | 100 | | | | 38.50 12/20/25 38.95 | Very stiff to hard,brownish gray, gravelly sandy SILT with occasional cobbles of weatherd basalt 41.50 |
| 39.70 | | 100 | | | | | |
| 40.70 | | 100 | | | | | |
| 41.50 | | 100 | | | | | |
| 42.00 | 89 | 70 | 0 | HF | | | Strong to very strong,fine grained dark gray,fresh to slightly weatherd and locally discolerd and moderately fractured BASALT. 44.00 |
| 43.00 | | 80 | 60 | 10 | 42.30 – 42.50 | | |
| 43.50 | | 40 | 0 | HF | | | |
| 44.00 | | 65 | 40 | 7 | | | |
| 45.00 | | 100 | | | | | Very stiff ,red to reddish brown, Highly plastic,clayey SILT 45.00 |

Core photography



31.00-36.00m

36.00-41.00m

41.00-46.00m

Contractor:
Subcontractor:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Getachew Tenisa
Approved By:Gutama Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample

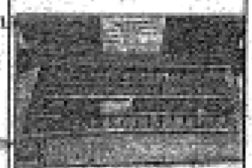
Bit Type: Diamond Bit
Bit diametr: 110,89,86

**Table C.1.j** Zemen Bank ProjectBH-2

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

| | | |
|---|---|---|
| **Drilling Log** | | Sheet: 4 of Sheet:4 |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling | |
| Location:Around A.A Commercial College<br>Easting: 0472392 Northing: 0996266 | Type of Rig: Wire line rottary rig SH4500 | |
| Ground water Elevation(m): 10.20 | BH-ID:BH-02 | |
| CasingDepth(m): / Hole Elevation(m):440.36 | Total Depth drilled(m):50.00m | |
| Date Started : 12-02-2013 / Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube | |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | APS(cm) | Sampling | SPT No. Blows | Graphic LOG | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 45.50 | | 100 | | | | 45.50<br>30/15/21<br>45.95 | | | |
| 46.40 | | 100 | | | | | | Dense,brownish gray,gravelly sandy Silt with occasional cobbles of weatherd Basalt. |  |
| 47.00 | 86 | 100 | | | | | | | |
| 47.50 | | 100 | | | 47.70 | | | | |
| 48.20 | | 100 | | | 48.60 | 48.20<br>17/R<br>48.47 | | 49.00 | |
| 49.00 | | 100 | | | | | | Strong,fine grained,brownish gray,highly weatherd and moderabely to highly fractured,vesicular BASALT. The fracures are clay filled. | |
| 49.50 | | 100 | 40 | 8 | | | | | |
| 50.00 | | 100 | 0 | 3 | | | | | 46.00-50.00 |
| | | | | | END OF BORE HOLE | | | | |

Contractor:
Subcontractor:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diamter: 110,89,86

**Table C.1.k** Zemen Bank ProjectBH-3

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

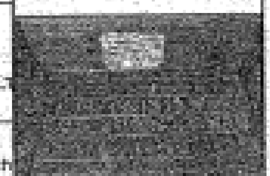
## Drilling Log

Sheet: 1 of Sheet:4

| | |
|---|---|
| Project : 4B+G+30 Zemen Bank Head quarter Building | Drilling Method: Rotary core drilling |
| Client: Zemen Bank S.C | |
| Location:Around A.A Commercial College<br>Easting: 0472408 Northing: 0996252 | Type of Rig: Wire line rottary rig SH4500 |
| Ground water Elevation(m): 6.80 | BH-ID:BH-03 |
| CasingDepth(m):6.00 | Hole Elevation(m):2362 |
| Total Depth drilled(m):50.00m | |
| Date Started : 02-02-2013 | Date Completed : 12-02-2013 |
| SPT: Authomatic Sampling: Splitt spoonand shelve Tube | |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | APS(cm) | Sampling | SPT No. Blows |
|---|---|---|---|---|---|---|
| 0.50 | 110 | 100 | | | | |
| 1.00 | | 100 | | | | |
| 1.60 | | 100 | | | | |
| 2.00 | | 100 | | | | |
| 2.30 | | 100 | | | | |
| 2.60 | | 100 | 0 | HF | | |
| 3.00 | | 100 | | | | |
| 3.45 | | 100 | - | | | |
| 4.00 | | 100 | | | | |
| 4.50 | | 100 | | | | |
| 5.00 | | 100 | | | | |
| 5.50 | | 100 | | | | |
| 6.00 | | 100 | 0 | HF | | |
| 6.40 | | 100 | | | | |
| 6.70 | | 100 | 50 | 10 | | |
| 7.20 | | 100 | 40 | 10 | | |
| 7.60 | | 100 | 0 | 4 | | |
| 8.00 | | 100 | 30 | 3 | | |
| 9.20 | 89 | 80 | 35 | 3 | | |
| 10.00 | | 75 | 25 | 4 | | |
| 11.00 | | 80 | 45 | 6 | | |
| 12.00 | | 100 | 90 | 13 | | |
| 13.00 | | 50 | 0 | HF | | |
| 13.45 | | 80 | 65 | 15 | | |
| 14.00 | | 100 | | | | |
| 14.55 | | 100 | | | | |
| 15.00 | | | | | | |

| Field Description | Core photography |
|---|---|
| 0.50 Fill material | |
| Medium stiff, gravelly clayey SILT 1.00 | |
| Fine grained,dark gray,highly weathered to decomosed and fragmented to Highly fractured BASALT 3.00 | |
| Dense to very dense,brownish to dark gray,gravelly silty SAND interlayerd with Highly weatherd to decomposed and fragmented BASALT(Highly weathord to decomposed vesicular BASALT) 5.50 | 0.00-4.00m |
| Strong to very strong,fine grained, dark gray, slightly to moderately weatherd and moderately to highly fractured BASALT. Fracturing and weathering intensity increases down to the depth. Fractures are clay filled. 13.45 | 4.00-8.00m |
| Stiff to very stiff, red to ... Highly plastic,clayey S... 15.00 | |



Contractor:
Subcontracter:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megersa

soil Sample
Rock Sample
Undisturbed Sampl...

**Table C.1.m** Zemen Bank ProjectBH-3

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Abeba, Ethiopia

## Drilling Log

| | |
|---|---|
| Drilling Log | Sheet: 2 of Sheet:4 |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472408 Northing: 0996252 | Type of Rig: Wire line rottary rig SH4500 |
| Ground water Elevation(m): 6.80 | BH-ID:BH-03 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 02-02-2013 / Date Completed : 12-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | AFS(cm) | Sampling | SPT No. Blows | Graphic Log | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 15.60 | | 100 | | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. 16.50 | |
| 16.00 | | 100 | | | | | | |  |
| 17.00 | 86 | 100 | | | 16.90 – 17.60 | 17.00 5/8/9 17.45 | | Stiff to very stiff, gray to yellowish brown ,sandy clayey SILT with gravel to cobble size of weatherd rock (Decomposed rock) 19.00 | |
| 18.00 | | 100 | | | | | | | 14.00-19.00m |
| 19.00 | | 100 | | | | | | | |
| 19.50 | | 75 | 0 | HF | | | | | |
| 20.00 | | 50 | 0 | HF | | | | | |
| | | 50 | 0 | HF | | | | | |
| 21.00 | | 65 | 0 | HF | | | | |  |
| 22.00 | 89 | 30 | 0 | HF | | | | Moderately Strong to strong, fine grained,dark gray,moderately to highly weatherd and highly fractured BASALT.The interval b/n 22.50-24.50m is highly sheared and brecciated and the fractures are filled with clay. | 1900-25.00m |
| 23.00 | | 40 | 0 | HF | | | | | |
| 24.00 | | 60 | 0 | HF | | | | | |
| 24.50 | | 65 | 0 | HF | | | | | |
| 25.00 | | 70 | 0 | HF | | | | | |
| 25.50 | | 60 | 0 | HF | | | | | |
| 26.00 | | 65 | 40 | 5 | 26.00 – 26.25 | | | |  |
| 27.00 | | 45 | 0 | HF | | | | | |
| 28.00 | | 40 | 0 | HF | | | | | |
| 29.00 | | 50 | 0 | HF | | | | 29.50 | |
| 29.50 | | | | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. | 25.00-30.00m |
| 30.00 | | | | | | | | | |

Contractor:
Subcontractor:
Consultant:JDAW Consulting Architects and Engineers
Logged By: Getachew Tenssa
Approved By:Gutama Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample
HF : Highly Fractured

Bit Type: Diamond Bit
Bit diametr: 110,89,86

**Table C.1.o** Zemen Bank ProjectBH-3

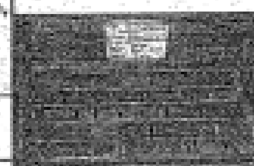
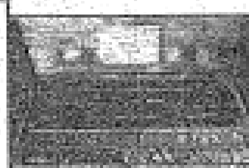
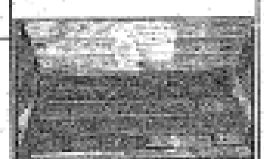
# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

Sheet: 3
of Sheet:4

| | |
|---|---|
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C. | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472408 Northing: 0996252 | Type of Rig: Wire line rotary rig SH4500 |
| Ground water Elevation(m): 6.80 | BH-ID:BH-03 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 02-02-2013 / Date Completed : 19-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

| core run (m) | Hole dia. (mm) | TCR% | RQD% | AFS(cm) | Sampling | SPT No. Blows | Graphic LOG | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 31.00 | 86 | 100 | | | 31.00–31.20 | 31.20–31.65 15/14/1? | | Very stiff to hard,red to reddish brown, highly plastic,clayey SILT. 32.50 |  31.00-36.00m |
| 32.50 | | 100 | | | | | | | |
| 33.00 | | 100 | | | | | | Medium stiff,whitish gray,low plastic, sandy clayey SILT with gravel 33.50 | |
| | | 100 | | | 33.80–34.60 | 34.00–34.45 1?/21/27 | | Very stiff to Hard,brownish gray, clayey SILT with sand. 35.50 | |
| 34.00 | | 100 | | | | | | | |
| 35.00 | | 100 | | | | | | | |
| 35.50 | | 100 | 0 | HF | | | | Strong to very strong,fine grained dark gray,fresh to slightly weatherd and moderately fractured BASALT. 40.00 |  36.00-42.00m |
| 36.00 | | 100 | 30 | 10 | | | | | |
| 37.00 | | 100 | 70 | 16 | | | | | |
| 38.00 | | 85 | 35 | 10 | | | | | |
| 38.30 | | 100 | 0 | HF | | | | | |
| 38.70 | | 100 | 100 | 15 | | | | | |
| 39.00 | | 100 | 70 | 13 | 39.00–39.30 | | | | |
| 40.00 | | 100 | | | | 41.00–41.14 R | | very stiff to hard,brownish gray, sandy clayey SILT with gravel. 41.50 | |
| 41.00 | | 100 | | | | | | | |
| 41.50 | | 100 | 0 | HF | | | | Strong to very strong,fine grained dark gray,fresh to slightly weatherd and moderately fractured BASALT. 44.00 |  42.00-47.00m |
| 42.00 | | 85 | 85 | 10 | | | | | |
| 43.00 | | 80 | 60 | 12 | | | | | |
| 44.00 | | 100 | | | | | | Very stiff ,red to reddish brown, Highly plastic,clayey SILT 45.00 | |
| 45.00 | | | | | | | | | |

Contractor:
Subcontracter:
Consultant: JDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample

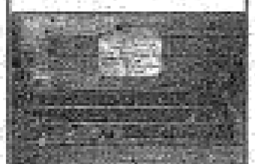
Bit Type: Diamond Bit
Bit diameter: 110,89,86

**Table C.1.p** Zemen Bank ProjectBH-3

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

| | | |
|---|---|---|
| **Drilling Log** | | Sheet: 4<br>of Sheet:4 |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling | |
| Location:Around A.A Commercial College<br>Easting: 0472408 Northing: 0996252 | Type of Rig: Wire line rottary rig SH4500 | |
| Ground water Elevation(m): 6.80 | BH-ID:BH-03 | |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m | |
| Date Started : 02-02-2013 / Date Completed : 12-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube | |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | Core Recovery RQD% | Core Recovery AFS(cm) | Sampling | SPT No. Blows | Graphic LOG | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 46.00 | 86 | 100 | | | 46.40 | 45.00<br>R | | very stiff to Hard,pinkish brown to gray,clayey silty SAND with gravel to cobble size of weatherd basalt. |  |
| 46.50 | | 100 | | | 47.30 | 46.50<br>27/32/36<br>46.95 | | | |
| 48.00 | | 100 | | | | | | | |
| 49.00 | | 100 | | | | 49.00<br>0/48/R<br>49.40 | | | |
| 50.00 | | 100 | | | | | | 50.00 | 45.00-50.00 |
| | | | | | END OF BORE HOLE | | | | |

Contractor:
Subcontracter:
Consultant:JDAW Consulting Architects and Engineers

Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample
HF: Highly fractured

Bit Type: Diamond and carbid Bit
Bit diametr: 110,89,86

Consulting Engineers

**Table C.1.p** Zemen Bank ProjectBH-4

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

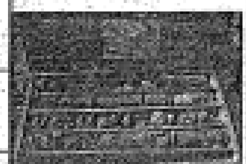
## Drilling Log

| | |
|---|---|
| | Sheet: 1 of Sheet:4 |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472413 Northing: 0996267 | Type of Rig: Wire line rottary rig SH1000 |
| Ground water Elevation(m): 5.00 | BH-ID:BH-0 4 |
| CasingDepth(m):6.00 / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 05-02-2013 / Date Completed : 16-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

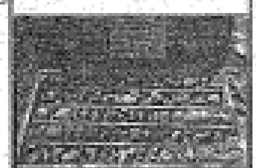
| core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | AFS(cm) | Sampling | SPT No. Blows |
|---|---|---|---|---|---|---|
| 0.35 | 116 | 100 | | | | |
| 1.10 | | 100 | | | | |
| 1.75 | 101 | 100 | | | | |
| 2.00 | | 100 | 0 | HF | | |
| 2.35 | | 100 | 0 | HF | | |
| 2.60 | | 100 | 0 | HF | | |
| 2.85 | | 100 | 0 | HF | | |
| 3.20 | | 100 | 0 | HF | | |
| 3.60 | | 100 | 0 | HF | | |
| 4.00 | | 100 | 0 | HF | | |
| 4.50 | | 100 | 0 | HF | | |
| 5.00 | | 100 | 0 | HF | | |
| 5.35 | | 100 | 0 | HF | | |
| 6.00 | | 100 | 0 | HF | | |
| 6.40 | | 100 | 0 | HF | | |
| 7.00 | | 100 | 0 | HF | | |
| 7.30 | | 100 | | HF | | |
| 8.30 | | 100 | 0 | HF | | |
| 8.50 | | 100 | 0 | HF | | |
| 9.00 | | 100 | 0 | HF | | |
| 10.40 | | 100 | | | | |
| 11.10 | | 100 | | | | |
| 12.15 | | 100 | | | | |
| 13.25 | | 100 | | | | |
| 14.20 | | 100 | | | | |
| 14.65 | | 100 | | | | |
| 15.00 | | | | | | |

| Graphic LOG (depth) | Field Description |
|---|---|
| 0.70 | Fill material |
| 1.75 | Medium stiff,dark brown gravelly silty CLAY |
| 3.20 | Fine grained,dark gray,slightlyto moderately weatherd and highly fractured BASALT. |
| 9.00 | Dense to very dense,brownish to dark gray,gravelly silty SAND interlayerdwith Highly weathered to decomposed and fragmented basalt(Highly weatherd to decomposed vesicular BASALT) |
| 13.50 | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT |
| | very stiff to hard, Gray to yellowish gray, sandy clayey SILT with gravel and corestone basalt (Decomposed rock) |

Core photography



0.00-4.00m



4.00-9.00m

9.00-14.00m

Contractor:
Subcontracter:
Consultant:IDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megersa

soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diameter 110,101,89

**Table C.1.q** Zemen Bank ProjectBH-4

# BEST Consulting Engineers

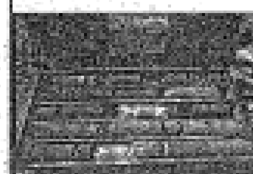
Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

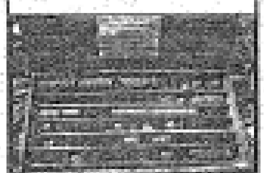
| | |
|---|---|
| Sheet: 2 of Sheet:4 | |
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling |
| Location:Around A.A Commercial College<br>Easting: 0472413 Northing: 0996267 | Type of Rig: Wire line rottary rig SH1000 |
| Ground water Elevation(m): 5.00 | BH-ID:BH-04 |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m |
| Date Started : 05-02-2013 / Date Completed : 16-02-2013 | SPT: Authomatic Sampling: Splitt spoonand shelve Tube |

| core run (m) | Hole dia. (mm) | Core Recovery TCR% | RQD% | APS(cm) | Sampling | SPT No. Blows | Field Description |
|---|---|---|---|---|---|---|---|
| 15.25 | 101 | 100 | | | | | Very stiff to Hard yellowish gray to brown ,sandy clayey SILT with gravel to cobble size of weatherd rock (Decomposed rock) |
| 16.40 | | 100 | | | | | |
| 17.00 | | 100 | | | | 17.00 R | |
| 18.60 | | 100 | | | | | |
| 19.00 | | 100 | | | | | |
| 19.50 | | 100 | | | | 19.50 R | |
| 20.20 | | 100 | | | 20.60–20.90 | | |
| 21.30 | | 100 | | | | 21.30 13/20/? 21.75 | |
| 22.30 | | 100 | | | | | |
| 23.25 | | 100 | | | 23.00–23.70 | 23.25 26/R 23.49 | |
| 24.00 | | 100 | | | | | |
| 24.65 | | 100 | | | | | 24.65 |
| 25.00 | 89 | 85 | 0 | HF | | | strong,fine grained,dark gray, slightly weatherd and moderately to highly fractured BASALT. |
| 25.85 | | 60 | 0 | HF | | | |
| 26.30 | | 95 | 88 | 9 | 26.30–26.60 | | |
| 27.00 | | 85 | 85 | 5 | | | |
| 27.60 | | 60 | 0 | HF | | | |
| 28.00 | | 40 | 0 | HF | | | |
| 28.60 | | 50 | 0 | HF | | | |
| 29.20 | | 40 | 0 | HF | | | 29.20 |
| 30.00 | | | | | | | Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. |

Core photography



19.00-24.00m



24.00-30.00m

30.00-36.00m

Contractor:
Subcontracter:
Consultant:JDAW Consulting Architects and Engineers

Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample
HF : Highly Fractured

Bit Type: Diamond Bit
Bit Diameter:110,101,89

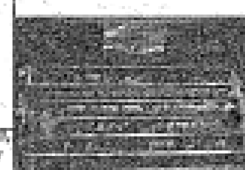
**Table C.1.r** Zemen Bank ProjectBH-4

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

| Drilling Log | | Sheet: 3 of Sheet:4 |
|---|---|---|
| Project : 4B+G+30 Zemen Bank Head quarter Building<br>Client:Zemen Bank S.C | Drilling Method: Rotary core drilling | |
| Location:Around A.A Commercial College<br>Easting: 0472413 Northing: 0996267 | Type of Rig: Wire line rottary rig SH4500 | |
| Ground water Elevation(m): 5.00 | BH-ID:BH-04 | |
| CasingDepth(m): / Hole Elevation(m):2362 | Total Depth drilled(m):50.00m | |
| Date Started : 05-02-2013 / Date Completed : 16-02-2013 | SPT: Authomatic Sampling: Split spoonand shelve Tube | |

| Depth | core run (m) | hole dia. (mm) | TCR% | RQD% | APS(cm) | Sampling | SPT No. Blows | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 31 | 31.00 | 86 | 100 | | | 31.00 (Undisturbed) 31.28 | | Very stiff to hard,red to reddish brown, highly plastic,clayey SILT. |  |
| 32 | 32.00 | | 100 | | | | 32.00 5/11/18 32.45 | 32.50 | |
| 33 | 33.00 | | 100 | | | | | Medium stiff,whitish gray,low plastic, clay SILT(It is contact zone b/n red clay and residual soil of vesicular basalt) 33.30 | |
| 34 | 34.20 | | 100 | | | 34.00 (Disturbed) 34.90 | 34.20 12/42/R 34.55 | | |
| 35 | 35.00 | | 100 | | | | | | 35.00-40.00m |
| | 35.50 | | 100 | | | | | | |
| 36 | 36.00 | | 100 | | | | 36.00 12/23/43 36.45 | Very stiff to hard,brownish gray, clayey silty SAND with gravel to core stone of weatherd basalt | |
| 37 | 37.00 | | 100 | | | | | | |
| | 37.60 | | 100 | | | | | | |
| 38 | 38.00 | | | | | | | | |
| | 38.20 | | 100 | | | | 38.20 | | |
| | 38.70 | | 100 | | | | 12/21/31 | | |
| 39 | 39.00 | | 100 | | | | 38.65 | | |
| | 39.40 | | 100 | | | | | | |
| 40 | 40.00 | | 100 | | | | 40.00 R | 40.20 | |
| | 40.30 | | 100 | | | | | Medium strong to Strong ,fine grained, dark gray,slightly weatherd and highly fractured BASALT. | |
| 41 | 41.00 | 89 | 100 | 0 | HF | | | 41.50 Very stiff to hard,brownish gray, clayey silty SAND with gravel to core stone of weatherd basalt 42.20 | |
| 42 | 42.00 | | 100 | | | | 42.00 R | | 40.00-45.00m |
| | 42.50 | | | | | | | | |
| 43 | 43.00 | | 80 | 0 | HF | | | Medium strong to Strong ,fine grained, dark gray,slightly weatherd and highly fractured BASALT. | |
| 44 | 43.80 | | 100 | 0 | HF | | | | |
| 45 | 44.90 | | 100 | 0 | HF | | | 44.90 | |



Contractor:
Subcontracter:
Consultant:IDAW Consulting Architects and Engineers

Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit Diameter: 110,89,86

**Table C.1.s** Zemen Bank ProjectBH-4

# BEST Consulting Engineers

Tel: +251-011-6630065/011-5159213 Fax: +251-011-6630064/011-5159216
P.O.Box 101472 Addis Ababa, Ethiopia

## Drilling Log

Sheet: 4
of Sheet:4

Project : 4B+G+30 Zemen Bank Head quarter Building
Client: Zemen Bank S.C

Drilling Method: Rotary core drilling

Location:Around A.A Commercial College
Easting: 0472413 Northing: 0996267

Type of Rig: Wire line rotary rig SH4500

Ground water Elevation(m): 5.00

BH-ID:BH-04

CasingDepth(m):

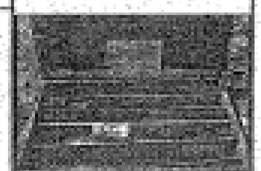
Hole Elevation(m):2362

Total Depth drilled(m):50.00m

Date Started : 12-02-2013

Date Completed : 19-02-2013

SPT: Authomatic Sampling: Splitt spoonand shelve Tube

| core run (m) | Hole dia. (mm) | Core Recovery TC% | RQD% | APB(cm) | Sampling | SPT No. Blows | Graphic LOG | Field Description | Core photography |
|---|---|---|---|---|---|---|---|---|---|
| 46.00 | | 100 | | | | | | Very stiff ,red to reddish brown, Highly plastic,clayey SILT 46.00 | |
| 48.10 | 86 | 100 | | | 46.00 46.90 | 46.10 R 27/32/36 46.55 | | Dense to verydense,pinkish brown to gray,clayey silty SAND with gravel to cobble size of weatherd basalt. |  |
| | | 100 | | | | 48.10 10/21/28 48.55 | | | |
| 50.00 | | 100 | | | | | | 50.00 | 45.00-50.00 |
| | | | | | END OF BORE HOLE | | | | |

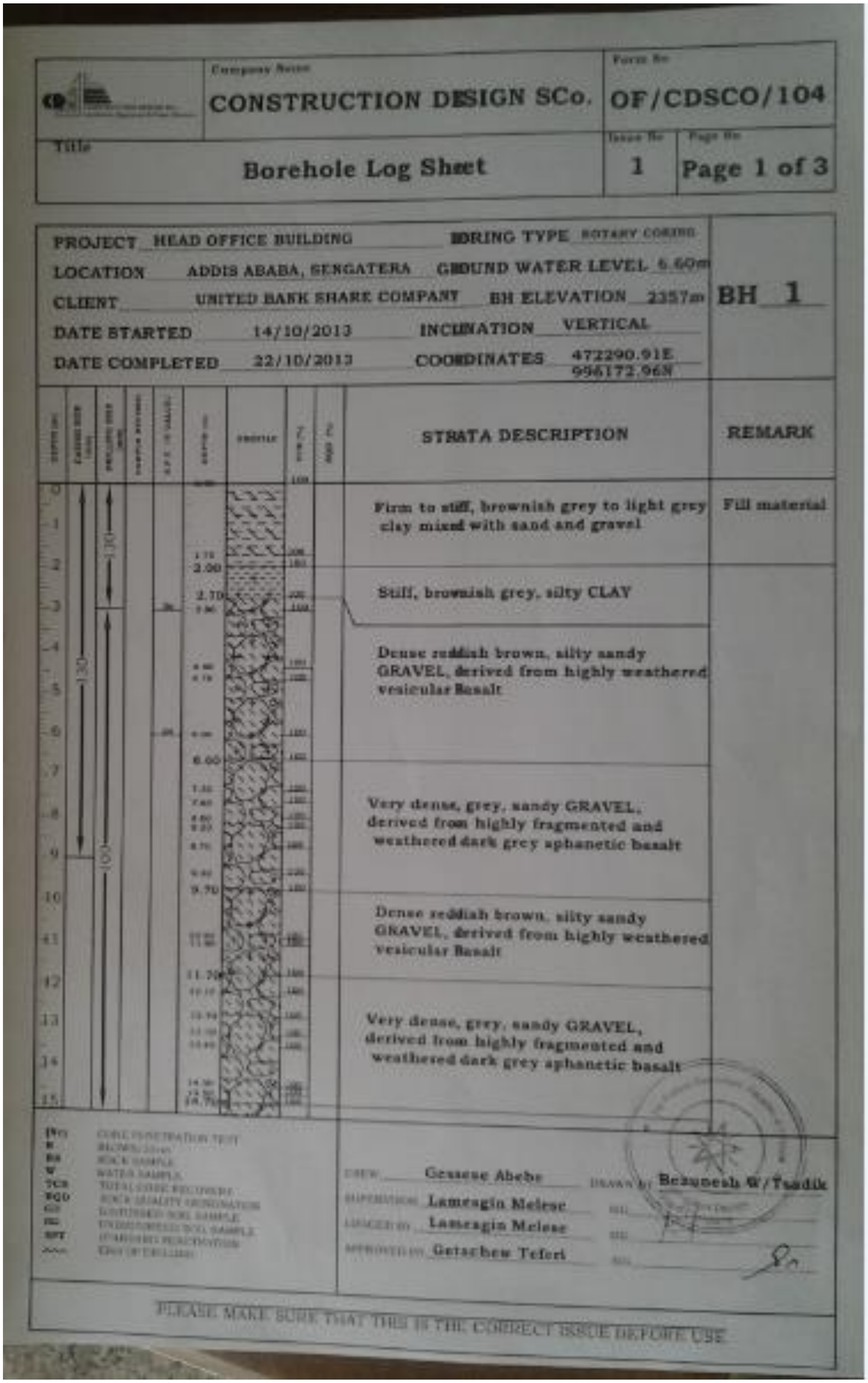
Contractor:
Subcontracter:
Consultant:JDAW Consulting Architects and Engineers

Logged By: Getachew Teresa
Approved By:Gutema Megersa

Disturbed soil Sample
Rock Sample
Undisturbed Sample
HF: Highly fractured

Bit Type: Diamond and carbid Bit
Bit diametr: 110,89,86

**Table C.2.a** United Bank ProjectBH-1BH-4



| Company Name | Form No |
|---|---|
| CONSTRUCTION DESIGN SCo. | OF/CDSCO/104 |

| Title | Issue No | Page No |
|---|---|---|
| Borehole Log Sheet | 1 | Page 1 of 3 |

PROJECT HEAD OFFICE BUILDING — BORING TYPE ROTARY CORING
LOCATION ADDIS ABABA, SENGATERA — GROUND WATER LEVEL 6.60m
CLIENT UNITED BANK SHARE COMPANY — BH ELEVATION 2357m
DATE STARTED 14/10/2013 — INCLINATION VERTICAL
DATE COMPLETED 22/10/2013 — COORDINATES 472290.91E 996172.96N

**BH 1**

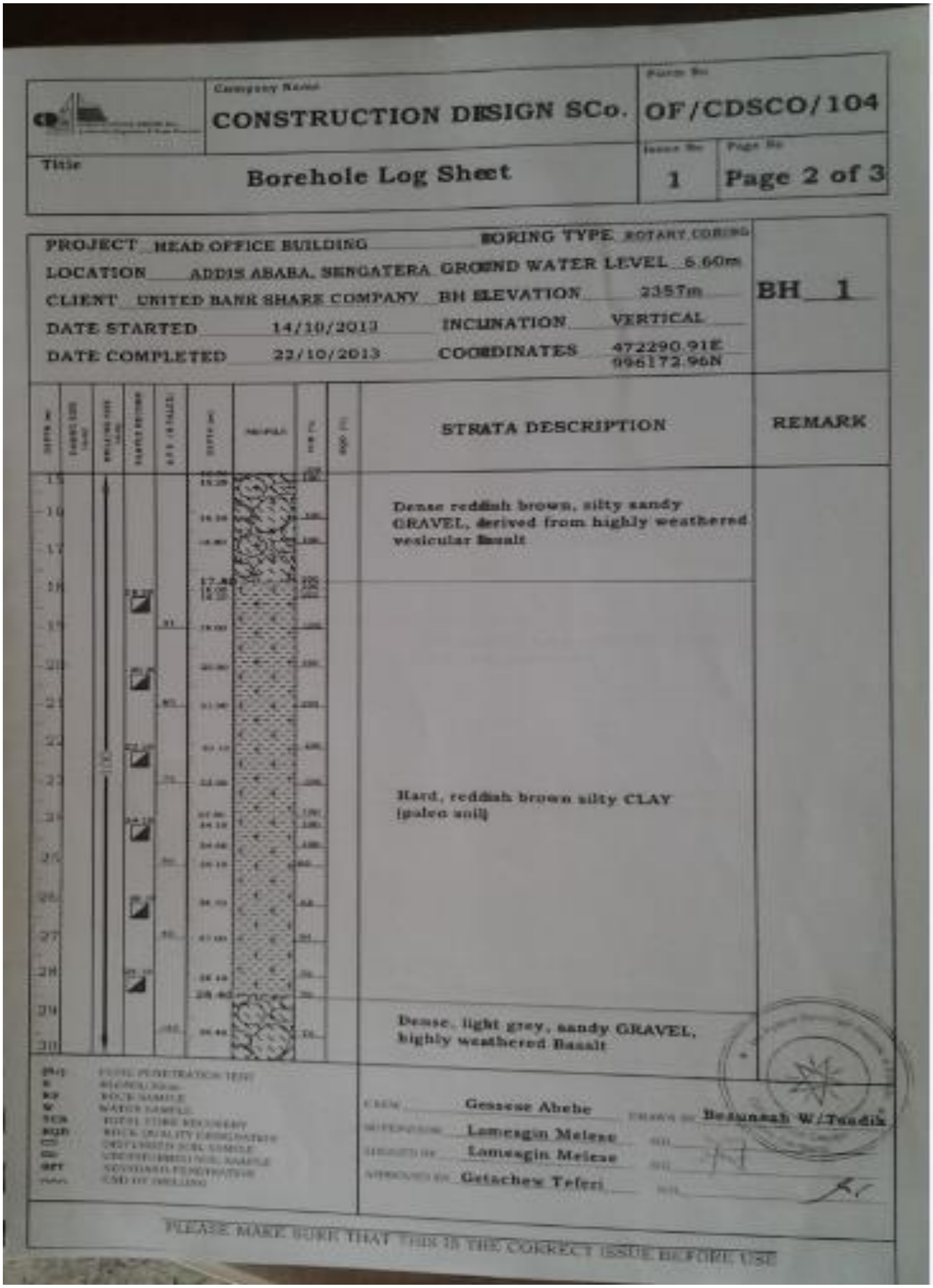
| STRATA DESCRIPTION | REMARK |
|---|---|
| Firm to stiff, brownish grey to light grey clay mixed with sand and gravel | Fill material |
| Stiff, brownish grey, silty CLAY | |
| Dense reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt | |
| Very dense, grey, sandy GRAVEL, derived from highly fragmented and weathered dark grey aphanetic basalt | |
| Dense reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt | |
| Very dense, grey, sandy GRAVEL, derived from highly fragmented and weathered dark grey aphanetic basalt | |

Depths: 2.00, 2.70, 6.00, 9.70, 11.70, 14.70

CHECKED BY Gezahegn Abebe — DRAWN BY Bezunesh W/Tsadik
SUPERVISED BY Lamesgin Melese
LOGGED BY Lamesgin Melese
APPROVED BY Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.b** United Bank ProjectBH-1



Company Name: **CONSTRUCTION DESIGN SCo.**

Form No: **OF/CDSCO/104**

Title: **Borehole Log Sheet**

Issue No: 1 | Page No: **Page 2 of 3**

| | | | |
|---|---|---|---|
| PROJECT | HEAD OFFICE BUILDING | BORING TYPE | ROTARY CORING |
| LOCATION | ADDIS ABABA, SENGATERA | GROUND WATER LEVEL | 6.60m |
| CLIENT | UNITED BANK SHARE COMPANY | BH ELEVATION | 2357m |
| DATE STARTED | 14/10/2013 | INCLINATION | VERTICAL |
| DATE COMPLETED | 22/10/2013 | COORDINATES | 472290.91E 996172.96N |

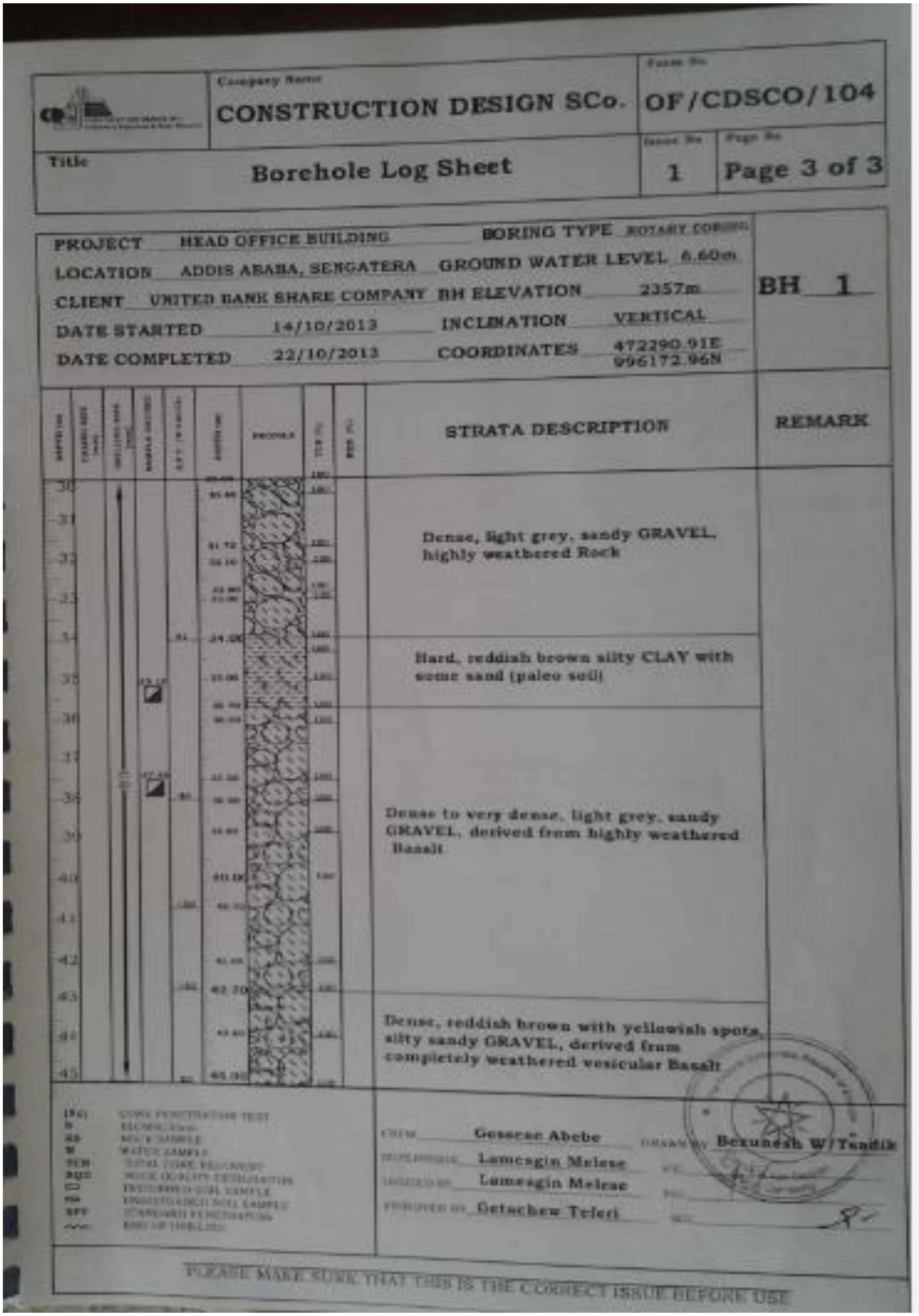
**BH 1**

| STRATA DESCRIPTION | REMARK |
|---|---|
| Dense reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt | |
| Hard, reddish brown silty CLAY (paleo soil) | |
| Dense, light grey, sandy GRAVEL, highly weathered Basalt | |

CREW: Gessese Abebe
DRAWN BY: Besuneah W/Tsadik
SUPERVISOR: Lamesgin Melese
CHECKED BY: Lamesgin Melese
APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.c** United Bank ProjectBH-1



| | | | |
|---|---|---|---|
| Company Name | CONSTRUCTION DESIGN SCo. | Form No | OF/CDSCO/104 |
| Title | Borehole Log Sheet | Issue No 1 | Page No Page 3 of 3 |

PROJECT HEAD OFFICE BUILDING
BORING TYPE ROTARY CORING
LOCATION ADDIS ABABA, SENGATERA
GROUND WATER LEVEL 6.60m
CLIENT UNITED BANK SHARE COMPANY
BH ELEVATION 2357m
DATE STARTED 14/10/2013
INCLINATION VERTICAL
DATE COMPLETED 22/10/2013
COORDINATES 472290.91E 996172.96N

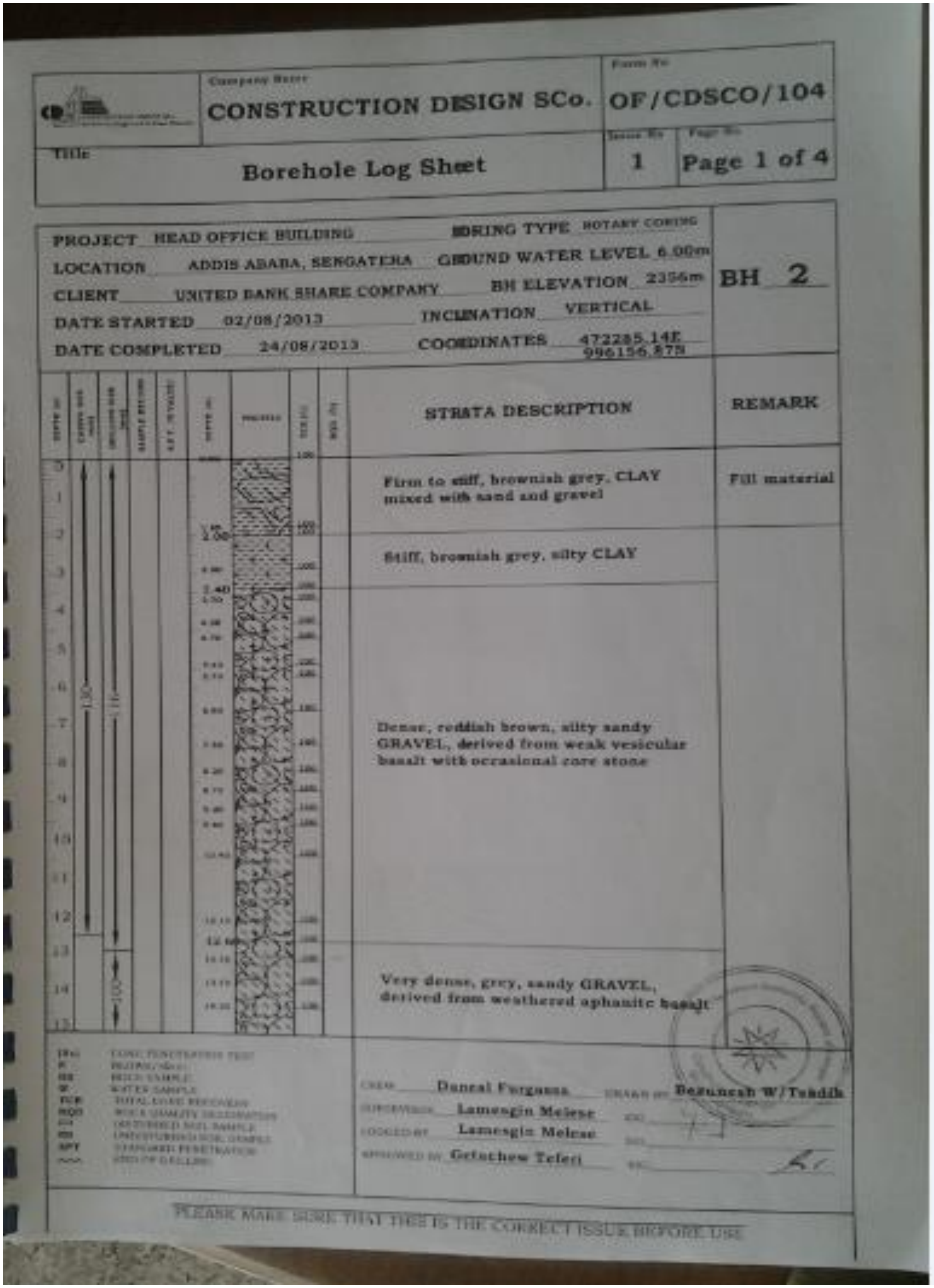
BH 1

| STRATA DESCRIPTION | REMARK |
|---|---|
| Dense, light grey, sandy GRAVEL, highly weathered Rock | |
| Hard, reddish brown silty CLAY with some sand (paleo soil) | |
| Dense to very dense, light grey, sandy GRAVEL, derived from highly weathered Basalt | |
| Dense, reddish brown with yellowish spots, silty sandy GRAVEL, derived from completely weathered vesicular Basalt | |

CREW Gesesse Abebe
DRAWN BY Bezunesh W/Tsadik
Lamesgin Melese
Lamesgin Melese
APPROVED BY Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.d** United Bank ProjectBH-2



| Company Name | Form No |
|---|---|
| CONSTRUCTION DESIGN SCo. | OF/CDSCO/104 |
| Title: Borehole Log Sheet | Issue No 1 — Page No Page 1 of 4 |

PROJECT HEAD OFFICE BUILDING — BORING TYPE ROTARY CORING

LOCATION ADDIS ABABA, SENGATERA — GROUND WATER LEVEL 6.00m

CLIENT UNITED BANK SHARE COMPANY — BH ELEVATION 2356m

DATE STARTED 02/08/2013 — INCLINATION VERTICAL

DATE COMPLETED 24/08/2013 — COORDINATES 472285.14E 996156.87N

BH 2

| STRATA DESCRIPTION | REMARK |
|---|---|
| Firm to stiff, brownish grey, CLAY mixed with sand and gravel | Fill material |
| Stiff, brownish grey, silty CLAY | |
| Dense, reddish brown, silty sandy GRAVEL, derived from weak vesicular basalt with occasional core stone | |
| Very dense, grey, sandy GRAVEL, derived from weathered aphanitic basalt | |

CREW Daneal Furgassa — DRAWN BY Bezunesh W/Tsadik

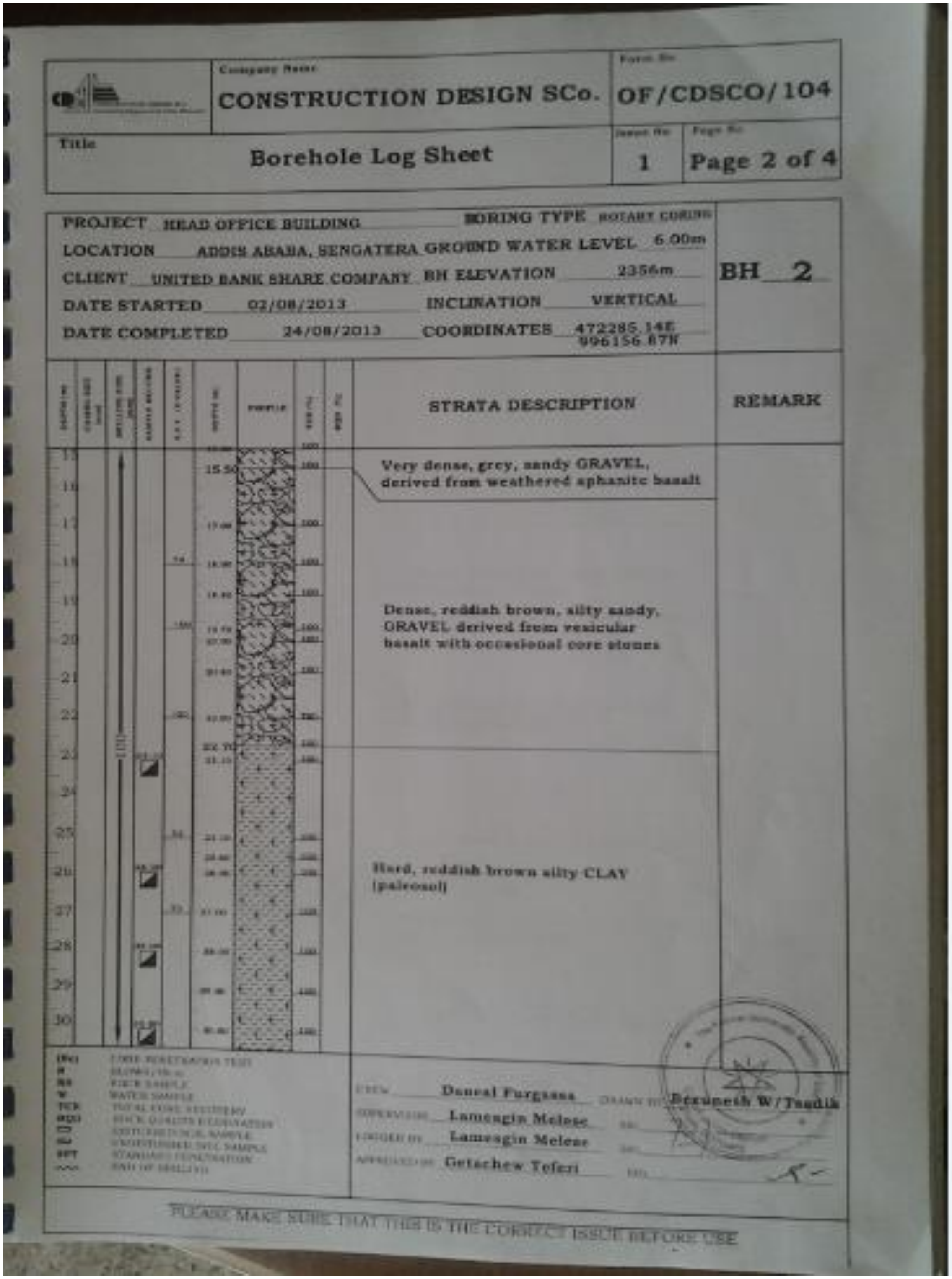
SUPERVISED Lamesgin Melese

LOGGED BY Lamesgin Melese

APPROVED BY Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.e** United Bank ProjectBH-2



CONSTRUCTION DESIGN SCo. | OF/CDSCO/104

Title: Borehole Log Sheet | 1 | Page 2 of 4

| | | | |
|---|---|---|---|
| PROJECT | HEAD OFFICE BUILDING | BORING TYPE | ROTARY CORING |
| LOCATION | ADDIS ABABA, SENGATERA | GROUND WATER LEVEL | 6.00m |
| CLIENT | UNITED BANK SHARE COMPANY | BH ELEVATION | 2356m |
| DATE STARTED | 02/08/2013 | INCLINATION | VERTICAL |
| DATE COMPLETED | 24/08/2013 | COORDINATES | 472285.14E 996156.87N |

BH 2

| Depth | Strata Description | Remark |
|---|---|---|
| 15.50 | Very dense, grey, sandy GRAVEL, derived from weathered aphanitc basalt | |
| | Dense, reddish brown, silty sandy, GRAVEL derived from vesicular basalt with occasional core stones | |
| | Hard, reddish brown silty CLAY (paleosol) | |

Dawal Furgassa

Drawn by: Bezuneh W/Tsadik

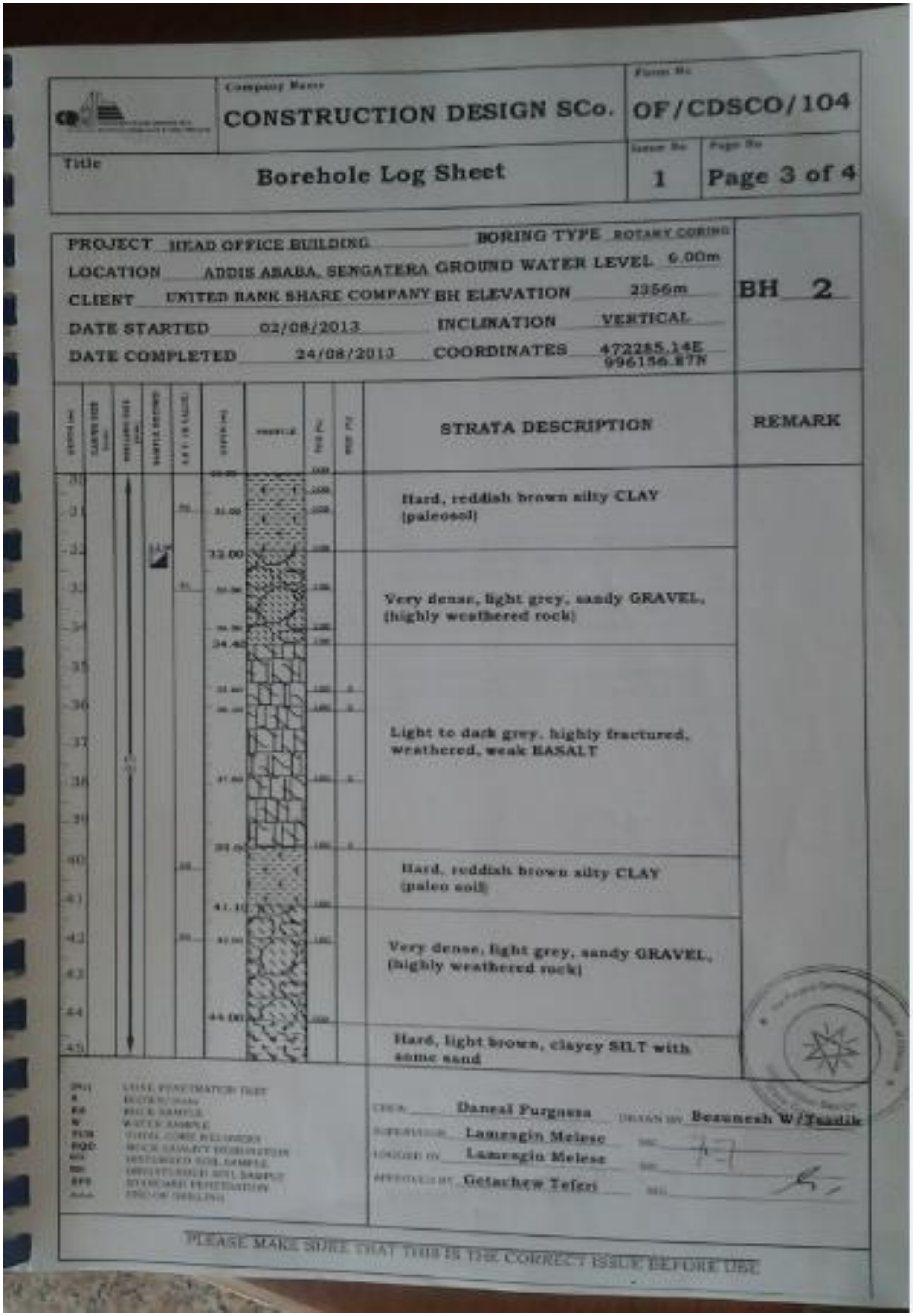
Supervisor: Lamesgin Melese

Logged by: Lamesgin Melese

Approved by: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.f** United Bank ProjectBH-2



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 3 of 4

PROJECT: HEAD OFFICE BUILDING
LOCATION: ADDIS ABABA, SENGATERA
CLIENT: UNITED BANK SHARE COMPANY
DATE STARTED: 02/08/2013
DATE COMPLETED: 24/08/2013
BORING TYPE: ROTARY CORING
GROUND WATER LEVEL: 6.00m
BH ELEVATION: 2356m
INCLINATION: VERTICAL
COORDINATES: 472285.14E 996156.87N

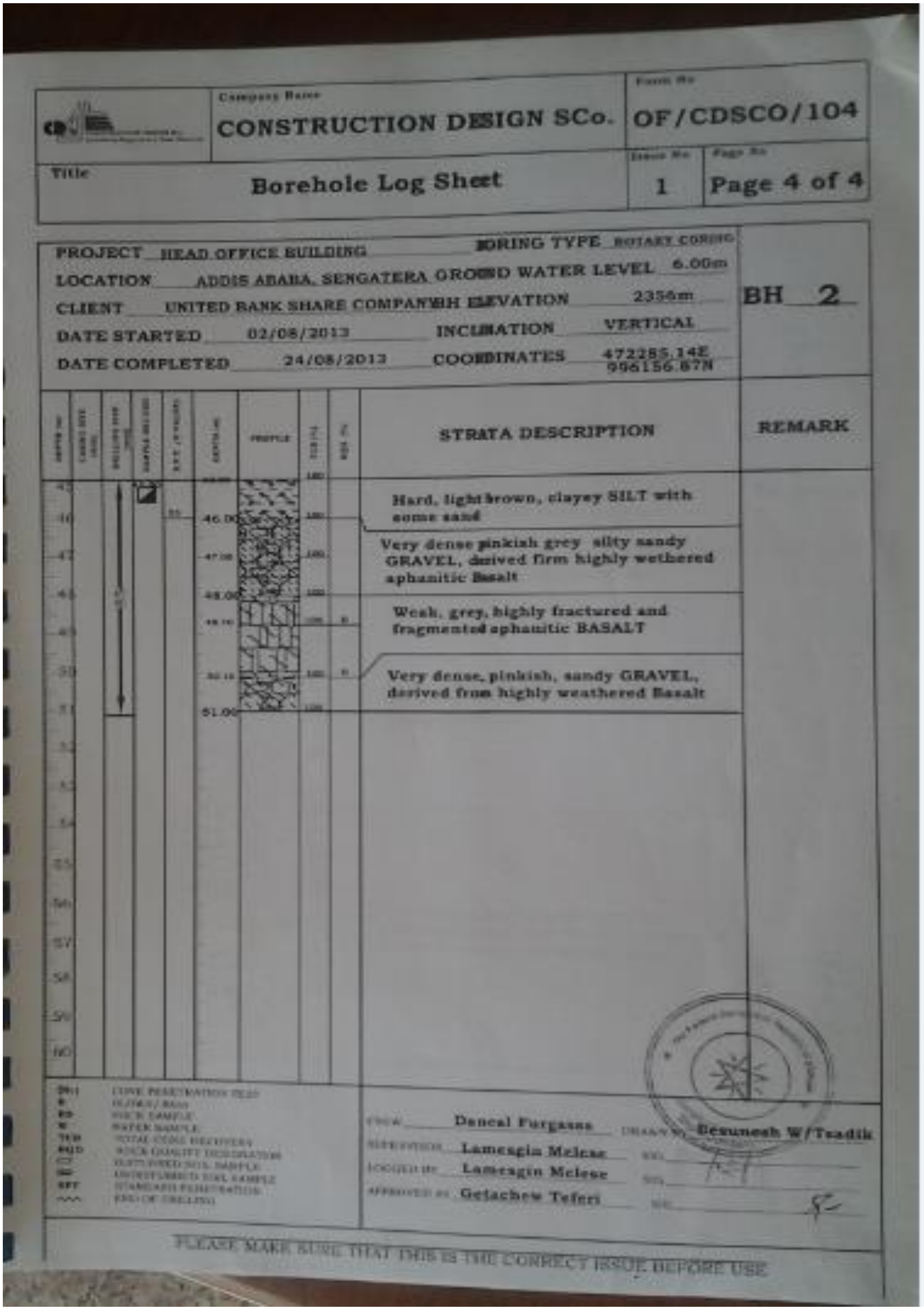
BH 2

| Depth (m) | Profile | STRATA DESCRIPTION | REMARK |
|---|---|---|---|
| 30.00 – 32.00 | | Hard, reddish brown silty CLAY (paleosol) | |
| 32.00 – 34.40 | | Very dense, light grey, sandy GRAVEL, (highly weathered rock) | |
| 34.40 – 39.60 | | Light to dark grey, highly fractured, weathered, weak BASALT | |
| 39.60 – 41.10 | | Hard, reddish brown silty CLAY (paleo soil) | |
| 41.10 – 44.00 | | Very dense, light grey, sandy GRAVEL, (highly weathered rock) | |
| 44.00 – 45.00 | | Hard, light brown, clayey SILT with some sand | |

CREW: Daneal Furgassa
DRAWN BY: Bezunesh W/Tsadik
SUPERVISOR: Lamesgin Melese
LOGGED BY: Lamesgin Melese
APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.g** United Bank ProjectBH-3



| | | | |
|---|---|---|---|
| | Company Name: CONSTRUCTION DESIGN SCo. | Form No: OF/CDSCO/104 | |
| Title: Borehole Log Sheet | | Issue No: 1 | Page No: Page 4 of 4 |

PROJECT: HEAD OFFICE BUILDING — BORING TYPE: ROTARY CORING
LOCATION: ADDIS ABABA, SENGATERA — GROUND WATER LEVEL: 6.00m
CLIENT: UNITED BANK SHARE COMPANY — BH ELEVATION: 2356m
DATE STARTED: 02/08/2013 — INCLINATION: VERTICAL
DATE COMPLETED: 24/08/2013 — COORDINATES: 472285.14E 996156.87N

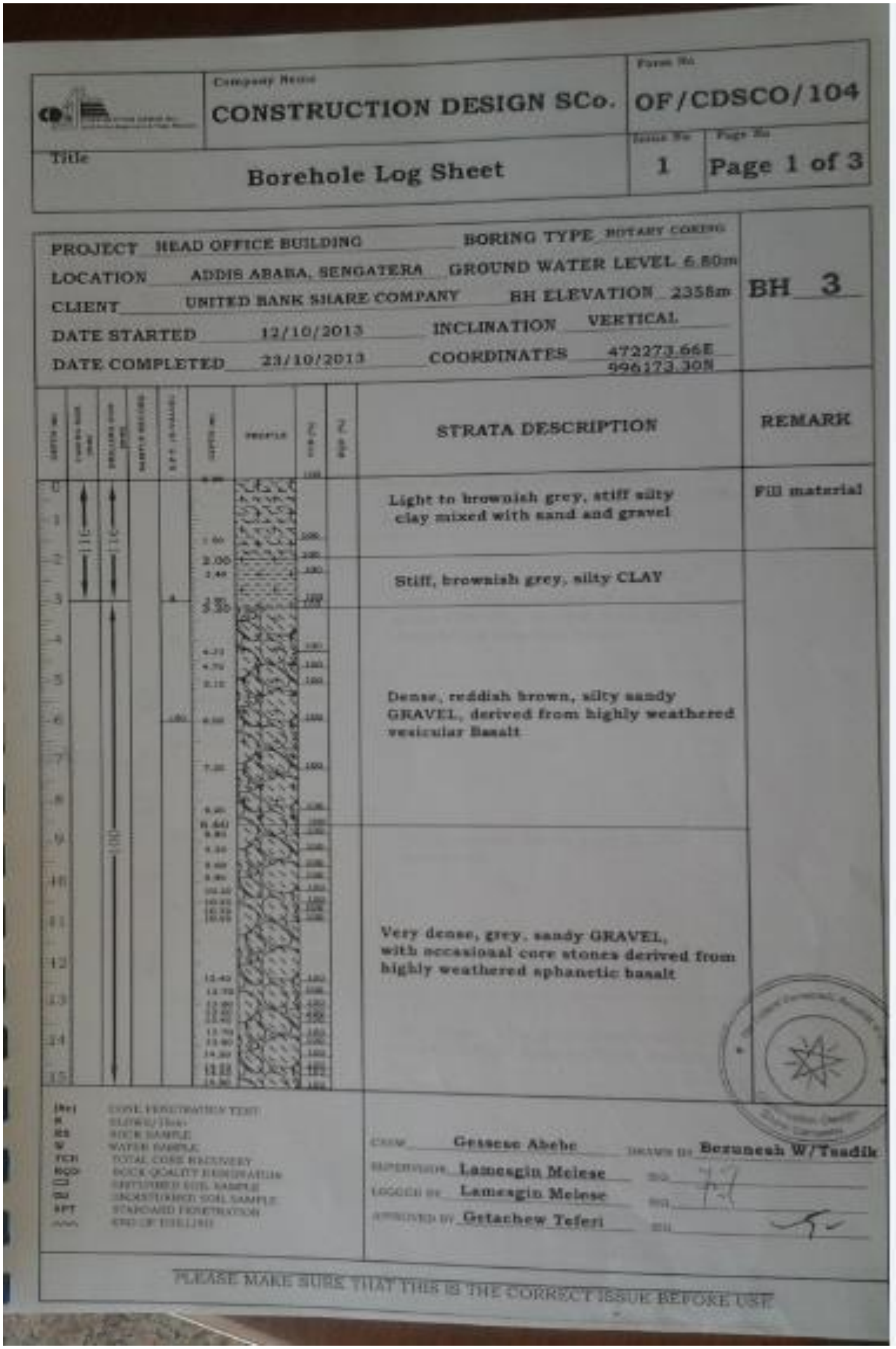
BH 2

| Depth | Profile | Strata Description | Remark |
|---|---|---|---|
| 45–46.00 | | Hard, light brown, clayey SILT with some sand | |
| 46.00–48.00 | | Very dense pinkish grey silty sandy GRAVEL, derived firm highly wethered aphanitic Basalt | |
| 48.00–50.10 | | Weak, grey, highly fractured and fragmented aphanitic BASALT | |
| 50.10–51.00 | | Very dense, pinkish, sandy GRAVEL, derived from highly weathered Basalt | |

DRAWN BY: Desuneh W/Tsadik
Daneal Furgasse
SUPERVISOR: Lamesgin Melese
LOGGED BY: Lamesgin Melese
APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.h** United Bank ProjectBH-3



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 1 of 3

| | | | |
|---|---|---|---|
| PROJECT | HEAD OFFICE BUILDING | BORING TYPE | ROTARY CORING |
| LOCATION | ADDIS ABABA, SENGATERA | GROUND WATER LEVEL | 6.80m |
| CLIENT | UNITED BANK SHARE COMPANY | BH ELEVATION | 2358m |
| DATE STARTED | 12/10/2013 | INCLINATION | VERTICAL |
| DATE COMPLETED | 23/10/2013 | COORDINATES | 472273.66E 996173.30N |

BH 3

| STRATA DESCRIPTION | REMARK |
|---|---|
| Light to brownish grey, stiff silty clay mixed with sand and gravel | Fill material |
| Stiff, brownish grey, silty CLAY | |
| Dense, reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt | |
| Very dense, grey, sandy GRAVEL, with occasional core stones derived from highly weathered aphanetic basalt | |

CREW: Gessese Abebe

DRAWN BY: Bezuneh W/Tsadik

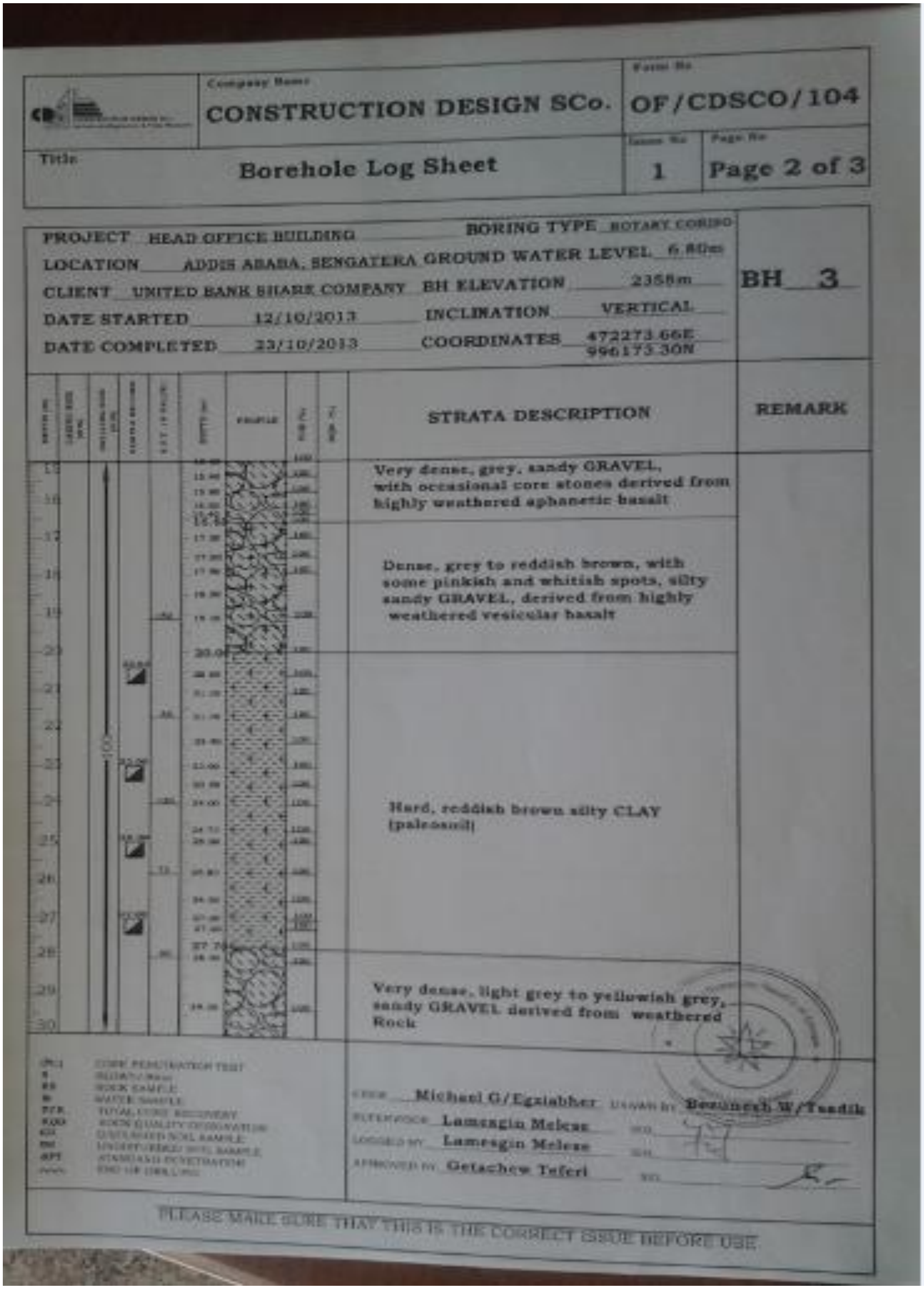
SUPERVISOR: Lamesgin Melese

LOGGED BY: Lamesgin Melese

APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.i** United Bank ProjectBH-3



| Company Name | | Form No |
|---|---|---|
| CONSTRUCTION DESIGN SCo. | | OF/CDSCO/104 |
| Title: Borehole Log Sheet | Issue No 1 | Page 2 of 3 |

PROJECT HEAD OFFICE BUILDING
BORING TYPE ROTARY CORING
LOCATION ADDIS ABABA, SENGATERA
GROUND WATER LEVEL 6.80m
CLIENT UNITED BANK SHARE COMPANY
BH ELEVATION 2358m
DATE STARTED 12/10/2013
INCLINATION VERTICAL
DATE COMPLETED 23/10/2013
COORDINATES 472273.66E 996173.30N

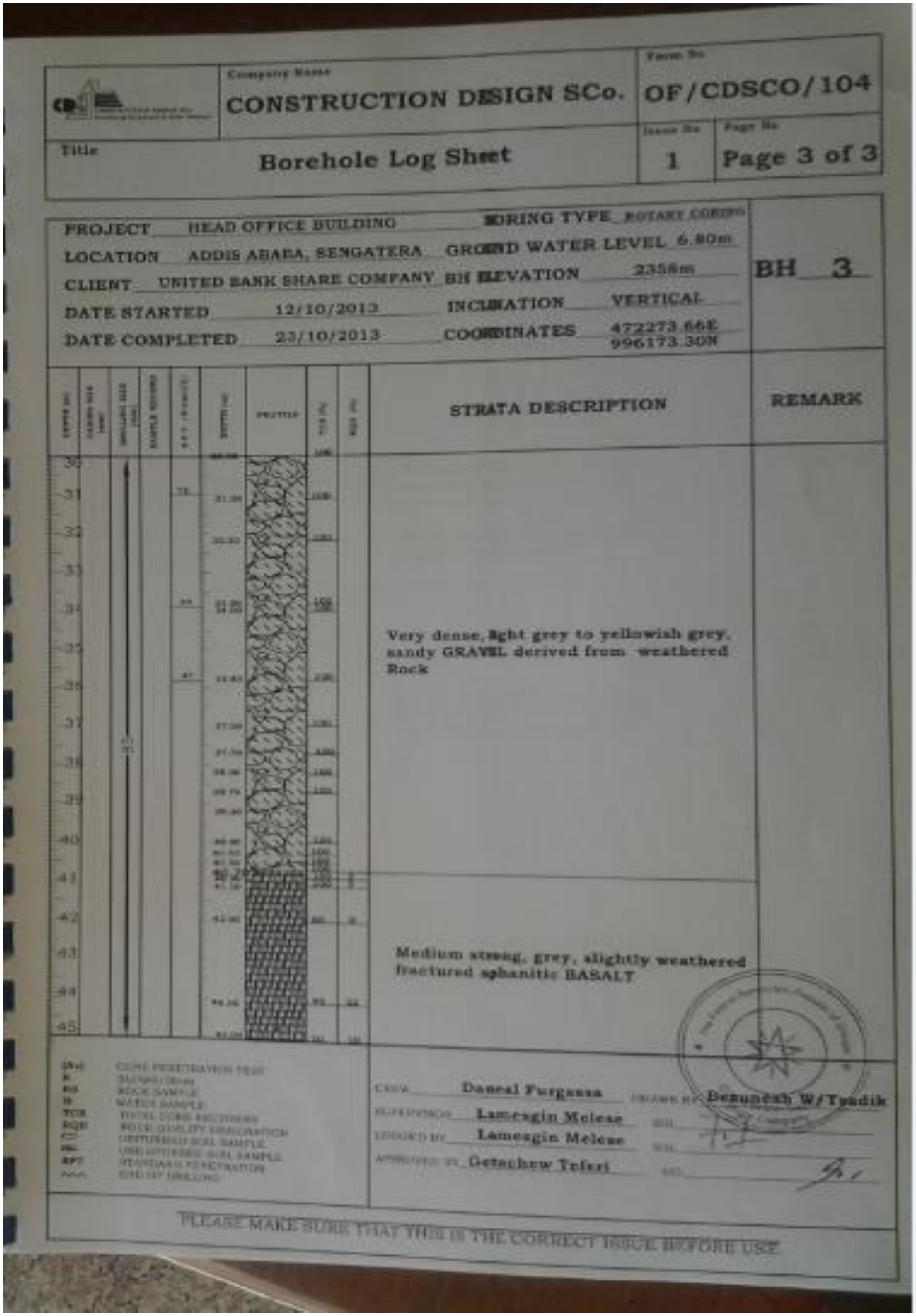
BH 3

| STRATA DESCRIPTION | REMARK |
|---|---|
| Very dense, grey, sandy GRAVEL, with occasional core stones derived from highly weathered aphanetic basalt | |
| Dense, grey to reddish brown, with some pinkish and whitish spots, silty sandy GRAVEL, derived from highly weathered vesicular basalt | |
| Hard, reddish brown silty CLAY (paleosoil) | |
| Very dense, light grey to yellowish grey, sandy GRAVEL derived from weathered Rock | |

Michael G/Egziabher DRAWN BY Bezuneh W/Tsadik
Lamesgin Melese
LOGGED BY Lamesgin Melese
APPROVED BY Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.j** United Bank ProjectBH-3



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 3 of 3

PROJECT: HEAD OFFICE BUILDING

LOCATION: ADDIS ABABA, SENGATERA

CLIENT: UNITED BANK SHARE COMPANY

DATE STARTED: 12/10/2013

DATE COMPLETED: 23/10/2013

BORING TYPE: ROTARY CORING

GROUND WATER LEVEL: 6.80m

BH ELEVATION: 2358m

INCLINATION: VERTICAL

COORDINATES: 472273.66E 996173.30N

BH 3

| STRATA DESCRIPTION | REMARK |
|---|---|
| Very dense, light grey to yellowish grey, sandy GRAVEL derived from weathered Rock | |
| Medium strong, grey, slightly weathered fractured aphanitic BASALT | |

CREW: Daneal Furgassa

SUPERVISOR: Lamesgin Melese

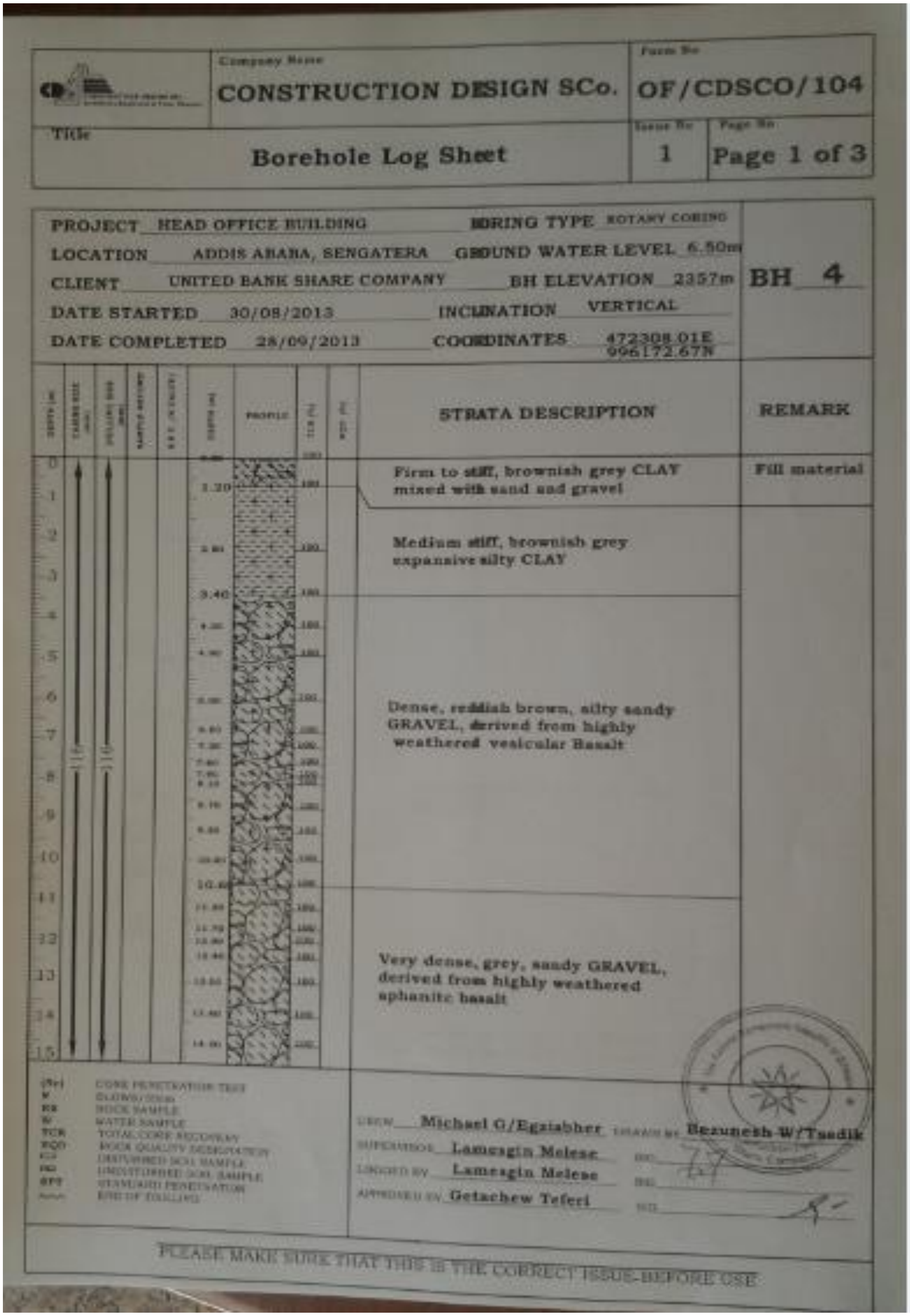
LOGGED BY: Lamesgin Melese

APPROVED BY: Getachew Teferi

DRAWN BY: Beaunesh W/Tsadik

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.k** United Bank ProjectBH-4



| Company Name | Form No |
|---|---|
| CONSTRUCTION DESIGN SCo. | OF/CDSCO/104 |

| Title | Issue No | Page No |
|---|---|---|
| Borehole Log Sheet | 1 | Page 1 of 3 |

PROJECT HEAD OFFICE BUILDING — BORING TYPE ROTARY CORING

LOCATION ADDIS ABABA, SENGATERA — GROUND WATER LEVEL 6.50m

CLIENT UNITED BANK SHARE COMPANY — BH ELEVATION 2357m — BH 4

DATE STARTED 30/08/2013 — INCLINATION VERTICAL

DATE COMPLETED 28/09/2013 — COORDINATES 472308.01E 996172.67N

| Depth (m) | Strata Description | Remark |
|---|---|---|
| 0.00 – 1.20 | Firm to stiff, brownish grey CLAY mixed with sand and gravel | Fill material |
| 1.20 – 3.40 | Medium stiff, brownish grey expansive silty CLAY | |
| 3.40 – 10.60 | Dense, reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt | |
| 10.60 – 15.00 | Very dense, grey, sandy GRAVEL, derived from highly weathered aphanitic basalt | |

(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
UD UNDISTURBED SOIL SAMPLE
SPT STANDARD PENETRATION
END OF DRILLING

CREW Michael G/Egziabher — DRAWN BY Bezunesh W/Tsadik

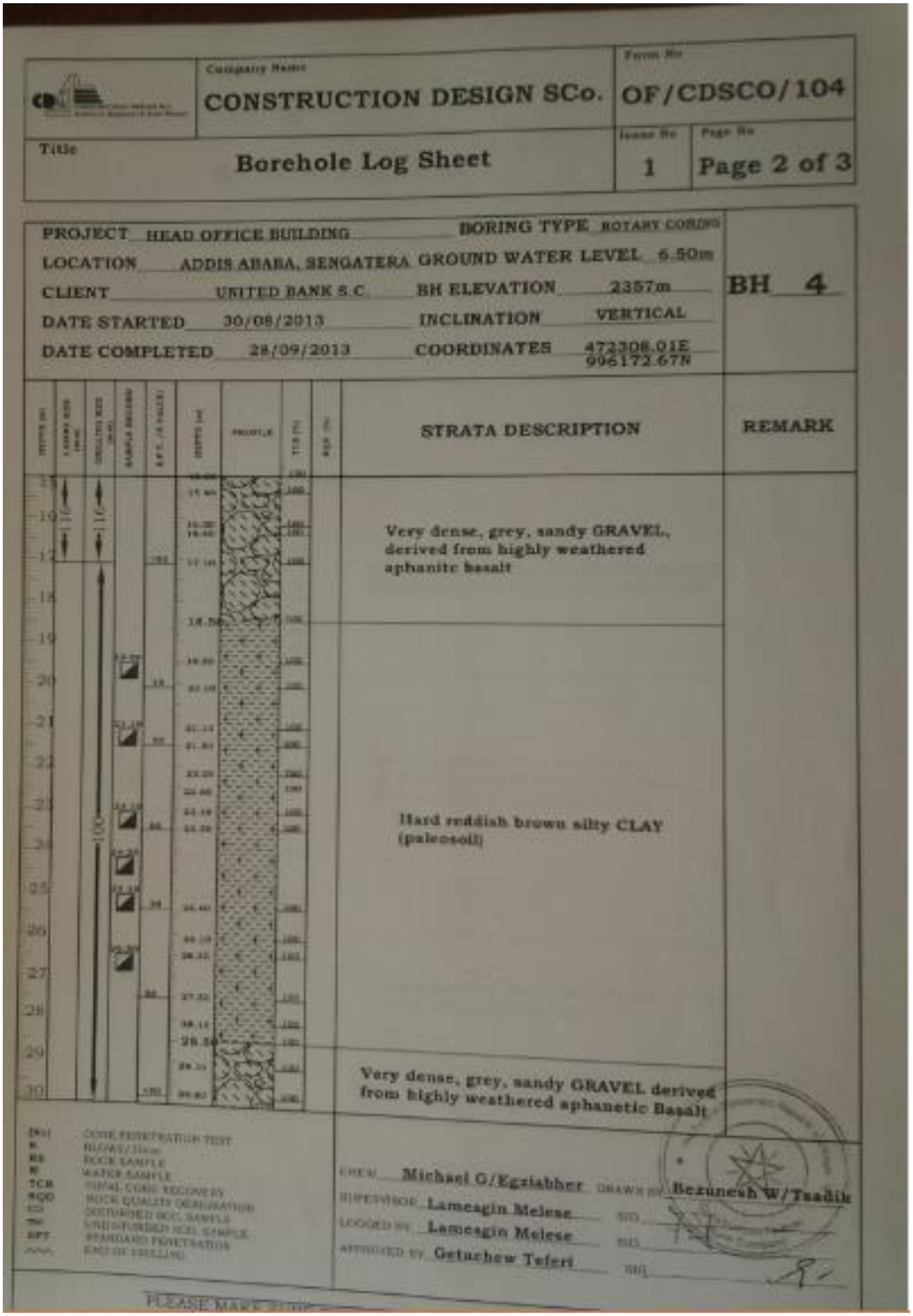
SUPERVISOR Lamesgin Melese

LOGGED BY Lamesgin Melese

APPROVED BY Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.m** United Bank ProjectBH-4



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 2 of 3

PROJECT HEAD OFFICE BUILDING

BORING TYPE ROTARY CORING

LOCATION ADDIS ABABA, SENGATERA

GROUND WATER LEVEL 6.50m

CLIENT UNITED BANK S.C

BH ELEVATION 2357m

DATE STARTED 30/08/2013

INCLINATION VERTICAL

DATE COMPLETED 28/09/2013

COORDINATES 472308.01E 996172.67N

BH 4

| STRATA DESCRIPTION | REMARK |
|---|---|
| Very dense, grey, sandy GRAVEL, derived from highly weathered aphanitc basalt | |
| Hard reddish brown silty CLAY (paleosoil) | |
| Very dense, grey, sandy GRAVEL derived from highly weathered aphanetic Basalt | |

CREW Michael G/Egziabher

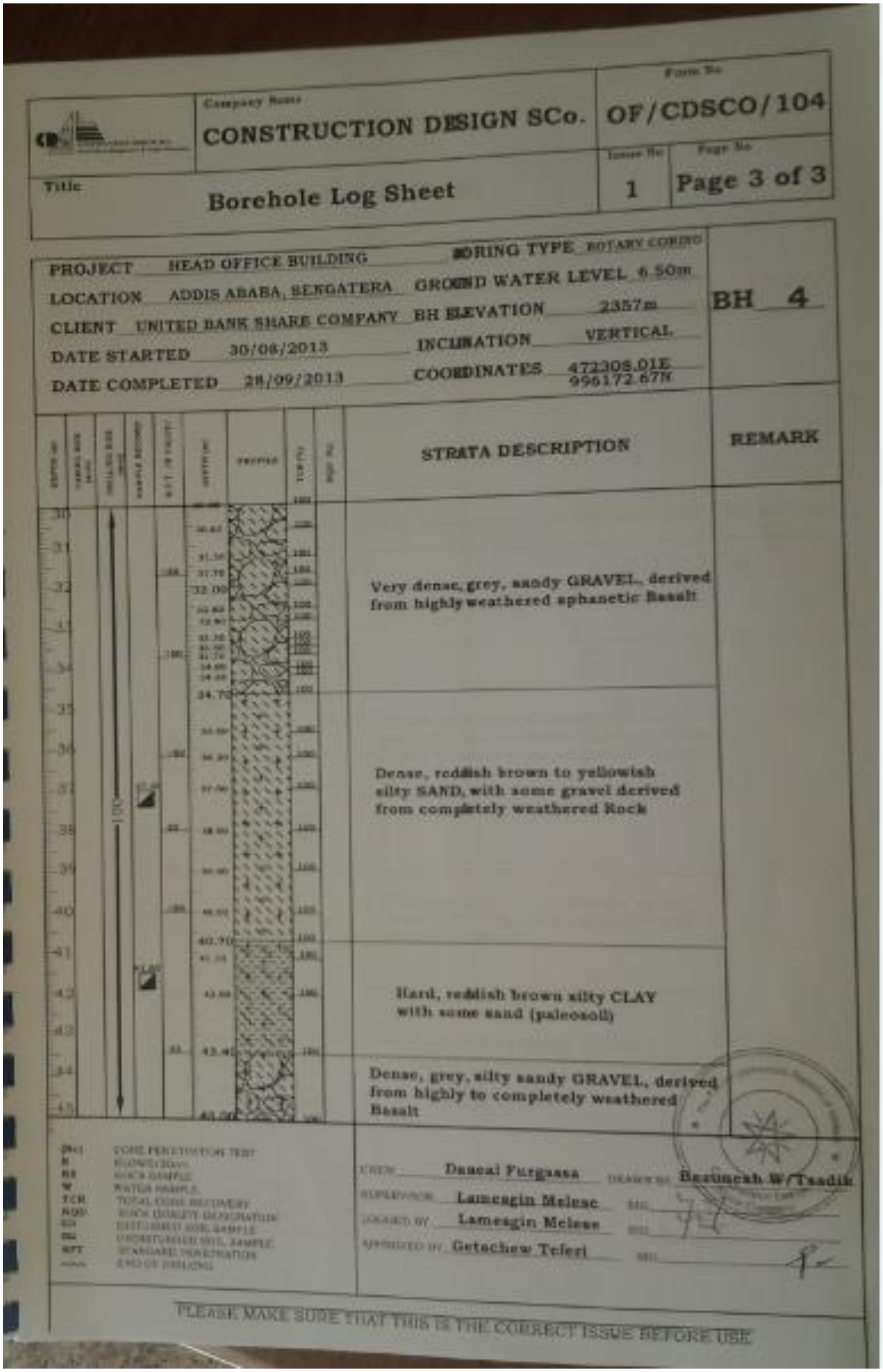
DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Lameagin Melese

LOGGED BY Lameagin Melese

APPROVED BY Getachew Teferi

**Table C.2.n** United Bank ProjectBH-4



CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 3 of 3

PROJECT: HEAD OFFICE BUILDING
LOCATION: ADDIS ABABA, SENGATERA
CLIENT: UNITED BANK SHARE COMPANY
DATE STARTED: 30/08/2013
DATE COMPLETED: 28/09/2013

BORING TYPE: ROTARY CORING
GROUND WATER LEVEL: 6.50m
BH ELEVATION: 2357m
INCLINATION: VERTICAL
COORDINATES: 472308.01E 996172.67N

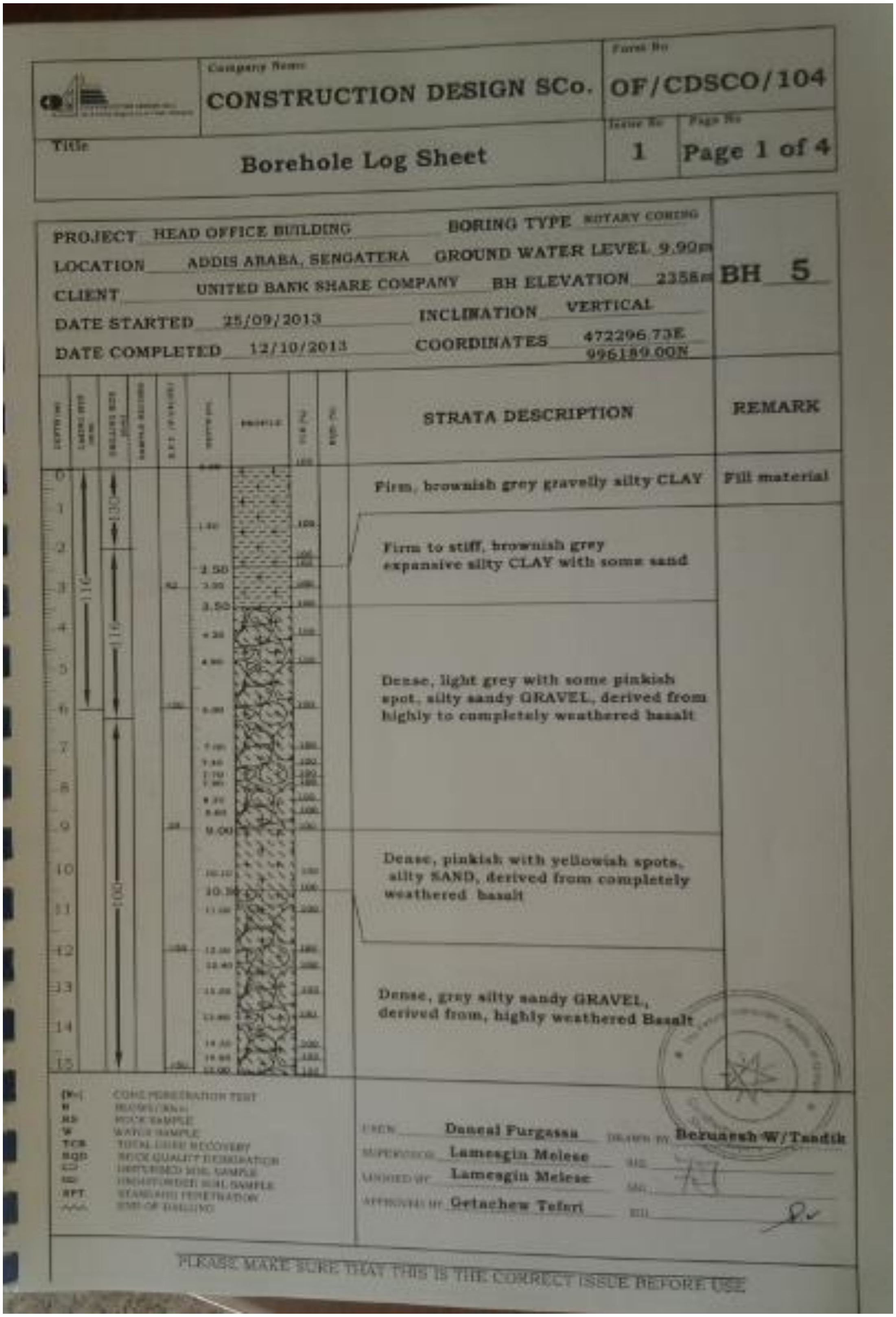
BH 4

| Depth (m) | Strata Description | Remark |
|---|---|---|
| 30–34.70 | Very dense, grey, sandy GRAVEL, derived from highly weathered aphanetic Basalt | |
| 34.70–40.70 | Dense, reddish brown to yellowish silty SAND, with some gravel derived from completely weathered Rock | |
| 40.70–43.40 | Hard, reddish brown silty CLAY with some sand (paleosoil) | |
| 43.40–45.00 | Dense, grey, silty sandy GRAVEL, derived from highly to completely weathered Basalt | |

CREW: Daneal Furgassa
DRAWN BY: Bezuneeh W/Tsadik
SUPERVISOR: Lamesgin Melese
LOGGED BY: Lamesgin Melese
APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.o** United Bank ProjectBH-5



| Company Name | CONSTRUCTION DESIGN SCo. | Form No OF/CDSCO/104 | |
|---|---|---|---|
| Title | Borehole Log Sheet | Issue No 1 | Page No Page 1 of 4 |

PROJECT HEAD OFFICE BUILDING — BORING TYPE ROTARY CORING
LOCATION ADDIS ABABA, SENGATERA — GROUND WATER LEVEL 9.90m
CLIENT UNITED BANK SHARE COMPANY — BH ELEVATION 2358m
DATE STARTED 25/09/2013 — INCLINATION VERTICAL
DATE COMPLETED 12/10/2013 — COORDINATES 472296.73E 996189.00N

BH 5

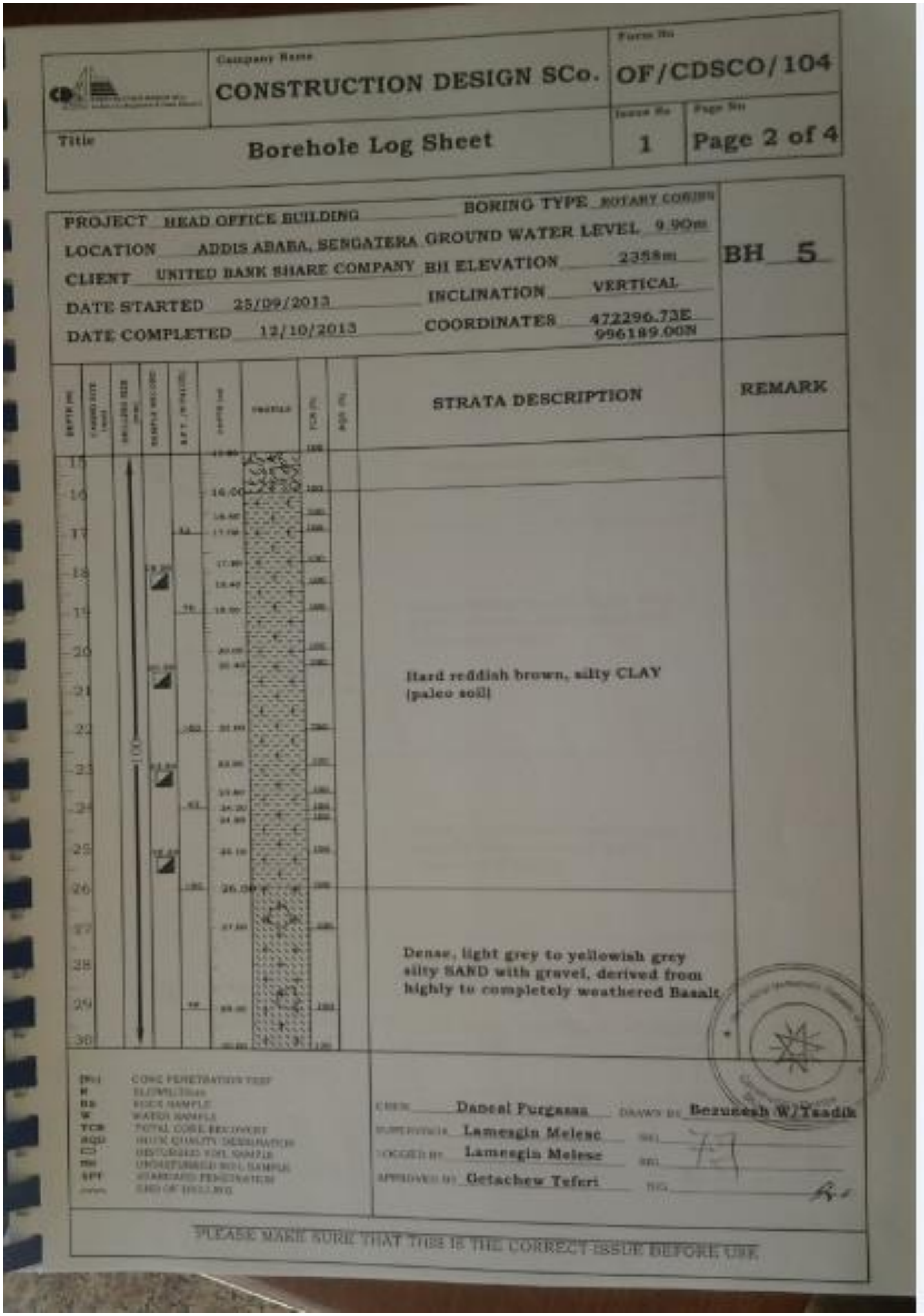
| Depth (m) | STRATA DESCRIPTION | REMARK |
|---|---|---|
| 0.00–1.80 | Firm, brownish grey gravelly silty CLAY | Fill material |
| 1.80–3.50 | Firm to stiff, brownish grey expansive silty CLAY with some sand | |
| 3.50–9.00 | Dense, light grey with some pinkish spot, silty sandy GRAVEL, derived from highly to completely weathered basalt | |
| 9.00–11.60 | Dense, pinkish with yellowish spots, silty SAND, derived from completely weathered basalt | |
| 11.60–15.00 | Dense, grey silty sandy GRAVEL, derived from, highly weathered Basalt | |

CONE PENETRATION TEST
BLOWS/(N...)
ROCK SAMPLE
WATER SAMPLE
TCR TOTAL CORE RECOVERY
RQD ROCK QUALITY DESIGNATION
DISTURBED SOIL SAMPLE
UNDISTURBED SOIL SAMPLE
SPT STANDARD PENETRATION
END OF DRILLING

CREW Daneal Furgassa — DRAWN BY Bezunesh W/Tsadik
SUPERVISOR Lamesgin Melese
LOGGED BY Lamesgin Melese
APPROVED BY Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.p** United Bank ProjectBH-5



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 2 of 4

| | |
|---|---|
| PROJECT | HEAD OFFICE BUILDING |
| LOCATION | ADDIS ABABA, SENGATERA |
| CLIENT | UNITED BANK SHARE COMPANY |
| DATE STARTED | 25/09/2013 |
| DATE COMPLETED | 12/10/2013 |
| BORING TYPE | ROTARY CORING |
| GROUND WATER LEVEL | 9.90m |
| BH ELEVATION | 2358m |
| INCLINATION | VERTICAL |
| COORDINATES | 472296.73E 996189.00N |

BH 5

| Depth (m) | Strata Description | Remark |
|---|---|---|
| 15–16 | | |
| 16.00–26.00 | Hard reddish brown, silty CLAY (paleo soil) | |
| 26.00–30.00 | Dense, light grey to yellowish grey silty SAND with gravel, derived from highly to completely weathered Basalt | |

CHECKED BY: Daneal Purgasaa

DRAWN BY: Bezuneesh W/Tsadik

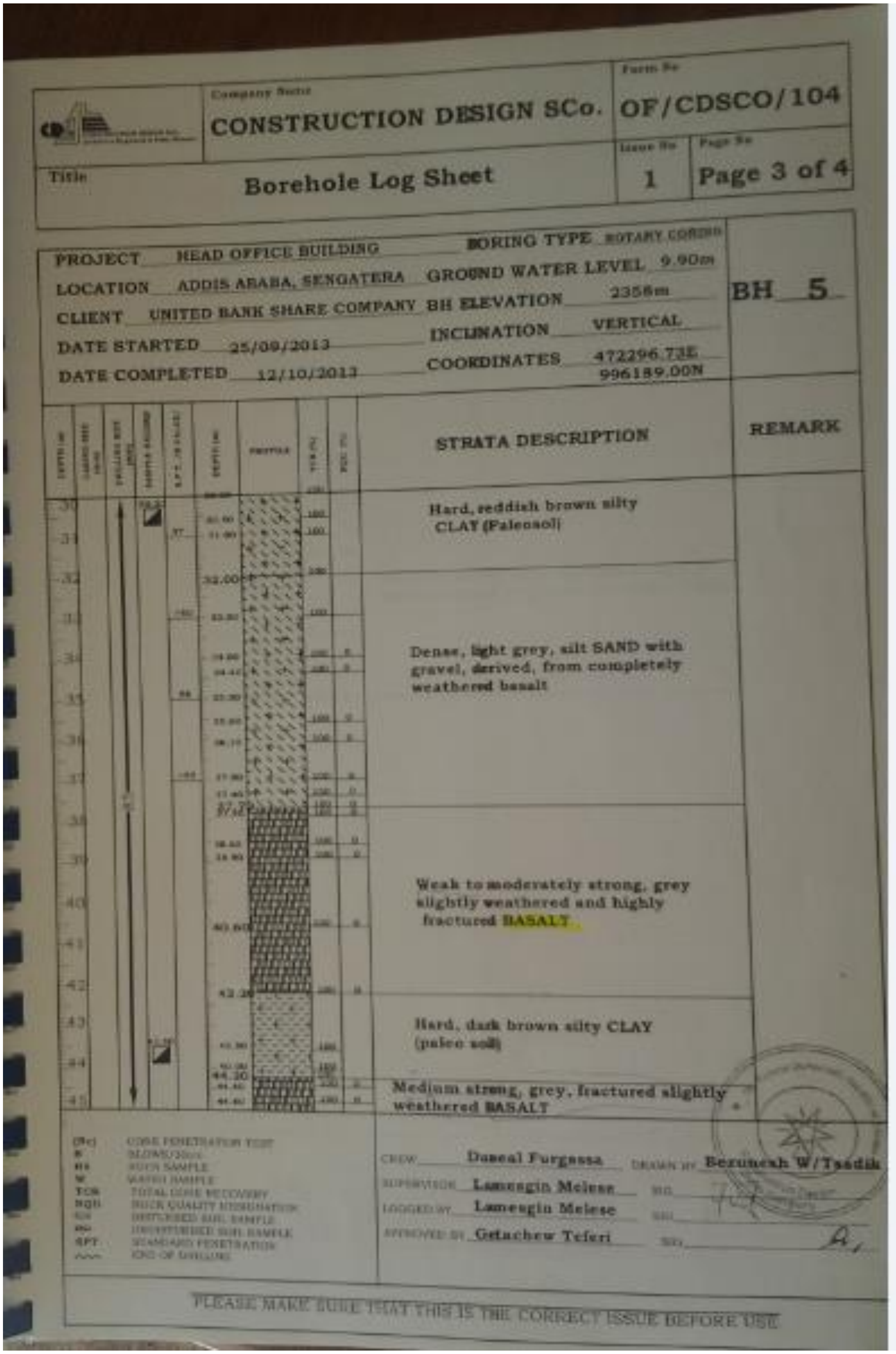
SUPERVISED BY: Lamesgin Melese

LOGGED BY: Lamesgin Melese

APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.q** United Bank ProjectBH-5


Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 3 of 4

PROJECT: HEAD OFFICE BUILDING

LOCATION: ADDIS ABABA, SENGATERA

CLIENT: UNITED BANK SHARE COMPANY

DATE STARTED: 25/09/2013

DATE COMPLETED: 12/10/2013

BORING TYPE: ROTARY CORING

GROUND WATER LEVEL: 9.90m

BH ELEVATION: 2358m

INCLINATION: VERTICAL

COORDINATES: 472296.73E 996189.00N

BH 5

| Depth (m) | Strata Description | Remark |
|---|---|---|
| 30–32.00 | Hard, reddish brown silty CLAY (Paleosol) | |
| 32.00–37.7 | Dense, light grey, silt SAND with gravel, derived, from completely weathered basalt | |
| 37.7–42.20 | Weak to moderately strong, grey slightly weathered and highly fractured BASALT | |
| 42.20–44.30 | Hard, dark brown silty CLAY (paleo soil) | |
| 44.30–45 | Medium strong, grey, fractured slightly weathered BASALT | |

CREW: Dassal Furgassa

DRAWN BY: Bezuneah W/Tsadik

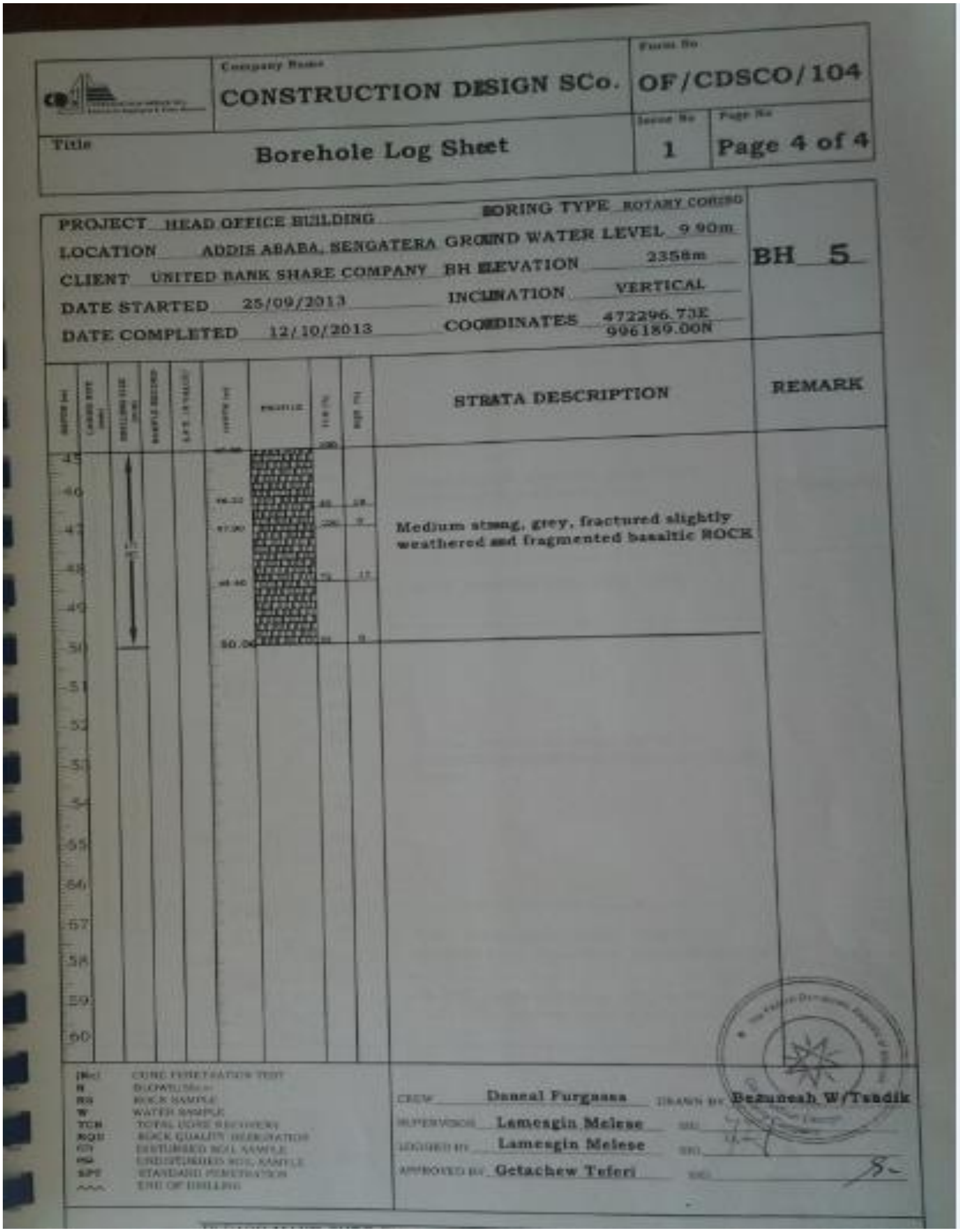
SUPERVISOR: Lamesgin Melese

LOGGED BY: Lamesgin Melese

APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.r** United Bank ProjectBH-5



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 4 of 4

PROJECT: HEAD OFFICE BUILDING

BORING TYPE: ROTARY CORING

LOCATION: ADDIS ABABA, SENGATERA

GROUND WATER LEVEL: 9.90m

CLIENT: UNITED BANK SHARE COMPANY

BH ELEVATION: 2358m

DATE STARTED: 25/09/2013

INCLINATION: VERTICAL

DATE COMPLETED: 12/10/2013

COORDINATES: 472296.73E 996189.00N

BH 5

PROFILE | STRATA DESCRIPTION | REMARK

Medium strong, grey, fractured slightly weathered and fragmented basaltic ROCK

CONE PENETRATION TEST

ROCK SAMPLE

WATER SAMPLE

TCR TOTAL CORE RECOVERY

RQD ROCK QUALITY DESIGNATION

DISTURBED SOIL SAMPLE

UNDISTURBED SOIL SAMPLE

SPT STANDARD PENETRATION

END OF DRILLING

CREW: Daneal Furgassa

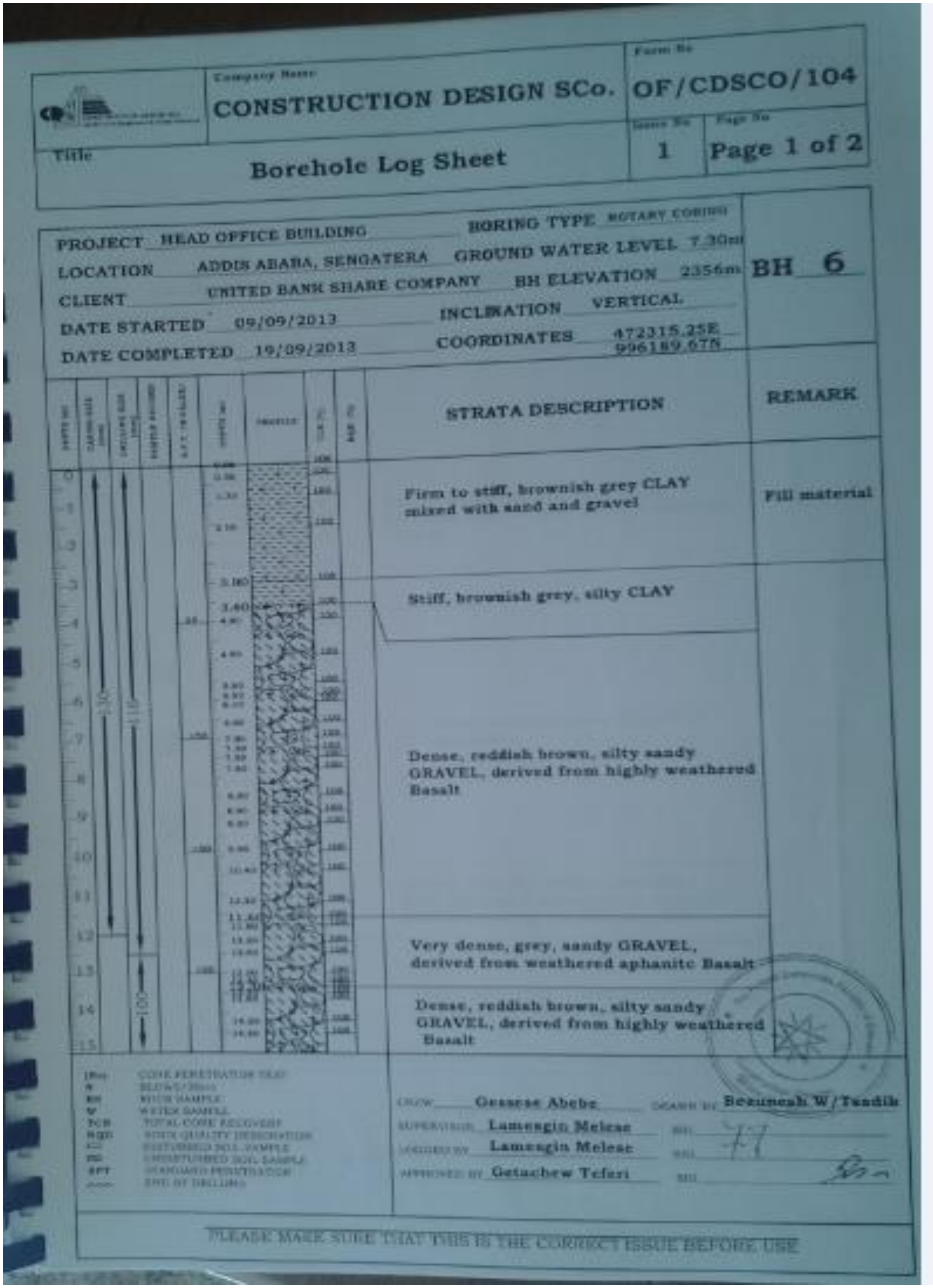
DRAWN BY: Bezuneah W/Tsadik

SUPERVISOR: Lamesgin Melese

LOGGED BY: Lamesgin Melese

APPROVED BY: Getachew Teferi

**Table C.2.s** United Bank ProjectBH-6



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Issue No: 1 | Page No: Page 1 of 2

Title: Borehole Log Sheet

PROJECT: HEAD OFFICE BUILDING

LOCATION: ADDIS ABABA, SENGATERA

CLIENT: UNITED BANK SHARE COMPANY

DATE STARTED: 09/09/2013

DATE COMPLETED: 19/09/2013

BORING TYPE: ROTARY CORING

GROUND WATER LEVEL: 7.30m

BH ELEVATION: 2356m

INCLINATION: VERTICAL

COORDINATES: 472315.25E 996189.67N

BH 6

| STRATA DESCRIPTION | REMARK |
|---|---|
| Firm to stiff, brownish grey CLAY mixed with sand and gravel | Fill material |
| Stiff, brownish grey, silty CLAY | |
| Dense, reddish brown, silty sandy GRAVEL, derived from highly weathered Basalt | |
| Very dense, grey, sandy GRAVEL, derived from weathered aphanitic Basalt | |
| Dense, reddish brown, silty sandy GRAVEL, derived from highly weathered Basalt | |

CREW: Gessese Abebe

DRAWN BY: Bezuneh W/Tsadik

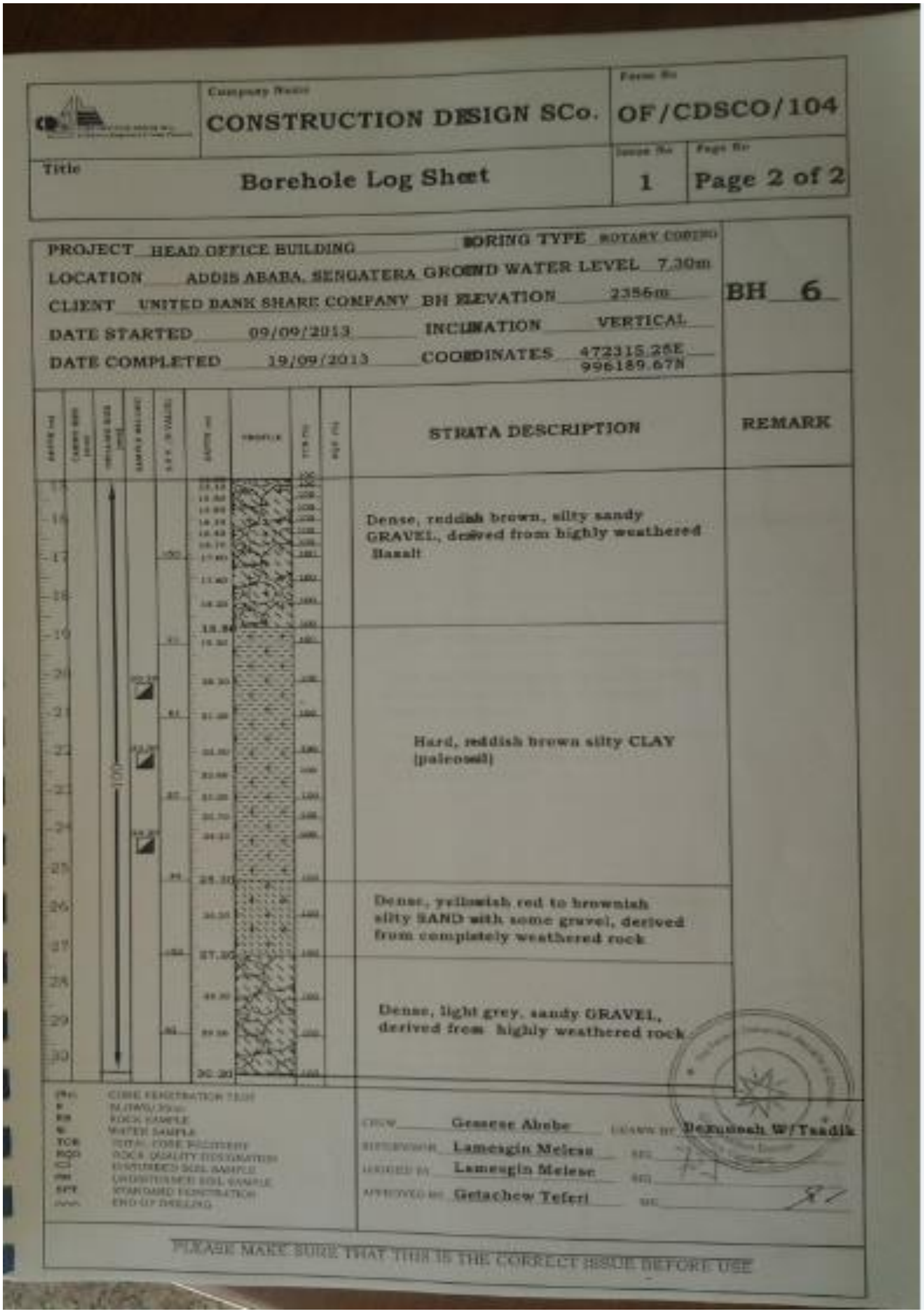
SUPERVISOR: Lamesgin Melese

CHECKED BY: Lamesgin Melese

APPROVED BY: Getachew Teferi

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE

**Table C.2.t** United Bank ProjectBH-6



Company Name: CONSTRUCTION DESIGN SCo. | Form No: OF/CDSCO/104

Title: Borehole Log Sheet | Issue No: 1 | Page No: Page 2 of 2

PROJECT: HEAD OFFICE BUILDING
BORING TYPE: ROTARY CORING
LOCATION: ADDIS ABABA, SENGATERA
GROUND WATER LEVEL: 7.30m
CLIENT: UNITED BANK SHARE COMPANY
BH ELEVATION: 2356m
DATE STARTED: 09/09/2013
INCLINATION: VERTICAL
DATE COMPLETED: 19/09/2013
COORDINATES: 472315.25E 996189.67N

BH 6

| STRATA DESCRIPTION | REMARK |
| --- | --- |
| Dense, reddish brown, silty sandy GRAVEL, derived from highly weathered Basalt | |
| Hard, reddish brown silty CLAY (paleosoil) | |
| Dense, yellowish red to brownish silty SAND with some gravel, derived from completely weathered rock | |
| Dense, light grey, sandy GRAVEL, derived from highly weathered rock | |

CREW: Gesesse Abebe
DRAWN BY: Dezasash W/Tsadik
SUPERVISOR: Lamesgin Melese
LOGGED BY: Lamesgin Melese
APPROVED BY: Getachew Teferi

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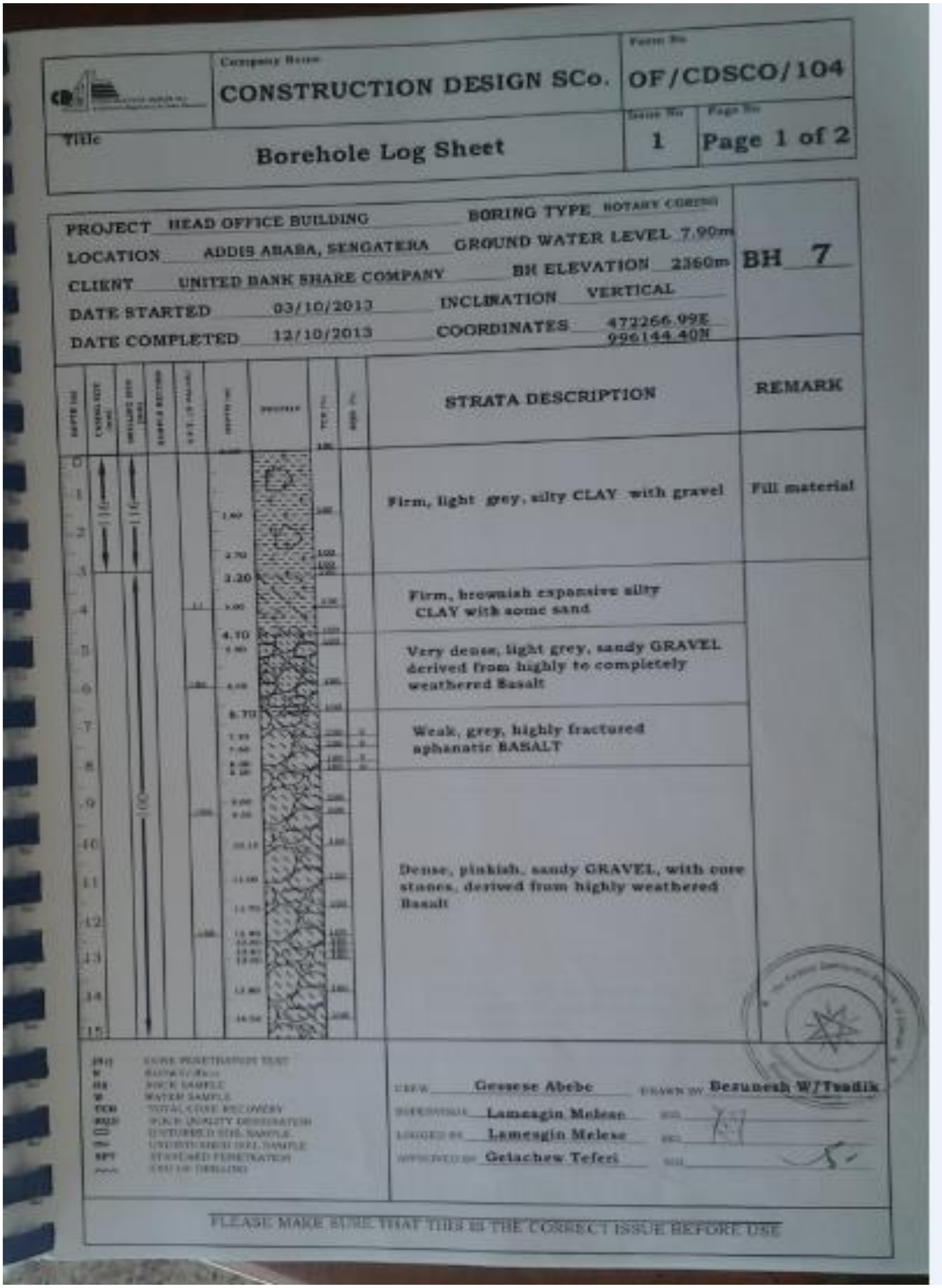
**Table C.2.u** United Bank ProjectBH-7



| Company Name | | Form No |
|---|---|---|
| CONSTRUCTION DESIGN SCo. | | OF/CDSCO/104 |
| Title: Borehole Log Sheet | Issue No: 1 | Page No: Page 1 of 2 |

PROJECT: HEAD OFFICE BUILDING
BORING TYPE: ROTARY CORING
LOCATION: ADDIS ABABA, SENGATERA
GROUND WATER LEVEL: 7.90m
CLIENT: UNITED BANK SHARE COMPANY
BH ELEVATION: 2360m
DATE STARTED: 03/10/2013
INCLINATION: VERTICAL
DATE COMPLETED: 12/10/2013
COORDINATES: 472266.99E 996144.40N

BH 7

| STRATA DESCRIPTION | REMARK |
|---|---|
| Firm, light grey, silty CLAY with gravel | Fill material |
| Firm, brownish expansive silty CLAY with some sand | |
| Very dense, light grey, sandy GRAVEL derived from highly to completely weathered Basalt | |
| Weak, grey, highly fractured aphanatic BASALT | |
| Dense, pinkish, sandy GRAVEL, with core stones, derived from highly weathered Basalt | |

Gessese Abebe
DRAWN BY: Bezunesh W/Tsadik
Lamesgin Melese
LOGGED BY: Lamesgin Melese
Getachew Teferi

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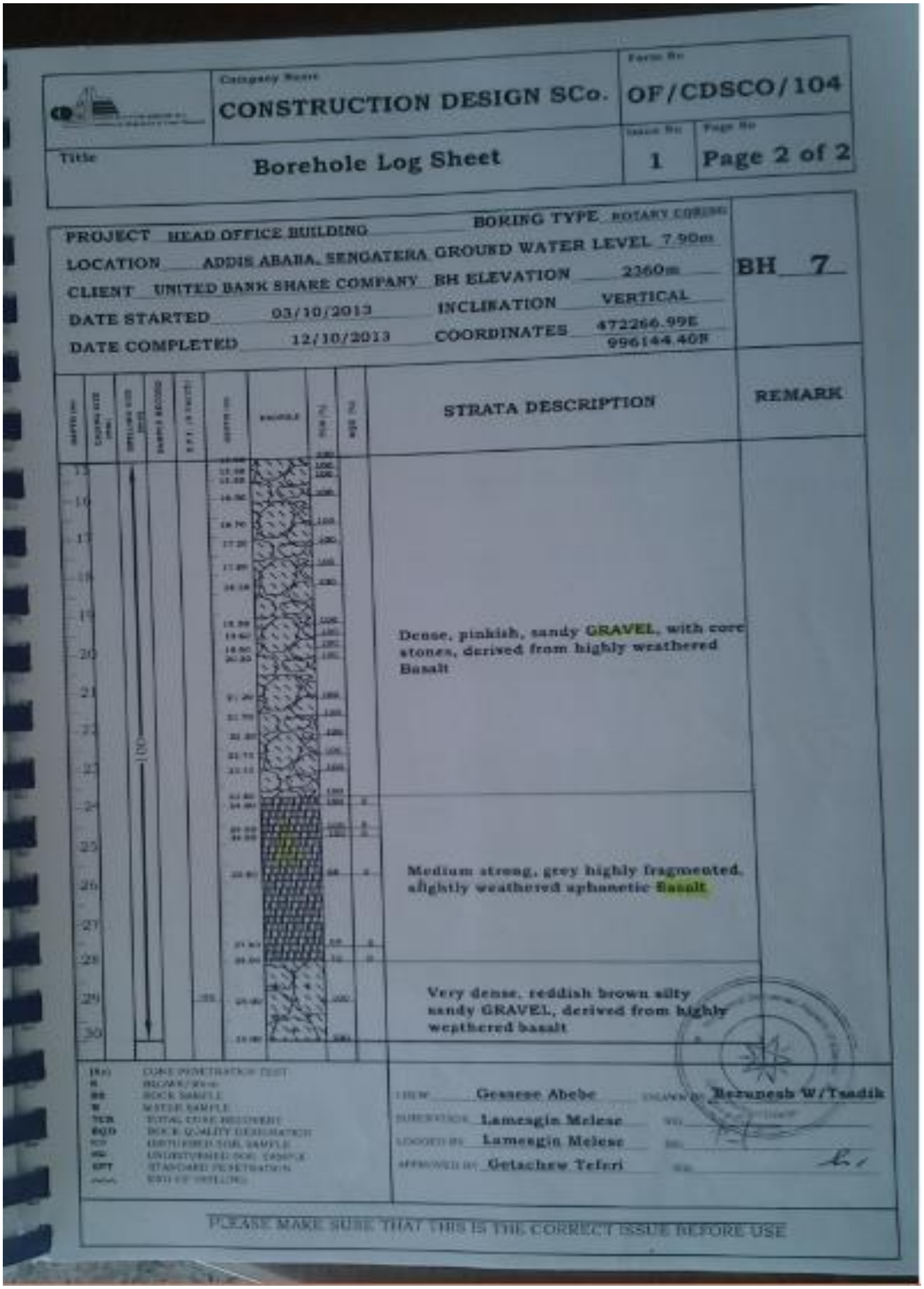
**Table C.2.v United** Bank ProjectBH-7



Company Name: CONSTRUCTION DESIGN SCo.

Form No: OF/CDSCO/104

Title: Borehole Log Sheet

Issue No: 1

Page No: Page 2 of 2

| | | | |
|---|---|---|---|
| PROJECT | HEAD OFFICE BUILDING | BORING TYPE | ROTARY CORING |
| LOCATION | ADDIS ABABA, SENGATERA | GROUND WATER LEVEL | 7.90m |
| CLIENT | UNITED BANK SHARE COMPANY | BH ELEVATION | 2360m |
| DATE STARTED | 03/10/2013 | INCLIBATION | VERTICAL |
| DATE COMPLETED | 12/10/2013 | COORDINATES | 472266.99E 996144.40N |

BH 7

| STRATA DESCRIPTION | REMARK |
|---|---|
| Dense, pinkish, sandy GRAVEL, with core stones, derived from highly weathered Basalt | |
| Medium strong, grey highly fragmented, slightly weathered aphanetic Basalt | |
| Very dense, reddish brown silty sandy GRAVEL, derived from highly weathered basalt | |

CREW: Gessese Abebe

DRAWN BY: Beruneshi W/Tsadik

SUPERVISED BY: Lamesgin Melese

LOGGED BY: Lamesgin Melese

APPROVED BY: Getachew Teferi

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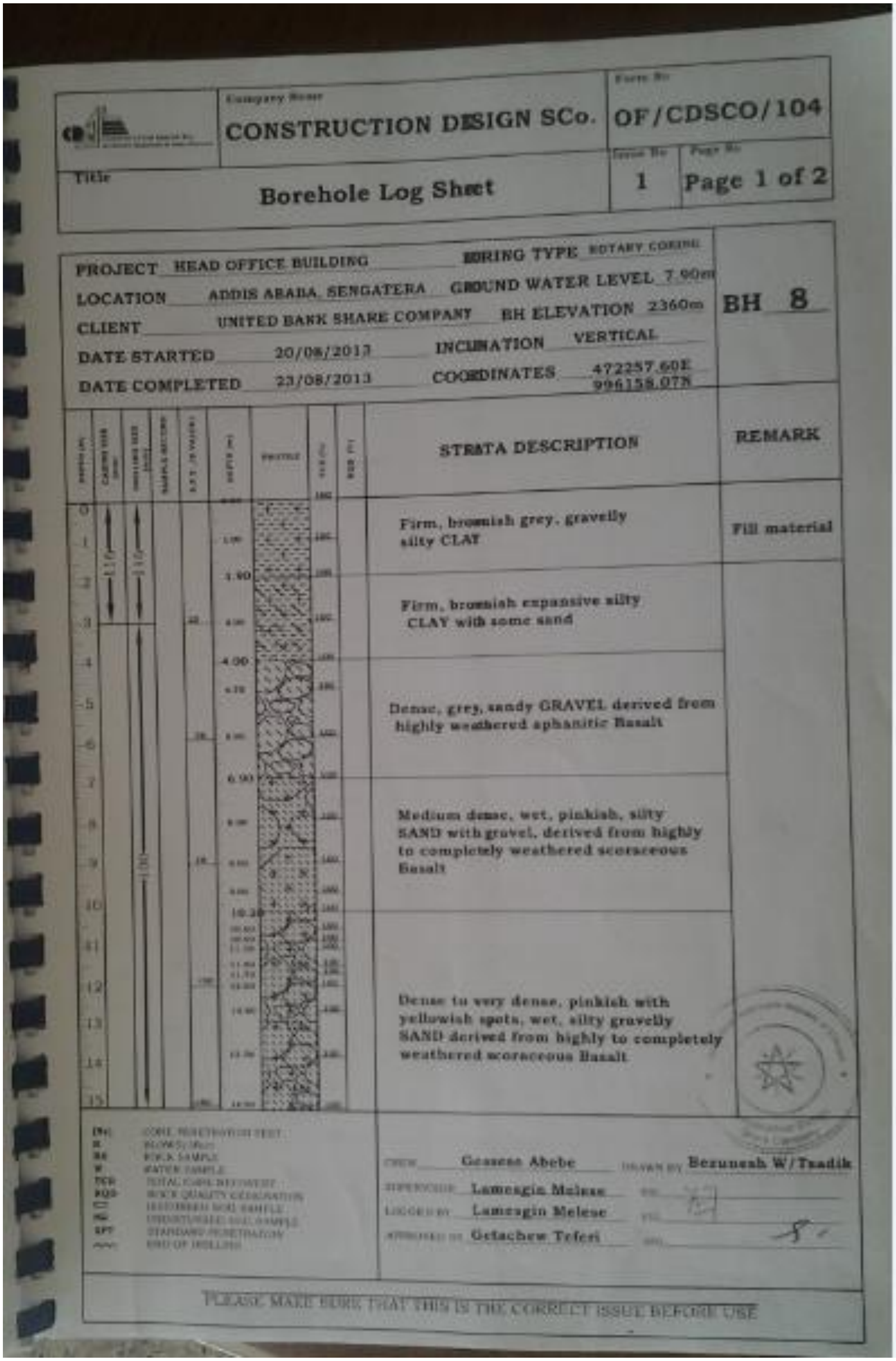
**Table C.2.w** United Bank ProjectBH-8



CONSTRUCTION DESIGN SCo. | OF/CDSCO/104

Title: Borehole Log Sheet | Issue No: 1 | Page No: Page 1 of 2

PROJECT: HEAD OFFICE BUILDING
BORING TYPE: ROTARY CORING
LOCATION: ADDIS ABABA, SENGATERA
GROUND WATER LEVEL: 7.90m
CLIENT: UNITED BANK SHARE COMPANY
BH ELEVATION: 2360m
DATE STARTED: 20/08/2013
INCLINATION: VERTICAL
DATE COMPLETED: 23/08/2013
COORDINATES: 472257.60E 996158.07N

BH 8

| STRATA DESCRIPTION | REMARK |
|---|---|
| Firm, brownish grey, gravelly silty CLAY | Fill material |
| Firm, brownish expansive silty CLAY with some sand | |
| Dense, grey, sandy GRAVEL derived from highly weathered aphanitic Basalt | |
| Medium dense, wet, pinkish, silty SAND with gravel, derived from highly to completely weathered scoraceous Basalt | |
| Dense to very dense, pinkish with yellowish spots, wet, silty gravelly SAND derived from highly to completely weathered scoraceous Basalt | |

CREW: Gessese Abebe
DRAWN BY: Bezuneah W/Tsadik
SUPERVISOR: Lamesgin Melese
LOGGED BY: Lamesgin Melese
APPROVED BY: Getachew Teferi

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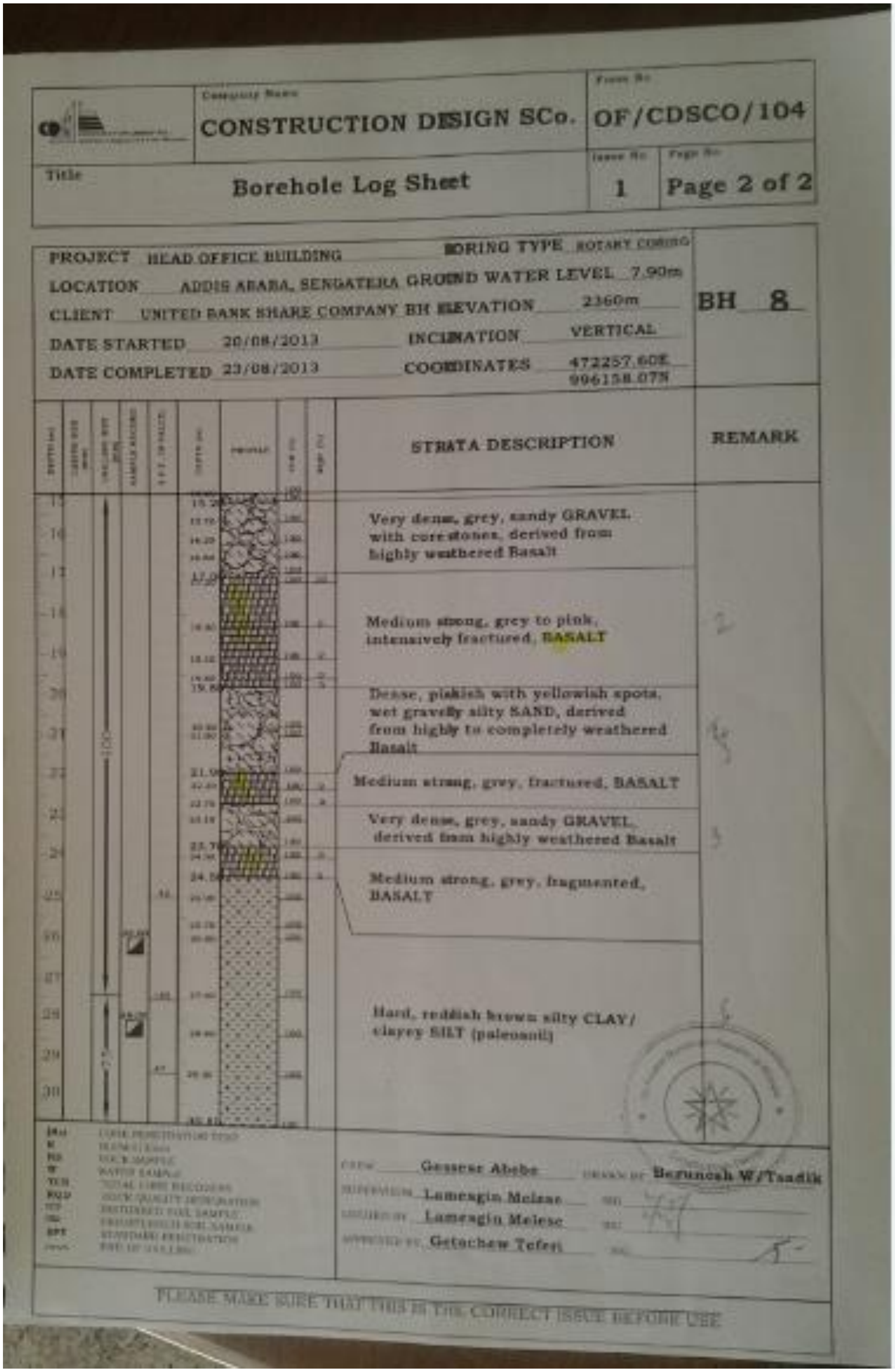
**Table C.2.r** United Bank Project BH-8



Company Name: CONSTRUCTION DESIGN SCo.
Form No: OF/CDSCO/104
Title: Borehole Log Sheet
Issue No: 1
Page No: Page 2 of 2

PROJECT: HEAD OFFICE BUILDING
BORING TYPE: ROTARY CORING
LOCATION: ADDIS ABABA, SENGATERA
GROUND WATER LEVEL: 7.90m
CLIENT: UNITED BANK SHARE COMPANY
BH ELEVATION: 2360m
DATE STARTED: 20/08/2013
INCLINATION: VERTICAL
DATE COMPLETED: 23/08/2013
COORDINATES: 472257.60E, 996158.07N

BH 8

| STRATA DESCRIPTION | REMARK |
|---|---|
| Very dense, grey, sandy GRAVEL with corestones, derived from highly weathered Basalt | |
| Medium strong, grey to pink, intensively fractured, BASALT | 2 |
| Dense, pinkish with yellowish spots, wet gravelly silty SAND, derived from highly to completely weathered Basalt | |
| Medium strong, grey, fractured, BASALT | |
| Very dense, grey, sandy GRAVEL, derived from highly weathered Basalt | |
| Medium strong, grey, fragmented, BASALT | |
| Hard, reddish brown silty CLAY/ clayey SILT (paleosoil) | |

CREW: Gessesse Abebe
DRAWN BY: Bezuneh W/Tsadik
SUPERVISOR: Lamesgin Melese
CHECKED BY: Lamesgin Melese
APPROVED BY: Getachew Tefera

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