SIMULATION OF PILE LOADING TEST IN A LAYERED SOIL WITH VERTICAL LOADING BY USING FINITE DIFFERENCE METHOD BASED SOFTWARE

MSc. THESIS

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SIMULATION OF PILE LOADING TEST IN A LAYERED SOIL WITH VERTICAL LOADING BY USING FINITE DIFFERENCE METHOD BASED SOFTWARE

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ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "SIMULATION OF PILE LOADING TEST IN A LAYERED SOIL WITH VERTICAL LOADING BY USING FINITE DIFFERENCE METHOD BASED SOFTWARE" and submitted in partial fulfillment of the requirements for the degree of Master's with specialization in GEOTECHNICAL ENGINEERING and has been carried out by **BEREKET GEBRESELASSIE GIRMAY** ID. No PGGeo/006/09, under my supervision Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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We, the undersigned, members of the Board of Examiners of the final open defense by **BEREKET GEBRESELASSIE GIRMAY** have read and evaluated his Thesis entitled “SIMULATION OF PILE LOADING TEST IN A LAYERED SOIL WITH VERTICAL LOADING BY USING FINITE DIFFERENCE METHOD BASED SOFTWARE”, and examined the candidate. This is, therefore, to certify that the thesis has been accepted in the partial fulfillment of the requirements for the degree of Master with a specialization in Geotechnical Engineering.

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DECLARATION

I, Bereket Gebreselassie, declare that this MSc. thesis is my own original work and has not been presented for a degree of masters in any other university, and all sources of materials used for this thesis have been duly acknowledged.

Name ………………………………………………….. Signature …………………

Date of submission …………………………………….

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List of abbreviations and symbols

BEM	Boundary Element Method
C	Cohesion
D − P	Drucker Prager constitutive model
E	Young's Modulus
FDM	Finite Difference Method
FDM	Finite Element Method
FLAC 3D	Fast Lagrangian Analysis of Continua in Three − Dimension
G	Shear modulus of soil
K	bulk modulus
L	Pile length
m. a. s. l	mean average sea level
M − C	Mohr − Coulomb constitutive model
SPT (N)	Standard Penetration Test (Number of Blow)
ψ	Dilatancy Angle
φ	Angle of internal friction
λ	Lame′s constant
ρ	Mass density
γ_b	bulk unit weight
ν	poisson′s ratio
σ	Stress

Abstract

Pile loading tests are usually performed in various projects to determine the ultimate pile capacity. However, the cost of running these tests and the time it takes is one of the difficulties that engineers face in current geotechnical practices. Finite difference method and finite element methods have comparable accuracy. However, finite difference method based tool was used for the analysis due to its simplicity, computational efficiency and simple structure codes. The research presents a numerical simulation of pile loading test using a finite difference program "FLAC 3D". The chosen software is memory and simulation time efficient. It solves almost all kinds of geotechnical problems, but the only downside is that it initially takes some time to get the feel of the software, but once understood, it can solve any problem and it also supports a wide range of material models. The objective of this study is to simulate a pile load test with vertical loading in a layered soil, in order to estimate the load-settlement characteristics and to determine the effect of young's modulus, angle of internal friction, lateral earth pressure coefficient, and the dilation angle on the load-settlement curve. Input parameters of the simulation were collected from Nib, United, and Zemen international bank's new headquarter projects. In the case of piles with incomplete data, the parameters were estimated from site experience data and/or using different equations obtained from a literature. The proposed numerical model has been validated with field data and published results provided by other studies. The validation produced good results with a minor deviation except for nib bank piles. The significant deviation in nib bank piles is due to the generalized soil parameters used in the analysis. The numerical analysis underestimated the ultimate pile capacity. However, Lateral pressure coefficient manipulation yields improved results. Underprediction of load-settlement curves of nib bank piles was due to lower young's modulus values estimated from various equations. The study on one of the piles showed that the base resistance carries the upper hand of the total capacity. The importance of using finer mesh near high-stress gradient zones was examined and it has been found that finer mesh generated based on the developed relation produced a good performance. If the required constitutive model, initial and boundary conditions, and good quality input data are available, the proposed numerical model can be used as an alternative method for the design purpose on projects involving pile foundations.

Keywords: Finite difference method, FLAC 3D, Simulation, Pile loading test

1. Introduction

1.1 Background

When soil immediately beneath the ground surface is not capable of supporting a structure, deep foundations are required to transfer the loads to deeper strata, mainly via skin friction and end bearing. The three basic forms of deep foundations are piles, piers, and caissons. The mechanism of transfer of the load to the soil is essentially the same in all types of deep foundations (Arora, 1997). Pile foundations are much more common than any other type of deep foundation, where the soil conditions are unfavorable (Venkatramaiah, 2006).

Piles are usually placed in service as a group rather than on an individual basis to meet loading demands and ensure stability (Gunaratne, 2006). It is common practice to design and test single piles, even when they are part of a pile group or cluster. Piles are placed so that the capacity of the pile group acting as a unit is equal to the sum of the capacities of the individual piles (Venkatramaiah, 2006).

The maximum settlement of the pile and its ultimate load-bearing capacity is the governing criterion in the design of axially loaded piles. The bearing capacity of isolated piles may be determined from pile loading tests, prevailing Building Codes, sounding tests, dynamic pile-driving formulas, and analytical methods. However, pile load testing can represent reasonable results, but such tests are expensive and time-consuming (Thounaojam & Sultana, 2016).

In recent years, along with the continuous development of computer technology, simulation is also gradually developed and continuously extend to all areas. At present, the simulation method has become a very important tool for geotechnical engineering problems. The numerical simulation method cannot only study the bearing characteristics of piles but also can observe the change of the displacement field of the soil (Xu & Guo, 2018). The advantage of numerical analysis methods lies in their ability to address complex soil formations and the interaction between soil and structures (Zhan et al., 2012).

This study focuses on the detailed evaluation of simulation of pile loading test by using the three-dimensional finite difference method based software (FLAC 3D). First, the available constitutive models representing the soil-pile interaction are discussed; then based on the different constitutive models, the load-displacement curves simulated by FDM are compared with actual load test results. Finally, a parametric study on one of the models is carried out by varying young's modulus, angle of internal friction, lateral pressure coefficient, and angle of dilatancy of the different soil layers.

1.2 Statement of the Problem

Even though there are different methods of determining the ultimate pile capacity, most of them have their drawbacks. For instance, empirical methods can only be used in restricted applications because they are developed for specific pile types, driving equipment, soil types, and soil conditions and are of limited usage (Lawton et al., 1986). The semi-empirical method ignores the initial elastic behavior of soil, which may lead to underestimation of pile head response at small deflections. On the other hand, ultimate soil resistance is not well defined for cohesionless soils in such methods and may lead to overestimation of pile head response, if lower than actual at large deflections (Saman Vazinkhoo, 1996). Dynamic formulas focus only on the kinetic energy of driving, not on the driving system. It also assumes constant soil resistance rather than a velocity-dependent resistance. Yet again, it ignores the length and axial stiffness of the pile (Long et al., 2009). In general, Static and dynamic formulas have limited usefulness because of the lack of load-deflection information. Analytical methods are only applicable to small soil strains and to soils that have a constant elastic modulus with depth (Saman Vazinkhoo, 1996). Additionally, it does not account for soil-pile slippage and\or separation. The major shortcomings of most dynamic pile analyzers are the uncertainties of the actual energy applied by the hammer to the pile and the distribution of soil resistance along the pile. It is also expensive, difficult to maintain and requires experienced personnel (Lawton et al., 1986).

The pile loading test is a direct method of determining the bearing capacity of piles. However, the cost of running these tests and the time it takes is one of the difficulties that engineers faced in current geotechnical practices. Sometimes, pile test results may contain inaccuracies due to inappropriate ground stress changes induced in the setup, improper load application scheme, and other side effects. Unless all these aspects are considered and excluded from the measurement, a reasonable interpretation of the pile test would be difficult. In reality, it is difficult to simultaneously meet all these requirements of the designer. However, with the improvement of computer performance and development of numerical methods, the extent to which tests can satisfy the designer's requirement can be extended by simulating a pile load test in a numerical model and analyzing the results in combination with the field test data (Yi, 2004).

Finite element and finite difference methods are widely employed in numerical methods in geotechnical engineering. In contrast to finite difference method based tools, finite element method based software took comparatively longer time for computation, has a difficulty of modeling large deformations, and uses large quantities of data storage due to the uses of stiffness matrix after each step (Lin et al., 2016; Feizee & Fakarian, 2008). Hence, in this study, the finite difference method based numerical model was used to simulate a pile load test to estimate load-settlement characteristics.

1.3 Objective of the study

1.3.1 General objective

The general objective of this study was to simulate a pile loading test with vertical loading in a layered soil by using finite difference method based software to estimate load-displacement characteristics of the pile.

1.3.2 Specific objective

- To estimate the load-settlement curve for pile loading test using FLAC^{3D}.
- To evaluate the performance of the simulated results by comparing them with the field pile load test results.
- To Study the effect of Young’s modulus, Angle of internal friction, Lateral pressure coefficient, and Dilation angle of the soil parameters on load-settlement behavior.

1.4 Significance of the Research

Pile foundations are used often in soft soil strata to transfer superstructure loads to the underlying firm ground. Pile loading test is the most commonly used type of technique to determine pile capacity and load settlement curve. Numerical simulation of the pile load test can also be carried out to study the bearing capacity of piles and to observe the change of the displacement field of the soil. The importance of numerical analysis lies in their ability to capture soil pile behavior with less cost, time, and memory capacity if supplied with the relevant data. This study simulates a pile load test to understand soil-pile deformation characteristics using FLAC^{3D} that can be used onwards for design purposes if good quality data is available.

1.5 Scope of the study

Vertically loaded bored piles in a layered soil are considered in this study for the simulation of pile load test using finite difference method based software.

With the attempts to achieve, the previously mentioned research objectives, required data is gathered from different project sites having a pile foundation. Three project sites were chosen based on the availability of data. The chosen projects are Nib, United, and Zemen bank new headquarter building projects. Soil investigation data and pile load settlement data were taken from these projects. However, almost all soil investigation reports did not provide the required sufficient information for the simulation. As a result, parameter values are estimated based on various works of literature.

2. Literature Reviews

2.1 Introduction

Pile testing is commonly carried out to assess the geotechnical capacity of piles within a foundation system or to check on the integrity of as-constructed piles. Despite the doubts expressed by many engineers that the settlement of a single pile bears little or no relationship to that of a pile group, pile testing has a useful role to play as a tool in the prediction of foundation settlements (Poulos, 2012). The bearing capacity of pile groups subjected to vertical and lateral load depends on the behavior of a single pile (Svjetlana, 2014).

An accurate interpretation of the pile test would be difficult unless some aspects such as whether the different types of load tests or test set-up may have any side-effects on the test results are clearly understood. Therefore, it is important to simulate the pile loading test in a numerical model and analyze the results in combination with the field test data (Yi, 2004). In this chapter, a review is conducted regarding the pile loading test, with the emphasis on the static pile load test. Numerical methods and some constitutive models will be reviewed. Finally, studies related to pile loading test simulation will be reviewed. Since it is not possible to cover every aspect of pile loading test and numerical methods, only some of the most important issues are briefly discussed. On the other hand, due to the shortage of related studies to the topic limited studies are presented here.

2.2 Pile Loading Test

2.2.1 General

A pile load test is the most acceptable method to determine the load-carrying capacity of a pile. It may be conducted on a driven pile or cast-in-situ pile, on a working pile or a test pile, and on a single pile or a group of piles (Venkatramaiah, 2006). The purpose of pile load testing can be either to prove the adequacy of the pile-soil system for the proposed pile design load (proof load test) or to develop criteria to be used for the design and installation of the pile foundation (design load test). Tests in the first category are generally routine, are carried to twice the proposed working load, and are conducted at the start of the job. Design load tests involve elaborate programs and the piles are usually tested to failure (Fuller & Hoy, 1970).

Several forms of pile load tests have been used in practice. Some methods such as static loading tests and dynamic tests have been routine in geotechnical engineering for many years, while the Osterberg cell test and statnamic test have been developed for less than twenty years (Yi, 2004). A static load test is the most basic test and involves the application of vertical load directly to the pile head. Loading is generally either by discrete increases of the load over a series of intervals of time (Maintained Load test and Quick Load test) or, alternatively, in such a manner that the pile head is pushed downward at a constant rate (Constant Rate Penetration test) (Yi, 2004).

In fact, of various types of pile testing, not all of them can be relied upon. Pile foundations, particularly bored piles, are influenced by the quality and workmanship in construction. Bored piles construction method will vary depending on the type of soil that will make a difference in integrity and bearing capacity (Limas, 2015).

Full-static loading is a widely accepted test method for an authoritative assessment of deep foundations. Testing is typically performed once and the result is considered the definitive answer regarding the pile's load-bearing capacity. However, there are technical and operational reasons that may cause misleading results (Hussien et al., 2005). Consequently, it would be important to use other cost-effective and efficient methods.

Pile load tests are expensive and can be quite time-consuming. For small projects, the cost of pile testing can represent a considerable portion of the overall foundation cost. In many cases, prior experience combined with adequate subsoil data and sound judgment can preclude the need for pile testing, especially if the pile design load is relatively low. (Fuller & Hoy, 1970).

2.2.2 Pile capacity determination

For pile foundation projects, it is usually necessary to confirm capacity and to verify that the behavior of the piles agrees with the assumptions of the design. The most common effort is through a static loading test and, normally, determining the capacity is the primary purpose of the test. The capacity is the total ultimate soil resistance of the pile determined from the load-movement behavior measured in a static pile-loading test. (Fellenius, 2006).

The capacity can, crudely, be defined as the load for which rapid movement occurs under sustained or slight increase of the applied load, the pile plunges (Curve-A). This definition is inadequate, however, because large movements are required for a pile to reach plunging mode and large movements are often governed less by the capacity of the pile-soil system and more by the capacity of the designer. On most occasions, a distinct plunging ultimate load is not obtained in the test (Curve-B) and, therefore, the pile capacity or ultimate load must be determined by some definition based on the load-movement data recorded in the test (Shariatmadari et al., 2007).

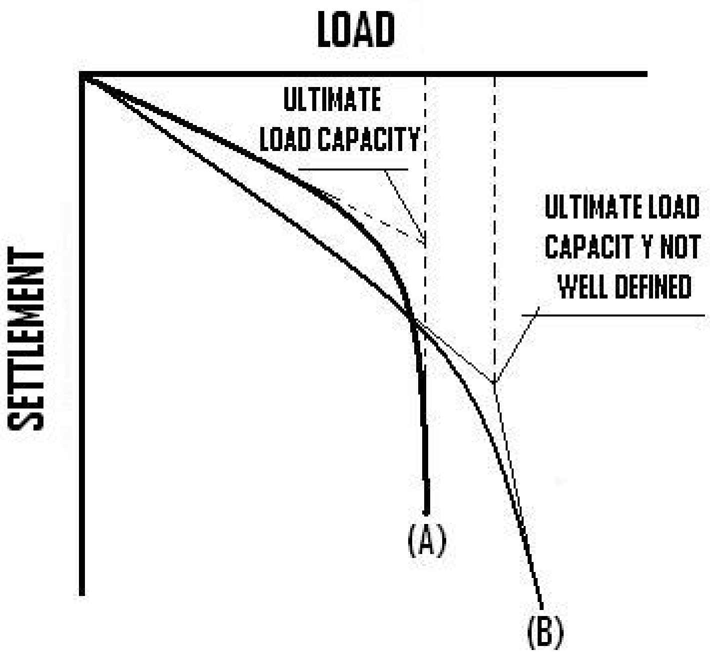

Figure 2.1 (A) Ideal plunging, (B) graph with unclear failure point (Shariatmadari et al., 2007)

An early definition of failure load is the load for which the pile head movement exceeds a certain value, for example, 10% of the diameter of the pile head. This definition does not consider the elastic shortening of the pile, which can be substantially for long piles and negligible for short piles. In reality, a movement limit relates only to the permissible movement allowed by the superstructure to be supported by the pile, and not to the capacity of piles. This definition leads to an irrational acceptance of excessive settlement (Shariatmadari et al., 2007).

It is difficult to make a rational choice of the best criteria to use because the preferred criterion depends heavily on one’s experience and conception of what constitutes the ultimate resistance of a pile. Like shallow foundations in some circumstances, the settlement of the pile head highly affects the design.

To reach the ultimate bearing capacity, large deformation is needed. Because of this reason some criteria are based on a specified settlement. On the other hand, some designers preferred that to achieve an allowable capacity of piles, use the ultimate bearing capacity and a safety factor (Shariatmadari et al., 2007).

Some of the methods of determining the limiting pile load capacity from the load-settlement curve suggested by various researchers and codes are listed below:

i. The load corresponding to the movement which exceeds the elastic compression of the pile by a value of 4 mm plus a factor equal to the diameter of the pile divided by 120 (Davisson, 1972)

ii. The point at which the end tangents of the load-settlement curve meet (Mansur/Kaufmann1956)

iii. Divide each load with its corresponding movement and plot the resulting value against the applied load. The point at which the apparent line intersects with the abscissa (Decourt, 1999)

iv. The point at which the curve manifests the steepest slope i.e., ds/dQ = max (Vesic, 1963)

v. For a total settlement of 10 percent of the pile diameter or 0.1d ((Terzaghi & Peck, 1961)

vi. For piles in compression, it is often difficult to define an ultimate limit state from a load-settlement plot showing a continuous curvature. In these cases, settlement of the pile top equal to 10% of the pile base diameter should be adopted as the "failure" criterion. (European standard; EBCS)

vii. Point of intersection of the load-movement, log-log plot (De Beer and Walays, 199, 1972)

2.3 Numerical Method (Discretized Continuum Approach)

Most of the engineering problems (or even real systems in general) involve complex physical phenomena (Chaskalovic, 2008). To understand such phenomena, engineers need to make some simplifying assumptions, which allow them to formulate a mathematical model (Darve et al., 2004; Wood, 2004). Quite frequently, the model consists of a set of Partial Differential Equations (PDE) for which (in most cases) there is no analytical solution (Rao, 2004; Zhao et al., 2009). In these cases, it is necessary either to simplify the model further or to obtain an approximate solution (Chaskalovic, 2008; Darve et al., 2004).

Unlike the beam-on-foundation method, the discretized continuum approach treats the surrounding soil in three dimensions (Ali et al., 2014). Three numerical techniques are widely used in the analysis of piles, namely the finite element method (Brown et al., 1989; Trochanis et al., 1991; Wang & Sitar, 2004; Noh et al., 2008; Eslami et al., 2012 and Elkady, 2013), the boundary element method (Banerjee & Davies, 1978; Poulos & Davis, 1980 and Basack & Dey, 2012) and the finite difference method (Ng & Zhang, 2001; Klar & Frydman, 2002; Feizee & Fakarian, 2008; Haldar & Babu, 2012; Ghee & Guo, 2014 and Luan et al., 2015).

The finite difference method is the oldest and simplest technique. It requires the knowledge of initial and\ boundary values. The solution is done by time-stepping using small intervals of time and the grid values are updated after each time step.

3-D FEM usually requires a large amount of computer storage and time but 3-D FDM on the other hand is memory and simulation time-efficient with practically acceptable accuracy. FDM is capable of modeling large systems with limited memory, as no stiffness matrix is required (as opposed to FEM, for example). In addition, large deformations do not significantly increase the run-time, as no stiffness matrix updates are required after each load or time increment. (Lin et al., 2016; Feizee & Fakarian, 2008).

In the finite difference method, every derivative in the set of governing equations is replaced directly by an algebraic expression written in terms of the field variables (e.g., stress or displacement) at discrete points in space; these variables are undefined anywhere else. In contrast, the finite element method has a central requirement that the field quantities (stress, displacement) vary throughout each element in a prescribed fashion using specific functions (interpolation functions) Controlled by parameters. The formulation consists of adjusting these parameters to minimize error terms on local or global energy (Itasca consulting group, 2015).

2.4 Previously Used Constitutive Models

Lately, more and more comprehensive constitutive models are being developed to describe the complex behavior of geomaterial under different loading conditions, which in turn leads to difficulties in numerical implementing and identifying model parameters by means of standard material tests (Lin et al., 2016). In the pile-loading test, simulation conducted by Zhan et al., (2012) the soil mass was modeled by soil block elements, and the pile was modeled by pile structure elements with interface elements. The soil was modeled using the Mohr-Coulomb model and the pile was simulated using a linear elastic model.

Elasto-perfectly plastic model with Mohr-Coulomb failure criterion, usually named as Mohr-Coulomb model, is widely used in finite element analysis of geotechnical engineering, due to its simplicity and sufficient accuracy. It is a well-known model, usually used as a first-order approximation of real soil behavior. Due to its simplicity, it is highly popular and gives reasonable results. The model involves five parameters, i.e. Young's modulus, E, Poisson's ratio, ν, cohesion, c, internal friction angle,φ, and dilatancy angle, ψ (Chen & Saleeb, 1982; Yi, 2004).

For reinforced concrete pile, the behavior was assumed to be linear-elastic. The linear elastic model represents Hooke's law of isotropic linear elasticity. The model involves two elastic stiffness parameters, i.e. Young's modulus, E, and Poisson's ratio, ν. The linear elastic model is seldom used to simulate soil behavior. It is primarily used for stiff massive structural systems installed in the soil, such as the test pile. (Yi, 2004)

The Hardening-Soil model is an advanced model developed by Schanz and Vermeer (1998) for simulating the behavior of different types of soil, both soft soils, and stiff soils. The model requires more complicated parameters (i.e. cohesion, c, internal friction angle, φ, dilatancy angle, ψ, power for the stress-level dependency of stiffness, m, secant stiffness in a standard drained triaxial test, E_{50}^{ref}, tangent stiffness for primary oedometer loading, E_{oed}^{ref}, unloading/reloading stiffness, E_{ur}^{ref}, Poisson's ratio for unloading-reloading, ν_{ur}, coefficient of lateral stress in normal consolidation, k_o^{NC} etc..). As stated by Ali & Hai (2011), The Hardening soil model is an elasto plastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. The basic feature of the hardening soil model is the stress dependency of soil stiffness.

Zeleke (2015) utilized plasticity models of the Mohr-Coulomb and Cap model to simulate the pile load test. The Drucker–Prager/cap plasticity model is appropriate to soil behavior because it is capable of considering the effect of stress history, stress path, dilatancy, and the effect of the intermediate principal stress. It consists of three parts: a Drucker–Prager shear failure surface, an elliptical cap, which intersects the mean effective stress axis at the right angle, and a smooth transition region between the shear failure surface and the cap.

According to Ji et al. (2008), a modified Cam-Clay model (Roscoe & Schofield, 1963 and Wood, 1990) was selected to represent the constitutive behavior of the clays and the Mohr-Coulomb plasticity model was used to simulate the constitutive behavior of the sands at the site. Based on the loading conditions in the FLAC^{3D} model simulation, the interaction between the piles and soils including the slippage along with the pile-soil interface in a vertical direction is considered critical and simulated adequately.

2.5 Related Studies

The acceptance of numerical analysis in general and FEM and FDM, in particular, is growing for the simulation and analysis of soil/rock-structure interaction behavior. As a result, various researchers are developing FEM and FDM models to emulate and analyze the actual site conditions. Some of the related studies to the topic in consideration are discussed below:

Ji et al., (2008), presented the findings of a 3-dimensional, large-strain, soil-structure interaction analysis for a piled raft foundation using the computer program FLAC^{3D}. The analysis includes studying the impact of sand fill and de-watering on ground settlements using a 1D coupled flow-mechanical, large strain FLAC^{3D} model, simulating the site preparation process, comparing the analytical predictions with the field measurements. The 3D finite difference model takes into consideration the interactions amongst soils, raft, and piles. Based on the comparison of these values with those measured in the field, the predicted settlements are in good agreement with those measured in the field.

FLAC^{3D} is used to simulate the pile loading and base grouting. The soil and bedrock are modeled with Mohr-Coulomb. The parameters of the constitutive law are established from soil triaxial tests and rock borehole deformation tests. The pile is assumed as an elastic body. The interface element is used to simulate the pile/soil interface and pile/rock interface. The numerical results are compared with the pile load test in terms of the t-z curve, q-z curve, and p-s curve. Finally, the pile base grouting is simulated and the uplift is analyzed. The result from the study was promising and the numerical modeling was successful (Cheng et al., 2006).

Zeleke (2015) attempted to simulate a pile-loading test by using FEM based software, ABAQUS. The pile is assumed linear elastic and for the different soil layers two constitutive models, namely, Mohr-Coulomb and Cap Plasticity Model were used. The results of the FEM analysis were compared with the results of actual pile load tests. Furthermore, a parametric study was also carried out based on soil parameter findings that affect the load-settlement behavior of the pile. In all the cases of the study, the cap model gives better simulation result than the Mohr Coulomb for the considered soil type when compared with the actual load test.

Lin et al., (2016), attempts to assess the piled raft foundation behavior and to broaden the understanding of the complex interaction between the piles, raft, and soil via numerical simulation using 3-D FDM (FLAC^{3D}) software. To achieve these objectives they utilized a pile load test model to calibrate input parameters of piles and performed a parametric study to examine the effect of raft thickness, the number of piles and the loading level on the settlement, the bending moment, and the load-carrying ratio of the raft for a typical Taipei 101 construction project soil profile. The soil layers were modeled using the Mohr-coulomb soil model and the pile was modeled using a pile structure element with an interface element. Simulation results indicate that FLAC^{3D} can capture the deformation behavior of the pile fairly well.

Ghee & Guo (2014), studied FLAC^{3D} analysis regarding the model pile tests subjected to lateral soil movement. The sand strata were modeled with the elasto-plastic Mohr-Coulomb model having a non-associated flow rule. The pile was modeled as an isotropic elastic hollow pile that consisted of cylindrical elements and interface elements that were placed between the soil and the pile. Each interface element was defined by Coulomb's failure criterion with the input parameters of friction angle, normal stiffness and shear stiffness, cohesion, and dilation value of zero and tensile strength of zero. In the event that the soil is moving away from the pile, as is observed in most analyses (at the surface of the soil), the interface elements are attached to the outer perimeter of the pile, not allowing separation between the soil and the pile, a limiting tensile force is imposed on the pile. The predictions show some difficulty in modeling the magnitude and the profile of the measured pile response. However, the ratio of maximum bending moment over shear force induced in each pile was well simulated.

3. Materials and Methods

3.1 Description of the Study Area

3.1.1. Location

Addis Ababa is located in the central highlands of Ethiopia. The city government of Addis Ababa is bounded by 9^{0} $00^{'}$ N and 10^{0} $00^{'}$ N latitudes, 37^{0} $30^{'}$ E, and 39^{0} $00^{'}$ E longitudes. The study covers three projects that are situated at a prime location in Addis Ababa city, traditionally known as the Sengatera area, where multi-story buildings are currently under construction or planned for construction over the next few years. These projects are Zemen Bank Head Quarter Building project, which is located in front of Addis Ababa university school of commerce, United Bank Head Quarter Building project near Biftu building, and Nib international Bank Head Quarter Building project just behind the national theatre.

3.1.2. Climate

The Climate of Addis Ababa is Woina Dega type (Daniel Gemechu, 1974). The Rainfall has a unimodal pattern, one distinct rainy and dry season. The dry season is October through May and the wet one is from June to September being the highest rainfall peak in August. The long-term mean annual rainfall observed at Addis Ababa Observatory is 1254 mm (Berhanu, 2002). The maximum temperature of Addis Ababa ranges between 20^{0} C (in the wet season) to 25^{0} C (in the dry season), while the minimum falls between 7 and 12^{0} C in the year (Elias Assefa, 2012).

3.1.3 Geology

From the aspect of general geology, Addis Ababa city is located within the western margin of the Ethiopian rift and consists of different volcanic rocks, such as Rhyolite and Trachyte at the northern part, Aphanitic basalt at the eastern part, mainly Trachyte in the southern and southwestern part, Rhyolite in the southeastern.

The geology of Addis Ababa area is represented by four volcanic units dominated in the lower part by basaltic lava flows (Addis Ababa basalt), followed by a pyroclastic sequence, mainly formed by ignimbrites (Addis Ababa ignimbrite), followed by central composite volcanoes (central volcanoes unit), and finally small spatter cones lava flows.

Aphanitic Basalt, mostly overlain by ignimbrite layer, covers the north and northwest parts of the city. Ignimbrite and welded tuff mainly cover the central part of the city, such as Haya Hulet, Megenagna, Gurdshola, Kirkos, and Kera areas. In Legahar, Mexico, and Lideta areas, which are in close proximity to the study area, the ignimbrite and welded tuff layers are underlain by weathered basalt. The ignimbrite layer is missing at the Sengatera area where the project sites are located.

Based on the geologic map of Addis Ababa city (Mulugeta HaileMariam et al., 2007), the following volcanic formations are predominantly found in the project and the surrounding area:

- **Quaternary Basalt:** It is the youngest area units in the area and extensively crops out around the project site. It is predominantly composed of olivine phyric vesicular basalt.
- **Repi Basalt:** extensively crops out east of the project area and predominantly constituted by alkaline and olivine Basalt.

Geologically, all project sites are parts of the Addis Ababa basalt. As a result, the area is composed of basalts with a varying degree of weathering.

3.1.4 Topography

United bank project site formation is characterized as a moderately sloping ground terrain from south-west to northeast with an elevation difference of around 4 meters. It is partly with a gentle slope to flat and occasionally undulating due to the accumulation of irregular demolished garbage materials, particularly at the southwestern side. The site's natural ground level (NGL) is generally located at a depth variation of 1.20 to 3.20m below the existing surface. The elevated surface above the NGL is composed of some construction debris left after the demolition of previous buildings and mainly of fill material used for previous constructions. The average thickness of the fill material is about 2.25m.

Zemen bank and Nib bank project sites are characterized by flat topography and the average elevation of Zemen bank project site is 2362 m.a.s.l.

3.2. Soil conditions, stratifications and parameters

3.2.1. General

Input parameters for the numerical simulation of geotechnical problems are established from the detailed soil investigation and material report data. An in-depth and good-quality geotechnical investigation report gives rise to a successful numerical emulation of the actual site conditions. However, most of the soil investigation reports are usually limited to the determination of basic soil properties like shear strength parameters. As a result, additional important parameters should be determined to get sufficient input parameters for the numerical model.

The main criteria for the selection of the study area were the availability of relatively sufficient soil investigation and pile loading test data. Accordingly, three projects located in Addis Ababa are chosen for this study, which are:

- United Bank Head Quarter Building project
- Nib International Bank Head Quarter Building project and
- Zemen Bank Head Quarter Building project

3.2.2. United Bank Head Quarter Building Project

The project site is located in Addis Ababa, Lideta sub-city, specifically at the area traditionally known as Sengatera. Main asphalt roads bound the site on its two sides, Ras Abebe Aregay Street on the southeast and Sengatera Road on the south-west. The recently constructed Biftu Building is located some 10 meters below the surface. The building comprises four basements and thirty-three storey's.

Soil investigation task to determine the stratification and the engineering properties of the soils or rock underlying the site is undertaken by Construction Design Share Company (CDSC).

The geotechnical investigation consists of eight boreholes drilled to a different depth. Five boreholes were drilled at the area of the high-rise part of the proposed building, out of which two boreholes were drilled to 50 m depth and three boreholes were drilled to 45 m depth from the surface. The remaining three boreholes at the low rise area of the building were drilled to a depth of 30 m from the surface.

The project site is entirely covered by highly to moderately weathered basalt, which is overlain by some plastic clay, forming the superficial layer. The weathered basalt is intercalated with over-consolidated clay (paleo soil) and sandy gravel mixed with some sand.

Static ground levels were measured in all the drilled boreholes as well as in the piezometer installed in the selected boreholes for monitoring and observations of the average ground level at the site over a longer period. The static groundwater table is found to be within the range between 6.6 to 7.9 m from the surface. As a result, the submerged unit weight of the soil is used in the analysis of layers located below the water table.

The site is subdivided into five major geotechnical layers and layer three is interbedded in layers four and five. As the building foundation is placed at a much deeper depth from the surface, due to the presence of four basements, 16.3 meters of soil were excavated. Only layers below 16.3 m are considered for the analysis and the summarized soil properties are shown in Table 3.1.

Two pile loading tests were conducted by ANCHOR Foundation Specialist Plc. on working piles constructed as part of the foundation. Both piles are loaded up to 200 % of the working load.

Working Pile 1:

The test pile has been drilled and cast with concrete on 26.04.2016 with a diameter of 600 mm to a depth of 21 m from the working platform level. The reinforcement of the pile consists of 14 numbers of vertical main bars diameter of 20 mm. The pile was covered by helix diameter 10 mm with a spacing of 200 mm over the length of the reinforcement and a 50 mm concrete cover was maintained by concrete spacers attached to the helical stirrup.

Working Pile 2:

The test pile has been drilled and cast with concrete on 09.05.2016 with a diameter of 800mm to a depth of 21 m from the working platform level. The reinforcement of the pile consists of 12 numbers of vertical main bars diameter of 20 mm. The pile was covered by helix diameter 8 mm with a spacing of 150 mm over the length of the reinforcement and a 50mm concrete cover was maintained by concrete spacers attached to the helical stirrup. C30 grade of concrete is used.

The load-settlement curve of the pile are shown in Figure 3.1 and 3.2.

For further reference on loading test data and borehole log sheets, refer to Appendix A and B, respectively.

Table 3.1 Description of soil strata of United Bank-building project, (obtained from the soil investigation report by CDSC)

Layer no.	Description	Depth (m)	Bulk unit weight, γ_b (KN/m^3)	Angle of internal friction(φ)	Cohesion (c)	SPT (N-values)	Poisson's ratio (ν)	Young's modulus, E (MPa)
1	Dense to very dense, brownish to gray sandy gravel with some sand (paleo soil)	16.3-19.5	19	34	-	48	0.3	-
2	Very stiff to hard, reddish brown Silty clay with some sand (paleo soil)	19.5-28.5	17	24	35	>50	0.3	12
3	Dense to very dense, brownish to gray sandy gravel with occasional rock cores	28.5-33	19	36	-	48	0.3	40
4	Very stiff to hard , reddish brown Silty clay with some sand (paleo soil)	33-36	17	24	35	>50	0.3	12
5	Dense to very dense, brownish to gray sandy gravel with occasional rock cores	36-41.8	19	36	-	48	0.3	40
6	Moderately weathered, medium strong fractured and fragmented basaltic rock	41.8-43	20	38	-	>50	0.3	45
7	Very stiff to hard , reddish brown silty clay with some sand (paleo soil)	43-45	17	24	35	>50	0.3	12
8	Moderately weathered, medium strong fractured and fragmented basaltic rock	45-50	20	38	-	>50	0.3	45

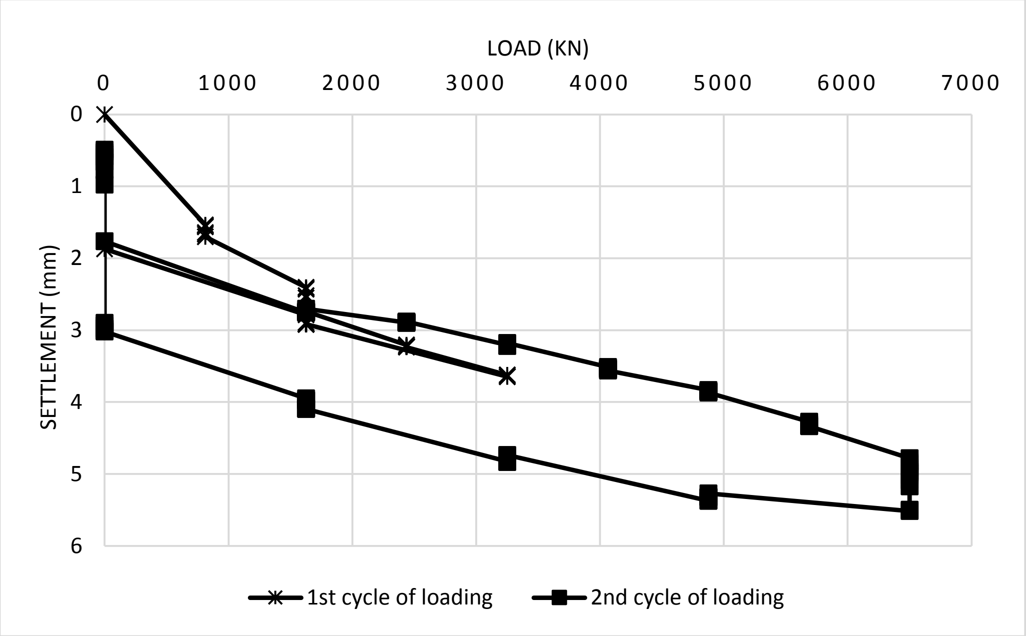

Figure 3.1 Load-settlement graph, United bank test pile 1(from test reports by Anchor Foundation)

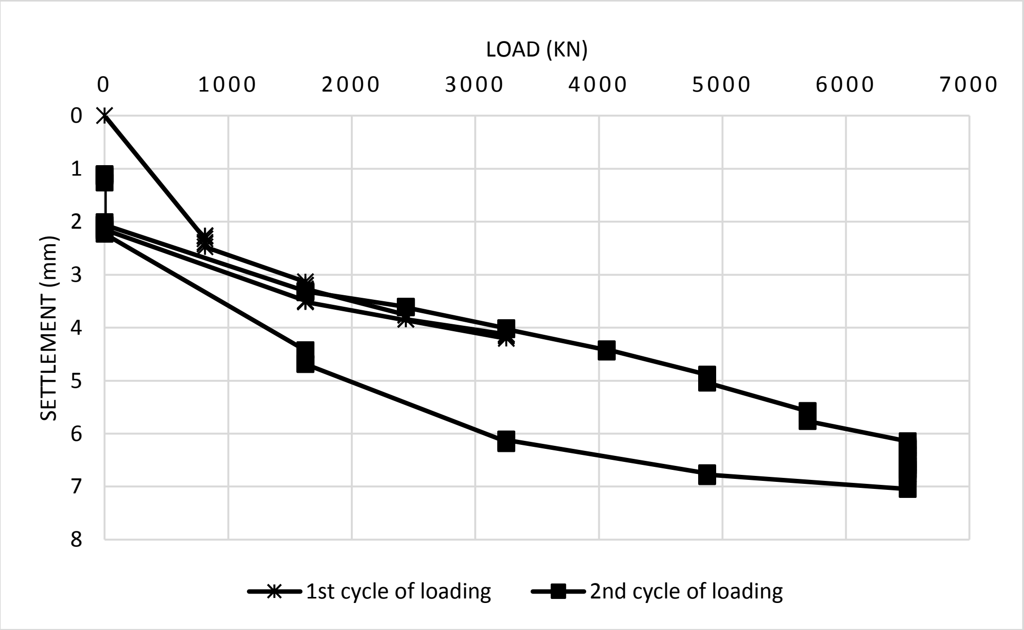

Figure 3.2 Load-settlement graph, United bank test pile 2(from test reports by Anchor Foundation)

3.2.3 Nib Bank Head Quarter Building Project

Nib International Bank Head Quarter project is located in Addis Ababa just in front of Bedlu Building. It is composed of a 4B+G+4 building at the periphery and a 4B+G+30 building at the center. Transport construction design Share Company (TCDSC) had carried out the soil investigation task to determine the engineering properties of the soil and the subsurface geological material.

Four vertical boreholes, designated as BH-1, BH-2, BH-3, and BH-4were drilled. BH-1 and BH-4 were drilled to a depth of 30 meters while BH-2 and BH-3 were drilled to a depth of 40 meters. The soil investigation report of the site stipulates that the geology of the area is composed of basalts with varying degrees of weathering. Stratification of the geotechnical layer is tabulated in Table 3.2

Groundwater level measurement was taken from BH-1, BH-3, and BH-4. It has been difficult to take the measurement in BH-2 because of collapsing. The groundwater level of the site lies in the range between 1.00 - 8.00 m. Since the depth of excavation for the foundation and the basement (16.3 m) is below the groundwater level, bulk unit weight is used for the analysis.

Two pile-loading tests were conducted by ANCHOR Foundation Specialist Plc. on working piles constructed as part of the foundation. Both piles are loaded up to 180 % of the working load.

Working Pile No. 1:

The test pile has been drilled and cast with concrete on 09.07.2015 with a diameter of 800 mm to a depth of 21.00 m from the working platform level. The reinforcement of the pile consists of 12 numbers of vertical main bars diameter of 20 mm. The pile was covered by helix diameter 8 mm with a spacing of 150 mm over the length of the reinforcement and a 50 mm concrete cover was maintained by concrete spacers attached to the helical stirrup. C30 grade of concrete is used.

Working Pile No. 2:

The test pile has been drilled and cast with concrete on 07.09.2015 with a diameter of 800 mm to a depth of 21.295 m from the working platform level. The reinforcement of the pile consists of 12 numbers of vertical main bars diameter of 20 mm. The pile was covered by helix diameter 8mm with a spacing of 150 mm over the length of the reinforcement and a 50 mm concrete cover was maintained by concrete spacers attached to the helical stirrup. C30 grade of concrete is used

The load-settlement curve for the pile load test of the Nib Bank project are shown in Figure 3.3 3.4. For further reference on loading test data and borehole log sheets, refer Appendix A and B, respectively.

Table 3.2 Description of soil strata of Nib Bank-building project, (obtained from the soil investigation report by TCDSC)

Layer no.	Description	Depth (m)	Bulk unit weight, γ_b (KN/m^3)	Angle of internal friction(φ)	Cohesion (C)	SPT (N-values)	Poisons ratio (ν)
1	Grayish highly fractured, rock (some changed to gravel)	16.3-20.6	17	-	-	40	0.3
2	Slightly weathered strong basalt with very weak rock	20.6-25.3	24	-	-	>50	0.3
3	Greyish highly fractured and decomposed weak rock with cavities	25.3-30	18	-	-	>50	0.3
4	Greyish black, slightly to highly weathered basalt decomposed to gravel	30-36.55	22	-	-	43	0.3
5	Dark grey slightly weathered strong to moderately strong basalt	36.55 -40	24	-	-	>50	0.3

The above table shows a general description of different soil layer formation and parametric values, which are retrieved from the soil investigation report. Shear strength values are not given in the above table. Young’s modulus values are not available for this site. Therefore, the values to be used for the numerical simulation will be estimated. The estimation of these parameters will be discussed in the next section.

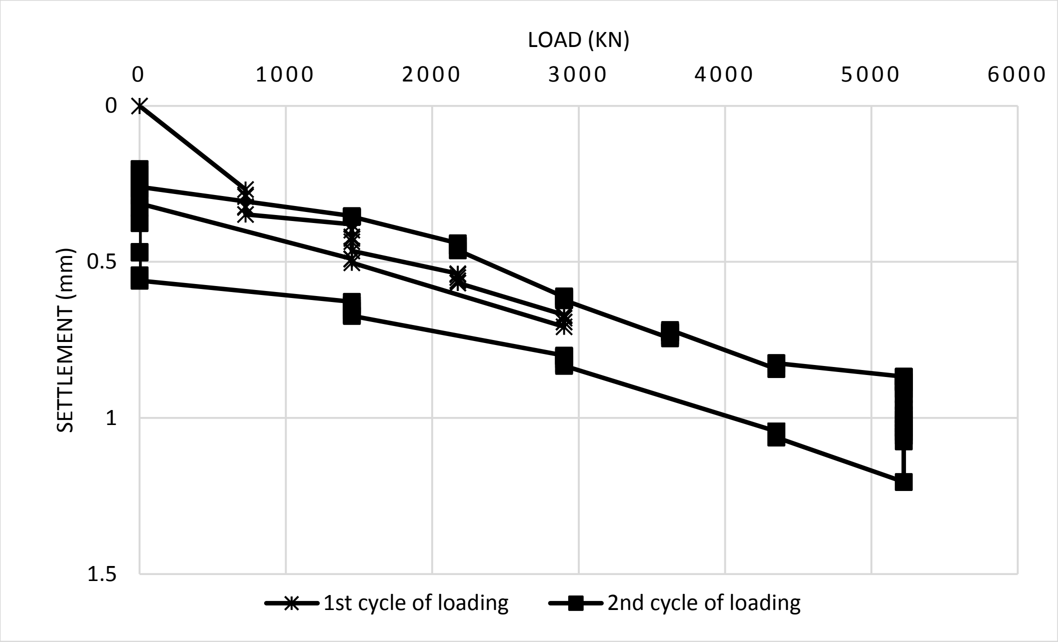

Figure 3.3 Load–settlement graph, Nib bank test pile 1(from test reports by Anchor Foundation)

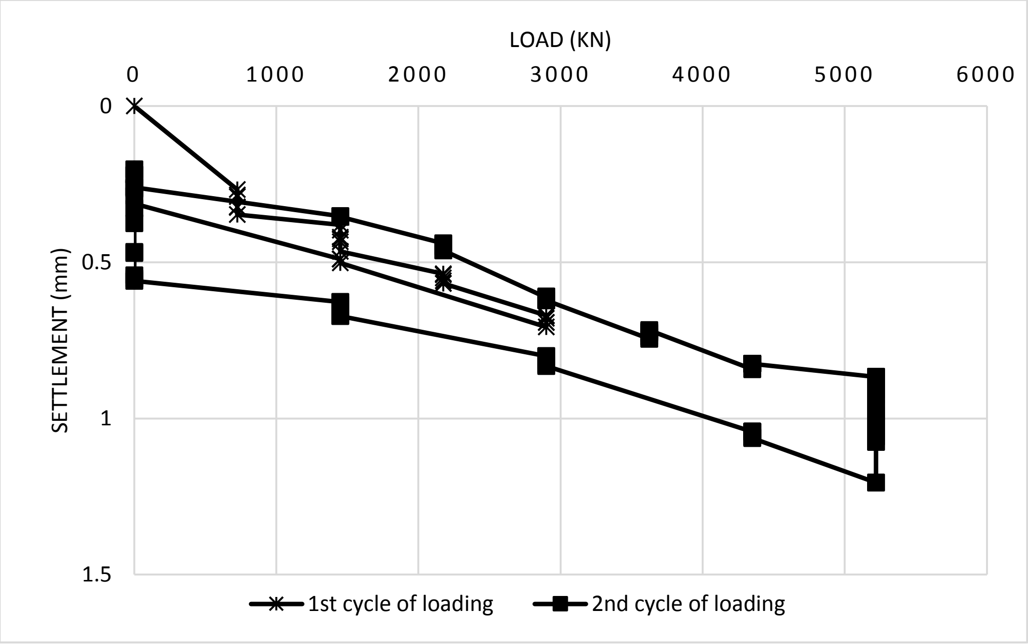

Figure 3.4 Load-settlement graph, Nib Bank test pile 2(from test reports by Anchor Foundation)

3.2.4 Zemen Bank Head Quarter Building Project

The project site is located in central Addis Ababa around Addis Ababa commercial college. The building is composed of four basements and thirty storeys (4B+G+30). The project site is characterized by flat topography with an average elevation of 2362 m.a.s.l.

BEST Consulting Engineers Plc has carried out the geotechnical investigation task and provided a sub-surface investigation report for the site. The geotechnical investigation comprises core drilling, In-situ, and laboratory tests on representative samples to determine the engineering properties of the sub-surface materials.

Four boreholes were sunk to a maximum depth of 50.00 meters below the natural ground level. The sub-surface geology is subdivided into fifteen layers. However, the top 16.5 m is excavated for the basement and foundation. The stratification of geotechnical layers of the site for the numerical analysis is described in Table 3.3.

The groundwater condition in the boreholes was closely observed and monitored. Following the completion of drilling, a 2 mm diameter piezometer was read several times after installation. The observed groundwater level lies in the range between 7.6 – 9.6 m.

One pile loading tests were conducted by ANCHOR Foundation Specialist Plc. on working piles constructed as part of the foundation.

Working Pile No. 1:

The test pile has been drilled and cast with concrete on 2015 with a diameter of 800 mm to a depth of 21.00 m from the working platform level. The reinforcement of the pile consists of 12 numbers of vertical main bars diameter of 20 mm. The pile was covered by helix diameter 8 mm with a spacing of 150 mm over the length of the reinforcement and a 50 mm concrete cover was maintained by concrete spacers attached to the helical stirrup.

The load vs settlement curve for the pile load test of Zemen Bank project is shown in Figure 3.5.

For further reference on loading test data and borehole log sheets, refer to Appendix A and B, respectively.

Table 3.3 Description of soil strata of Zemen Bank-building project, (obtained from the soil investigation report by BEST consulting plc.)

Layer no.	Description	Depth (m)	Bulk unit weight, γ_b (KN/m^3)	Angle of internal friction(φ)	Cohesion (c)	SPT (N-values)	Poisons ratio (ν)
1	Stiff sandy clayey silt	16.5-24.65	19.61	19	25	48	0.3
2	Fine grained, grey to dark grey often yellowish grey, fresh to moderately weathered highly fractured to fragmented basalt	24.65-29.2	18.2	-	-	>50	0.3
3	Stiff to very stiff, red to reddish brown, highly plastic clayey silt	29.2-32.5	17	19	29	32	0.3
4	Very stiff to hard , reddish brown Silty clay with some sand (paleo soil)	32.5-40	18	18	32	50	0.3
5	Stiff to very stiff, yellowish brown sandy clayey silt (decomposed rock)	40-42	19	18	-	>50	0.3
6	Stiff to very stiff, brownish, reddish brown, highly plastic clayey silt	42-44	17	-	-	32	0.3
7	Stiff to very stiff, pinkish brown to gray, clayey Silty sand contains highly plastic clayey silt, basaltic gravel and cobble (decomposed rock)	44-50	16.6	-	-	50	0.3

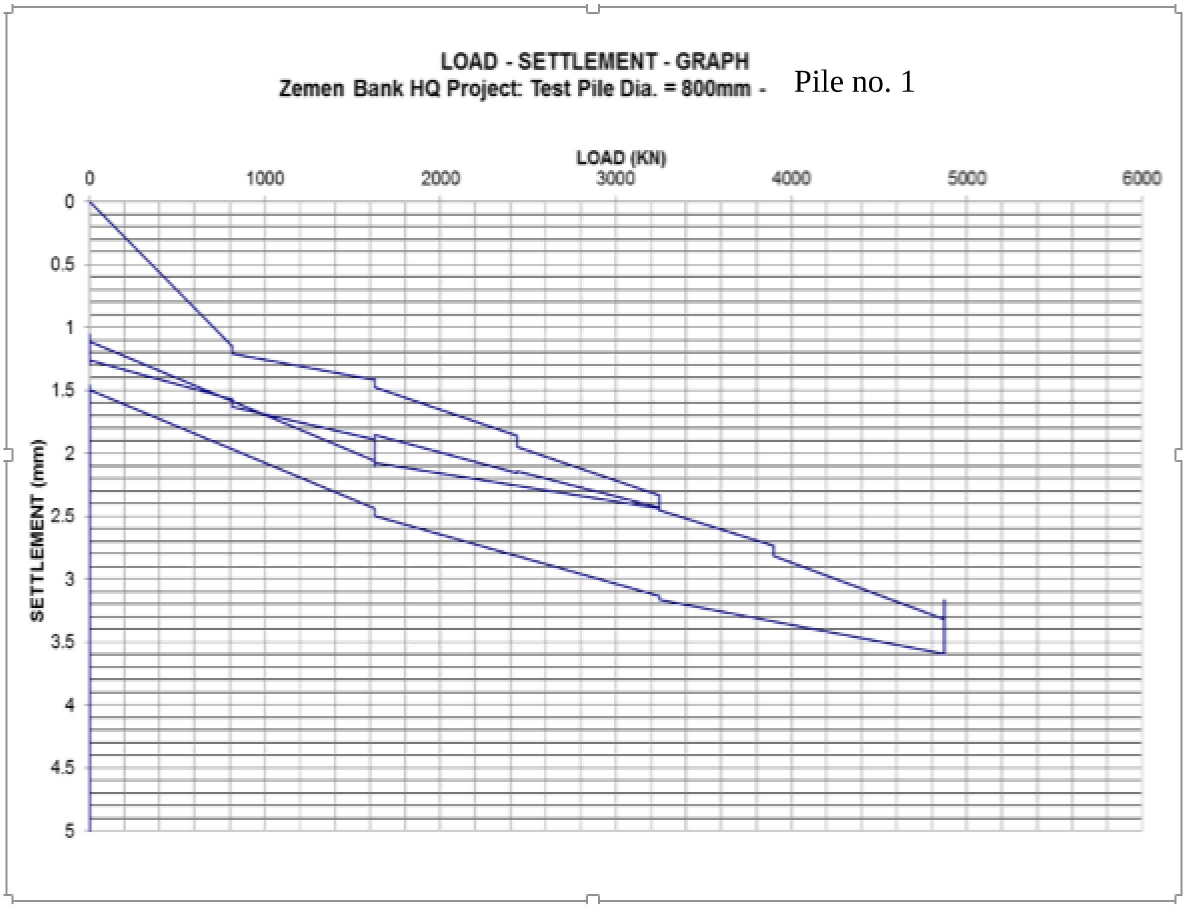

Figure 3.5 Load-settlement graph, Zemen bank test pile 1(from test reports by Anchor Foundation)

Parameters that are necessary for numerical analysis can be determined directly from field and laboratory tests or indirectly from empirical correlations or generally used ranges based on the recommendations of various works of literature.

Young's Modulus, E

Young's modulus is commonly referred to as soil elastic modulus. It is an elastic soil parameter and a measure of soil stiffness. It is defined as the ratio of stress along an axis over the strain along that axis in the range of elastic soil behavior. Using appropriate and reasonable young's modulus values representing the in-situ soil condition has a great deal of importance for a better finite difference analysis.

The determination of Young's modulus for a given soil deposit can be done through laboratory and in-situ testing. Young's modulus can be determined from unconfined compression tests, triaxial compression tests, and in-situ tests. Triaxial tests tend to produce more usable values of E than an unconfined compression test which gives conservative values. The in-situ tests of SPT and CPT tend to use empirical correlations to obtain E.

For the United Bank project, all values of E except for the first layer are provided in the geotechnical investigation report. However, for Nib and Zemen Bank projects E values are not incorporated in the geotechnical investigation report. For this study, E value is obtained from experience data of similar site conditions and using the equations given below.

i. Empirical equations to determine E values according to Bowles(1996)

- For sand (Normally consolidated)

$$\mathrm{E} = 500(\mathrm{N} + 15) \qquad (3.1)$$

$$\mathrm{E} = 7000(\mathrm{N})^{1/2}$$

$$\mathrm{E} = 6000\mathrm{N}$$

- For gravelly sand

$$\mathrm{E} = 600(\mathrm{N} + 6) \quad \text{For N<15} \qquad (3.2)$$

$$\mathrm{E} = 600(\mathrm{N} + 6) + 2000 \quad \text{For N>15}$$

- For clayey sand

$$\mathrm{E} = 320(\mathrm{N} + 15) \qquad (3.3)$$

- For silts, sandy silt, or clayey silt

$$\mathrm{E} = 300(\mathrm{N} + 6) \qquad (3.4)$$

Where

$$\mathrm{N} = \mathrm{N}_{55} = \frac{70}{55} * (\mathrm{N}_{70})$$

ii. Determination of E values according to Mezenbach (1961)

$$\mathrm{E} = \mathrm{c}_1 + \mathrm{c}_2 * \mathrm{N}_{\mathrm{cor}}$$

$$\mathrm{N}_{\mathrm{cor}} = \mathrm{c}_{\mathrm{n}} * \mathrm{N}$$

Where

$$c_n = \frac{2}{1 + 0.01 * \sigma_v} \tag{3.5}$$

C_n = depth

Table 3.4 Constants to be used for the computation of E using Mezenbach equation

Soil type	$c_1(\text{kg}/\text{cm}^2)$	$c_2(\text{kg}/\text{cm}^2)$
Fine sand (above Gwt)	52	3.3
Fine sand (below Gwt)	71	4.9
Sand (medium)	39	4.5
Coarse sand	38	10.5
Sand & gravel	43	11.8
Silty sand	24	53
Silt	12	5.8

Stiffness information that is retrieved from the respective geotechnical investigation report gives a single value for each layer. Similarly, the values computed from various equations are assigned in the respective layer. However, in real soils, the stiffness values significantly depend on the stress level and increase with depth. Hence, a linear variation of Young's modulus in combination with the Duncan-Chang constitutive model where the soil tangent modulus changes according to minor principal stress is employed in this paper. The expression is given by,

$$E = \text{KPa}\left(\frac{\sigma_3}{\text{Pa}}\right)^n \tag{3.6}$$

Where, σ_3- Minor principal stress

n - Stress influence exponent (0.38)

Pa - Atmospheric pressure (101325 Pa), and K – Modulus number (704)

Deformation within the elastic range is described by two elasticity constants: bulk modulus (K) and shear modulus (G) and their relationship with young's modulus (E) and poison's ratio (ν) can be described as,

$$K = \frac{E}{3(1 - \nu)} \tag{3.7}$$

$$G = \frac{E}{2(1 + \nu)} \tag{3.8}$$

Tables 3.5, 3.6, and 3.7, shows stiffness modulus values that are taken directly from the respective geotechnical investigation report data and computed using the previously mentioned equations.

Table 3.5 Stiffness modulus values of the soil for United bank building project

Depth (m) mid-layer	Description	N60 (SPT-N values)	N55 (SPT-N values)	Poisons ratio (υ)	Young's modulus, E (MPa)	Bulk modulus, K (MPa)	Shear modulus, G (MPa)
17.9	Dense sandy gravel	36	40	0.3	30(ii)	14.28	11.54
24	Very stiff to hard , Silty clay with some sand	32	35	0.3	12	5.71	4.62
29.25	Dense sandy gravel with occasional rock cores	36	40	0.3	35	16.67	13.46
34.5	Very stiff Silty clay with some sand	32	35	0.3	12	5.71	4.62
38.9	Dense sandy gravel with occasional rock cores	36	40	0.3	35	16.67	13.46
42.4	Moderately weathered, fractured and fragmented basaltic rock	>50	>50	0.3	45	21.43	17.31
44	Very stiff Silty clay with some sand	>50	>50	0.3	12	5.71	4.62
48	Moderately weathered, fractured and fragmented basaltic rock	38	42	0.3	45	21.43	17.3

(ii) values are computed from the above equation based on the soil type

Table 3.6 Stiffness modulus values of the soil for Nib bank building project bank project

Depth (m) mid-layer	Description	N(SPT)	Poison's	Young's modulus E (MPa)	Bulk modulus K (MPa)	Shear modulus, G (MPa)
18.45	Highly fractured rock some changed to gravel	40	0.3	30.6(ii)	14.57	11.77
22.95	Slightly weathered strong basalt with very weak rock	>50	0.3	100*	47.62	38.46
27.65	Highly fractured and decomposed weak rock with cavities	>50	0.3	30*	14.28	11.54
33.3	Slightly to highly weathered basalt with highly fractured rock	43	0.3	50*	23.81	19.23
38.3	Slightly weathered strong to moderately basalt	>50	0.3	150*	71.43	57.69

Where, (*) values are estimated based on experience data (in reference to the united bank project) and (ii) computed based on the above equation based on the soil type

Table 3.7 Stiffness modulus values of the soil for Zemen bank building project

Depth (m) mid-layer	Description	N	N70	N55	Young's modulus, E (MPa)	Bulks modulus, K (MPa)	Shear modulus, G (MPa)
20.58	Stiff sandy clayey Silt	48	23	30	10.8(i)	5.14	4.15
26.93	Fine grained to highly fractured to fragmented basalt	50	22	28	50*	23.81	19.23
30.85	Highly plastic clayey silt	32	14	18	7.2(i)	3.43	2.77
36.25	Stiff Silty clay with some sand	48	18	23	8.7(i)	4.14	3.35
41	Stiff sandy clayey silt	50	19	25	9.3(i)	4.43	3.59
43	Highly plastic clayey silt	32	14	18	50*	23.81	19.23
47	Clayey silty sand and basaltic gravel	50	18	23	8.7(i)	4.14	3.35

Where, (i) values are estimated from the above equation based on the soil type and (*) values are estimated from experience data (in reference to united bank project) of the site.

Shear strength parameters

Soil strength is the resistance to mass deformation developed from a combination of particle rolling, sliding, and crushing and is reduced by any pore pressure that exists or develops during particle movement. This resistance to deformation is the shear strength of the soil as opposed to the compressive or tensile strength of other engineering materials. The shear strength is measured in terms of two soil parameters: inter-particle attraction or cohesion c, and resistance to interparticle slip called the angle of internal friction φ (Bowles, 1996).

Shear strength parameters are determined directly from direct shear test results. However, in conditions where it is difficult to take samples as in deeper soil layers, Bowles (1996) suggested an empirical correlation for sandy and granular soil, which is developed by Shioi and Fukui (1982) Japanese railway standard.

$$\varphi = 0.36N_{70} + 27, \text{ For Buildings} \tag{3.9}$$

For this study, shear strength parameters are directly taken from the respective geotechnical investigation report. However, for granular and sandy soils where the angle of internal friction is not specified in the geotechnical report, the computation is made based on equation (3.9).

Table 3.8 Angle of internal friction of the soil for United bank building project

Depth (m)	Description	N	φ	C (kPa)
17.9	Dense sandy gravel	48	34	30
24	Very stiff to hard, Silty clay with some sand	50	24	35
29.25	Dense sandy gravel with occasional rock cores	48	36	30
34.5	Very stiff Silty clay with some sand	50	24	35
38.9	Dense sandy gravel with occasional rock cores	48	36	30
42.4	Moderately weathered, fractured, and fragmented basaltic rock	50	38	30
44	Very stiff Silty clay with some sand	50	24	35
48	Moderately weathered, fractured, and fragmented basaltic rock	50	38	30

Where all values in the above table are directly taken from the soil investigation data

Table 3.9 Angle of internal friction of the soil for Nib bank building project

Depth (m)	Description	N_{70}	φ	C (kPa)
18.45	Highly fractured rock some converted to gravel	40	41.4(Eq. 3.9)	-
22.95	Slightly weathered strong basalt with very weak rock	50	43*	100*
27.65	Highly fractured and decomposed weak rock with cavities	50	42*	-
33.3	Slightly to highly weathered basalt decomposed to gravel	43	42.5(Eq. 3.9)	50 *
38.3	slightly weathered Strong to moderately strong basalt	50	45*	100*

(*) values are assumed from experience data and for layer 1 and layer 3 cohesion values are assumed to be zero due to the soil/rock type.

Table 3.10 Angle of internal friction of the soil for Zemen bank building project

Depth (m)	Description	N_{70}	φ	C (KPa)
20.58	Stiff sandy clayey Silty	48	19	25
26.93	Fine-grained to highly fractured to basalt	50	41*	-
30.85	Highly plastic clayey silt	32	19	29
36.25	Stiff Silty clay with some sand	48	18	32
41	Stiff sandy clayey silt	50	18	26
43	Highly plastic clayey silt	32	19	29
47	Clayey silty sand and basaltic gravel	50	22*	10

Poisson's ratio, ν

Poisson's ratio is defined as the ratio of lateral expansion to axial compression strains of the body for a uniaxial stress state (Bowles, 1996; Hudson & Harisson, 1997). For an elastic material, the value varies from 0 to 0.5. Since soil is not purely elastic and a value outside the elastic range of 0 to 0.5 is also occasionally encountered (Arora, 1997). For most rocks, the

Poisson's ratio is slightly less than 0.3 (Steve Hencher, 2012). It is difficult to ascertain the exact value of Poisson's ratio. Fortunately, the effect of Poisson's ratio on the computed stresses is not significant and an approximate value can be used without much error (Arora, 1997). For this study, a typical value of 0.3 and 0.15 had been used for the soil/rock and the pile, respectively.

Coefficient of lateral pressure at rest (k_0)

The ratio of horizontal stress to vertical stress is known as the coefficient of lateral stress. In natural deposits, generally, there is no lateral strain and the lateral stress coefficient for this case is termed as the coefficient of lateral pressure at rest, k_0 (Arora, 1997). In a uniform layer of soil/rock with a free surface, the vertical stresses are computed using (Arora, 1997; Hudson & Harrison, 1997).

$$\sigma_\mathrm{v} = \rho * \mathrm{g} * \mathrm{z} \tag{3.10}$$

Where, $\sigma_v =$ vertical stress

$\mathrm{g} =$ Gravitational acceleration,

$\rho =$ Mass density of the material, and

$\mathrm{z} =$ Depth below the surface.

The in-situ horizontal stresses are more difficult to estimate. However, there is a common – but erroneous – belief that there is some “natural” ratio between horizontal and vertical stresses which is given by,

$$\mathrm{k}_0 = {}^{\upsilon}/_{1-\upsilon} \tag{3.11}$$

Where υ is Poisson’s ratio

Additionally, Brooks and Ireland (1965) recommended Jacky’s equation for cohesionless soil and their own equation, given below for cohesive soils, respectively.

$$\mathrm{k}_0 = 1 - \sin\varphi \tag{3.12}$$

$$\mathrm{k}_0 = 0.95 - \sin\varphi \tag{3.13}$$

Where, The **v**alues of the at-rest pressure coefficient are tabulated in Table 3.11, 3.12**,** and 3.13.

Table 3.11 Initial stress coefficient values of the soil for United bank building project

Depth (m)	Description	Coefficient earth pressure at rest
17.9	Dense sandy gravel	0.44(Eq. 3.12)
24	Stiff Silty clay with some sand	0.54(Eq. 3.13)
29.25	Dense sandy gravel with occasional rock cores	0.41(Eq. 3.12)
34.5	Stiff Silty clay with some sand	0.54(Eq. 3.13)
38.9	Dense sandy gravel with occasional rock cores	0.41(Eq. 3.11)
42.4	Moderately weathered, fractured, and fragmented basaltic rock	0.43(Eq. 3.11)
44	Stiff Silty clay with some sand	0.54(Eq. 3.13)
48	Moderately weathered, fractured, and fragmented basaltic rock	0.43(Eq. 3.11)

Table 3.12 Initial stress coefficient values of the soil for Nib bank building project

Depth (m)	Description	Coefficient earth pressure at rest
18.45	Highly fractured rock with some converted to gravel	0.43
22.95	Slightly weathered strong basalt with very weak rock	0.43
27.65	Highly fractured and decomposed weak rock with cavities	0.43
33.3	Slightly to highly weathered basalt decomposed to gravel	0.43
38.3	Slightly weathered strong to moderately strong basalt	0.43

*All values in the above table were computed from Eq. 3.13.

Table 3.13 Initial stress coefficient values of the soil for Zemen bank building project

Depth (m)	Description	Coefficient earth pressure at rest
20.58	Stiff sandy clayey Silt	0.43
26.93	Fine-grained highly fractured to fragmented basalt	0.43
30.85	Highly plastic clayey silt	0.43
36.25	Stiff Silty clay with some sand	0.43
41	Stiff sandy clayey silt	0.43
43	Highly plastic clayey silt	0.43
47	Clayey silty sand and basaltic gravel	0.43

*Where, all values in the above table were computed from Eq. 3.13.

3.3 Theory and Implementation of the Proposed Constitutive Model

3.3.1 The State of Stress at a Point within a Soil Mass

A major problem in geotechnical analysis is the estimation of the state of stress at a point at a particular depth in a soil mass. A load acting on a soil mass, whether internal, due to its self-weight, or external, due to a load applied at the boundary, creates stresses within the soil. If we consider an elemental cube of soil at the point considered then a solution by elastic theory is possible. Each plane of the cube is subjected to stress, σ, acting normal to the plane, together with shear stress acting parallel to the plane.

To describe the stresses acting on the six sides of a cubic element three symbols $\sigma_x, \sigma_y, \sigma_z$ are necessary for normal stresses and six symbols $\sigma_{xy}, \sigma_{yx}, \sigma_{xz}, \sigma_{zx}, \sigma_{yz}, \sigma_{zy}$ for shearing stresses. By a simple consideration of the moment equilibrium of the element of the number of symbols for shearing stresses can be reduced to three (i.e.$\sigma_{xy} = \sigma_{yx}, \sigma_{xz} = \sigma_{zx}, \sigma_{yz} = \sigma_{zy}$). There are therefore a total of six stress components acting on the cube (Figure 3.6). Once the values of these components are determined then they can be compounded to give the magnitudes and directions of the principal stresses acting at the point considered (Timoshenko and Goodier, 1951). The nine stress Components of stress σ_{ij} can be organized into the stress matrix:

$$\boldsymbol{\sigma} = \begin{bmatrix} \sigma_{xx} & \sigma_{xy} & \sigma_{xz} \\ \sigma_{yx} & \sigma_{yy} & \sigma_{yz} \\ \sigma_{zx} & \sigma_{zy} & \sigma_{zz} \end{bmatrix} \tag{3.14}$$

This arrangement of stresses is also known as stress tensor. The two subscript letters indicate the direction of the normal to the plane under consideration and the direction of the component of the stress, respectively.

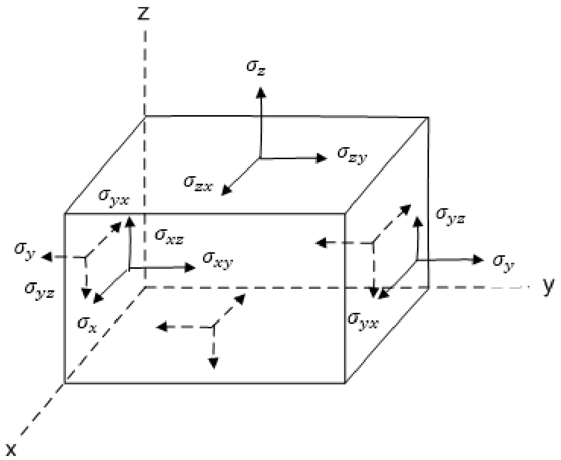

Figure 3.6 Stress in three-dimensional space

3.3.2 Constitutive Models

The two fundamental principles used to derive the governing differential equations of geotechnical problems are equilibrium and compatibility equations. Equilibrium equations are derived based on the concept that in a deformable solid, the force at each point must be balanced. For a static case, the summation of forces in an infinitesimal element is required to be zero (Sadd, 2009). While for a dynamic problem, the resultant force must equal the mass times the elements acceleration. To satisfy the compatibility equation deformed components must fit together (Barber, 2002). The easiest way to satisfy the equation of compatibility is to express all the strains in terms of displacement.

Compatibility and equilibrium conditions produce nine equations and twelve unknowns (i.e. indeterminate). In this case, the provision of constitutive relations turns the equation to determinate. Which is the description of material behavior and it usually takes a relationship between stresses and strains. Therefore, it provides a link between equilibrium and compatibility (Sadd, 2009). Moreover, further simplification is necessary by considering the boundary conditions of specific problems.

The detailed aspect of the proposed constitutive models is discussed in the next section based on the FLAC 3D user's guide manual.

3.3.3 Incremental Formulation

All constitutive models in FLAC 3D share the same incremental numerical algorithm. Given the stress state at time t, and the total strain increment for a time step, Δt, the purpose is to determine the corresponding stress increment and the new stress state at time t + Δt. When plastic deformations are involved, only the elastic part of the strain increment will contribute to the stress increment. In this case, a correction must be made to the elastic stress increment as computed from the total strain increment to obtain the actual stress state for the new time step.

All models in FLAC^{3D} operate on effective stresses only; pore pressures are used to convert total stresses to effective stresses before the constitutive model is called. The reverse process occurs after the model calculations are complete.

3.3.3.1 Incremental equation of the theory of plastic flow

The stresses at time t + Δt are computed as "new stress values," indicated by a superscript 'N'. $[\underline{\sigma}]$ is defined as a general stress vector of dimension n with components $\underline{\sigma}_{\mathrm{i}}, \quad \mathrm{i} = 1, \mathrm{n}$ and $\Delta[\underline{\epsilon}]$ as a generalized strain increment vector with components $\Delta\epsilon_{\mathrm{i}}, \mathrm{i} = 1, \mathrm{n}$ the components of the generalized stress and strain-increment vectors may consist of the six components of the stress and strain-increment tensors or other appropriately defined combinations of variables, giving a measure of stress and strain increments in specific constitutive model contexts.

As a notation convention in this section, the subscript n refers to the range of generalized components from $i = 1$ to $i = n$ (e.g., $f(\underline{\sigma_n})$ is used to represent $f(\underline{\sigma_1}, \underline{\sigma_2}, \underline{\sigma_n})$).

The description of plastic flow in FLAC^{3D} rests on several relations:

- The failure criterion

$$f(\underline{\sigma}_n) = 0 \qquad (3.15)$$

Where f, the yield function is a known function that specifies the limiting stress combination for which plastic flow takes place. (This function is represented by a surface in the generalized stress space, and all stress points below the surface are characterized by elastic behavior)

- the decomposition of strain increments into the sum of elastic and plastic parts

$$\Delta\underline{\epsilon}_i = \Delta\underline{\epsilon}_i^e + \underline{\epsilon}_i^p \qquad (3.16)$$

- the elastic relations between elastic strain increments and stress increments

$$\Delta\underline{\sigma}_i = S_i\left(\Delta\underline{\epsilon}_n^e\right), \quad i = 1, n \qquad (3.17)$$

Where S_i is a linear function of the elastic strain increments, $\Delta\underline{\epsilon}_n^e$

- the flow rule specifying the direction of the plastic strain increment vector as that normal to the potential surface $g\left(\underline{\sigma}_n\right) = \text{constant}$

$$\Delta\underline{\epsilon}_i^p = \lambda \frac{\partial g}{\partial \underline{\sigma}_i} \qquad (3.18)$$

Where λ is constant. (The flow rule is said to be associated if g ≡ f, and non-associated Otherwise.)

3.3.3.2 Implementation

In FLAC^{3D}, an elastic guess $\underline{\sigma}_i^I, i = 1, n$ for the stress state at time t + Δt is first evaluated by adding to the stress components at a time, t, increments computed from the total-strain increment for the step, using an incremental elastic stress-strain law (Eq. (3.19)). If the elastic guess violates the yield function, Eq. (3.21) is used to place the new stress exactly on the yield curve. Otherwise, the elastic guess gives the new stress state at time t + Δt.

If the stress point $\underline{\sigma}_i^I, i = 1, n$ is located above the yield surface in the generalized stress space, the coefficient λ in Eq. (3.21) is given by Eq. (3.20), provided the yield function is a linear function of the generalized stress vector components. The equation Eq. (3.21) is still valid, but λ is set to zero in case $\underline{\sigma}_i^I, i = 1, n$ is located below the yield surface (elastic loading or unloading).

$$\underline{\sigma_i^I} = \underline{\sigma_i} + S_i(\Delta\underline{\epsilon_n}) \tag{3.19}$$

$$\lambda = \frac{f\left(\underline{\sigma_n^I}\right)}{f\left(S_n\left(\frac{\partial g}{\partial \underline{\sigma_n}}\right)\right) - f\left(\underline{0}_n\right)} \tag{3.20}$$

$$\underline{\sigma_i^N} = \underline{\sigma_i^I} - \lambda S_i\left(\frac{\partial g}{\partial \underline{\sigma_n}}\right) \tag{3.21}$$

where, $S_i\left(\frac{\partial g}{\partial \underline{\sigma_n}}\right)$ – is the stress increment obtained from incremental elastic law,

$\frac{\partial g}{\partial \underline{\sigma_n}}$ – is substituted for $\Delta\underline{\epsilon_i}, \quad i = 1, n$

$f\left(\underline{0}_n\right)$ – is the constant term of the function

3.3.4 Elastic Model

The models in this group are characterized by reversible deformations upon unloading; the stress-strain laws are linear and path-independent. The elastic models include both isotropic and anisotropic elastic models. The isotropic elastic model provides the simplest description of the mechanical behavior of a material. It fits the homogeneous, isotropic, and continuous material that only represents a linear stress-strain relationship. Hence, this paper employs the isotropic elastic model for the pile.

3.3.4.1 Elastic, Isotropic Model

In this elastic, isotropic model, strain increments generate stress increments according to the linear and reversible law of Hooke:

$$\Delta\sigma_{ij} = 2G\Delta\epsilon_{ij} + \alpha_2\Delta\epsilon_{kk}\delta_{ij} \tag{3.22}$$

Where, δ_{ij} is the Kronecker delta (symmetric) symbol, and α_2 is a material constant related to the bulk modulus, K, and shear modulus, G, as

$$\alpha_2 = K - \tfrac{2}{3}G \tag{3.23}$$

New stress values are then obtained from the relation

$$\sigma_{ij}^N = \sigma_{ij} + \Delta\sigma_{ij} \tag{3.24}$$

The required parameters for this model are bulk modulus (K), shear modulus (G), and mass density (ρ).

Table 3.14 Parameters for pile constitutive model

Material	Constitutive model	Young's modulus, E (GPa)	Poisson's ratio, ν	Bulk modulus, K (GPa)	Shear modulus, G (GPa)	$\rho(kg/m^3)$
concrete	Linear elastic	25	0.15	9.804	10.87	2500

3.3.5 Plastic Model

The main difference between elastic response and plastic response is that plastic flow will be irreversible. The plastic flow formulation in FLAC is based on the assumption that the total strain increment is the sum of elastic and plastic strains. The elastic strain increment is governed by elastic relations and stress increment.

The characteristics of plastic groups are best described by their flow rule, hardening/softening rule, and yield function. The flow rule defines the relationship between the failure envelope and the directions of the plastic strain increment vector. It is used to describe the deformation behavior beyond the yield point. An associated flow rule occurs when the yield function and the plastic potential function coincide, where the plastic potential function is orthogonal to all of the plastic strain increment vectors. For perfectly plastic material, the normality condition is achieved when the plastic strain increment vector is normal to the yield surface. The yield function determines the stress condition for which plastic flow takes place. An incremental elastic or plastic behavior is determined by the stress condition below or on the yield surfaces in a generalized stress space, respectively. The hardening/softening rule gives the magnitude of the plastic strain increments.

For this study, two elasto-plastic constitutive models (i.e. Mohr-Coulomb and Drucker Prager) had been chosen for the simulation. The data retrieved from various projects are only limited to some common parameters, as a result, the first-order approximation of real soil behavior is adopted. Complex and advanced constitutive models are subjected to higher uncertainties due to the difficulty of establishing the required complicated parameters. Besides, such models are inefficient and require increased time and memory for computation.

In general, these models are widely used despite the availability of advanced models vowing to simplicity and easily determinable model parameters. In the succeeding sections the sign convention is based on FLAC (i.e. positive stresses and strains indicate tension, and negative stress and strains indicate compression).

3.3.5.1 Drucker-Prager Model

It is the most efficient plastic model than the other plasticity models that require increased memory and additional time for computation. It uses a simple failure criterion in which the shear yield stress is a function of isotropic stress. It is provided mainly to allow comparison of FLAC^{3D} to other numerical programs that have the Drucker-Prager model but not the Mohr-Coulomb model. However, it can be applied to soft clay with low friction.

The failure envelope for this model involves a Drucker-Prager criterion with tension cutoff. The position of a stress point on this envelope is controlled by a non-associated flow rule for shear failure and an associated rule for tension failure.

Generalized Stress and Strain Components

The generalized stress vector $[\underline{\sigma}]$ involved in the definition of the Drucker-Prager model has two components (n=2): the tangential stress, τ, and mean normal stress, σ, defined as

$$\tau = \sqrt{\frac{1}{2} S_{ij} S_{ij}} \tag{3.25}$$

$$\sigma = \frac{\sigma_{kk}}{3} \tag{3.26}$$

Where S_{ij} is the deviatoric stress tensor and σ_{kk} refers to the normal-stress tensor.

The components of the associated generalized strain increment vector $\Delta[\underline{\epsilon}]$ are the shear strain increment, Δγ and volumetric strain increment $\Delta\epsilon$ introduced as,

$$\Delta\gamma = \sqrt{2\Delta e_{ij}\Delta e_{ij}} \tag{3.27}$$

$$\Delta\epsilon = \Delta\epsilon_{kk} \tag{3.28}$$

Where, $\Delta[e]$ is the incremental deviatoric strain tensor

Composite Failure Criterion and Flow Rule

The failure criterion used for this FLAC^{3D} model is a composite Drucker-Prager criterion with tension cutoff as sketched in the (τ, σ) representation of Figure 3.7. The failure envelope $f(\tau, \sigma) = 0$ is defined, from point A to B on the figure, by the Drucker-Prager failure criterion $f^s = 0$, with

$$f^s = \tau + q_\phi \sigma - K_\phi \tag{3.29}$$

And, from B to C, by the tension failure criterion $f^t = 0$, with

$$f^t = \sigma - \sigma^t \tag{3.30}$$

Where $q_{\phi\,,}K_\phi$ and σ^t are positive material constants, and σ^t is the tensile strength for the Drucker Prager model. Note that, for a material whose property q_ϕ is not equal to zero, the maximum value of the tensile strength is given by,

$$\sigma^t_{max} = \frac{K_\phi}{q_\phi} \tag{3.31}$$

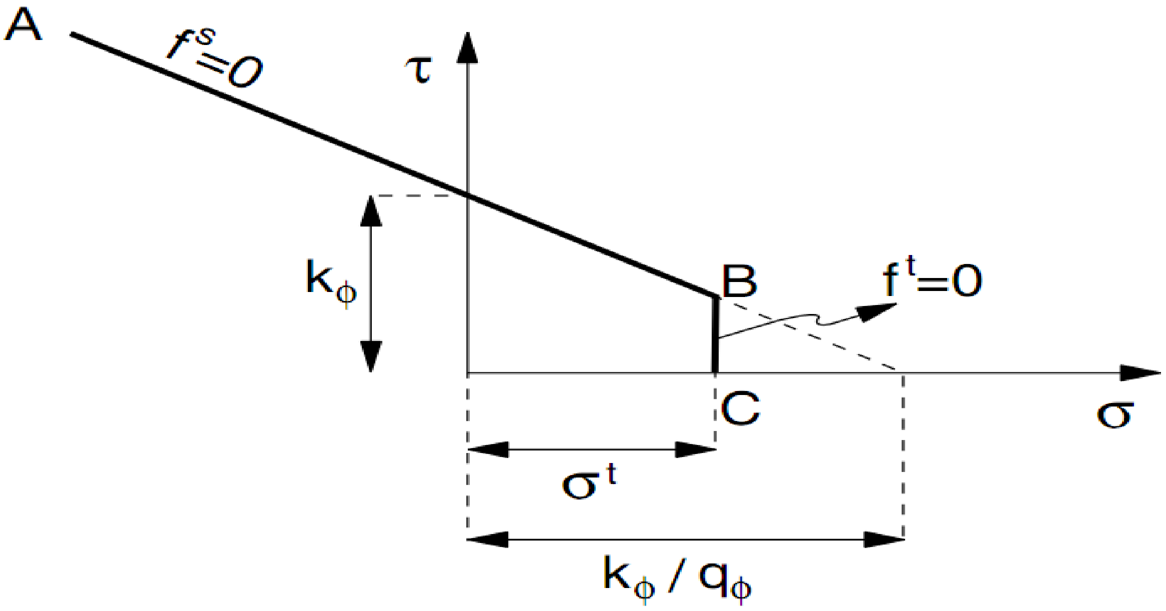

Figure 3.7 FLAC^{3D} Drucker-Prager failure criterion (Itasca Consulting Group, 2012)

The potential function $g(\tau, \sigma) =$ constant is composed of two functions, $g^s\ and\ g^t$, used to describe shear and tensile plastic flow, respectively. The function g^s corresponds in general to a non-associated law, and has the form

$$g^s = \tau + q_\psi \sigma \tag{3.32}$$

Where q_ψ is a constant which is equal to q_ϕ if the flow rule is associated. The function g^t corresponds to an associated flow rule, and is given by

$$g^t = \sigma \tag{3.33}$$

The flow rule is given a unique definition by application of the following technique. A function h (τ, σ) = 0, which is represented by the diagonal between the representation of $f^s = 0$ and $f^t = 0$ in the (τ, σ) plane (Figure 3.8), is defined. The function is selected with its positive and negative domains, as indicated on the figure, and has the form,

$$\tau - \tau^P - a^P(\sigma - \sigma^t) \tag{3.34}$$

Where τ^P and τ^P are two constants, defined as

$$\tau^P = K_\phi - q_\phi \sigma^t \tag{3.35}$$

$$a^P = \sqrt{1 + q_\phi^2} - q_\phi \tag{3.36}$$

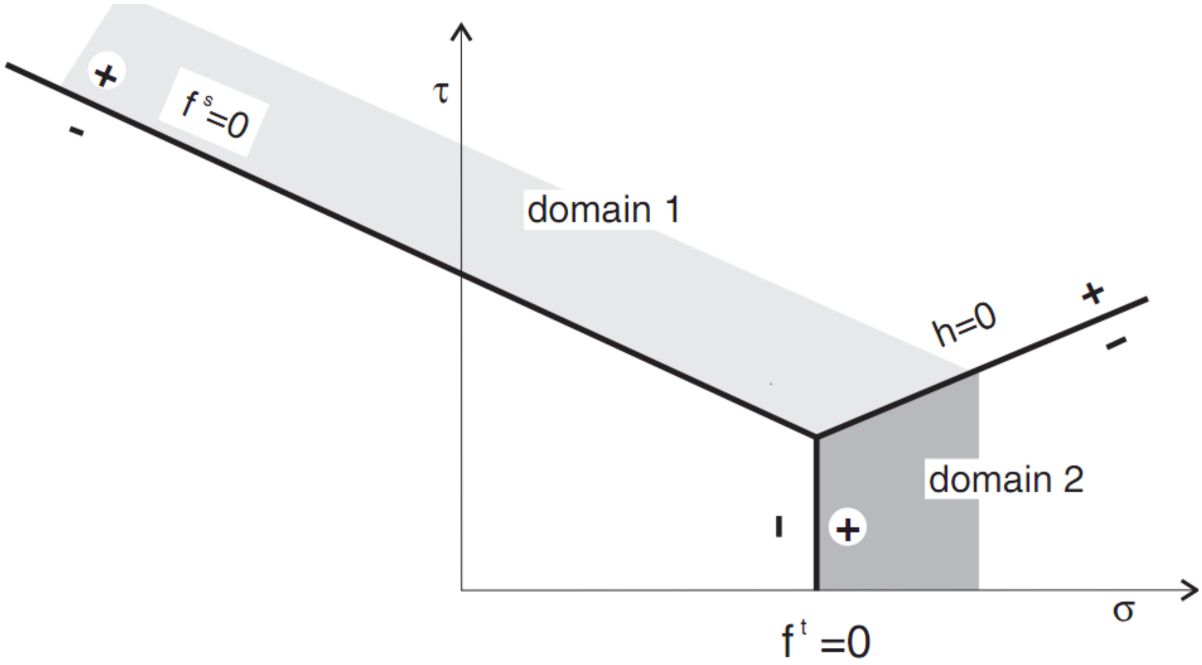

Figure 3.8 Drucker Prager model domains used in the definition of the flow rule (Itasca Consulting Group, 2012)

An elastic guess violating the composite yield function is represented by a point in the (τ, σ)-plane, located either in domain 1 or 2, corresponding to positive or negative domains of h = 0, respectively (Figure 3.8). If the stress point falls within domain 1, shear failure is declared, and the stress point is placed on the curve $f^s = 0$ using a flow rule derived using the potential function g^s. If the point falls within domain 2, tensile failure takes place and the new stress point satisfies $f^t = 0$ using a flow rule derived using g^t.

Model Parameters

The Drucker-Prager model in FLAC^{3D} is described by utilizing four parameters: q_ϕ, K_ϕ, q_ψ and σ^t. The particular case q_ϕ, $f^s = 0$ gives the Von Mises criterion. By appropriate adjustment of the parameters, the criterion may also be fitted to approximate the geometry of the Mohr-Coulomb or Tresca criterion in the (τ, σ)-plane.

The Drucker-Prager shear criterion $f^s = 0$ is represented in the principal stress space $(\sigma_1, \sigma_2, \sigma_3)$ by a cone with an axis along $\sigma_1 = \sigma_2 = \sigma_3$ and apex at $(\sigma_1, \sigma_2, \sigma_3) = (a, a, a)$, with $a = K_\phi / q_\phi$ (Figure 3.9). The Mohr-Coulomb criterion, characterized by two parameters (cohesion and friction angle), is represented there by an irregular hexagonal pyramid with the same axis and three “outer” and three “inner” edges (Figure 3.10).

The parameters q_ϕ and K_ϕ can be adjusted so that the Drucker-Prager cone will pass through either the outer or the inner edges of the Mohr-Coulomb pyramid.

For the outer adjustment, we have

$$q_\phi = \frac{6}{\sqrt{3}(3-\sin\varphi)}\sin\varphi \tag{3.37}$$

$$K_\phi = \frac{6}{\sqrt{3}(3-\sin\varphi)}c\cos\varphi \tag{3.38}$$

For the inner adjustment, we have

$$q_\phi = \frac{6}{\sqrt{3}(3+\sin\varphi)}\sin\varphi \tag{3.39}$$

$$K_\phi = \frac{6}{\sqrt{3}(3+\sin\varphi)}c\cos\varphi \tag{3.40}$$

In the special case $q_\phi = 0$, the Drucker-Prager criterion degenerates into the von Mises criterion, which corresponds to a cylinder in the principal stress space. The Tresca criterion is a special case of the Mohr-Coulomb criterion for which $\varphi = 0$. It is represented in the principal stress space by a regular hexagonal prism. The von Mises cylinder circumscribes the prism for

$$q_\phi = 0 \tag{3.41}$$

$$K_\phi = \frac{2}{\sqrt{3}}c \tag{3.42}$$

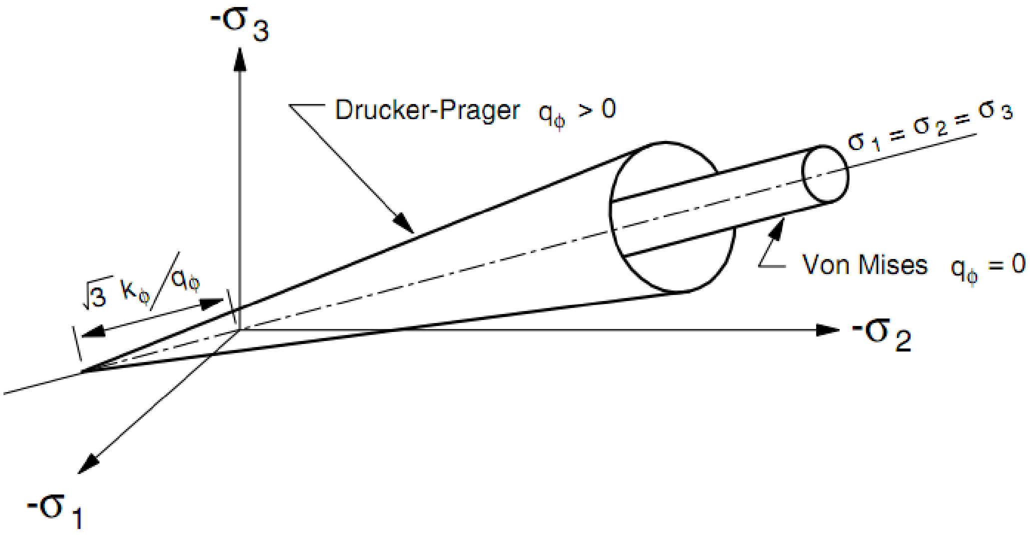

Figure 3.9 Drucker-Prager and von Mises yield surfaces in principal stress space (Itasca Consulting Group, 2012)

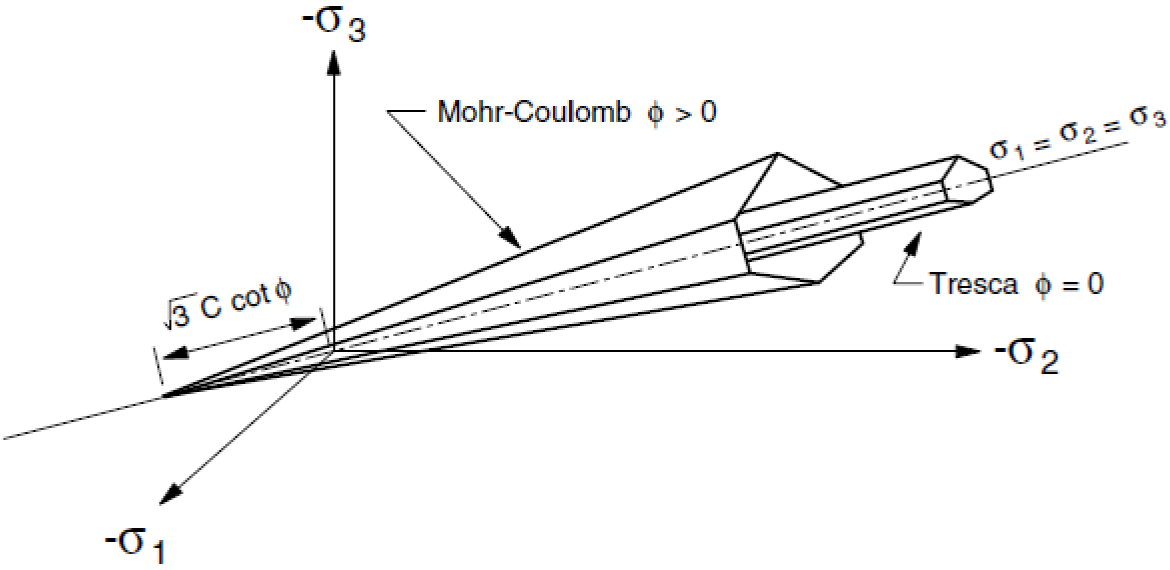

Figure 3.10 Mohr-Coulomb and Tresca yield surfaces in principal stress space (Itasca Consulting Group, 2012)

Table 3.15 Summary of D-P soil parameters for United bank building project

Property keyword	bulk	Density	Kshear	qdil	qvol	Shear	Tension		
Symbol	elastic bulk modulus, K	mass density, ρ	material parameter, K_ϕ	material parameter, q_ψ	material parameter, q_ϕ	elastic shear modulus, G	tension limit , σ^t	friction angle, φ	cohesion, c
Unit	$[MN/m^2]$	$[kg/m^3]$	$[kN/m^2]$	$[-]$	$[-]$	$[MN/m^2]$	$[kN/m^2]$	$[°]$	$[kN/m^2]$
Dense sandy gravel	14.28	1900	35296.71	0.511	0.794	11.54	44474.25	34	30
Stiff Silty clay	5.71	1700	42708.51	0.350	0.543	4.62	78614.46	24	35
Dense sandy gravel with rock cores	16.67	1900	34853.56	0.543	0.844	13.46	41289.55	36	30
Stiff Silty clay	5.71	1700	42708.51	0.350	0.543	4.62	78614.46	24	35
Dense sandy gravel	16.67	1900	34853.56	0.543	0.844	13.46	41289.55	36	30
Fractured rock	21.43	2000	34346.08	0.576	0.894	17.31	38395.32	38	30
Stiff Silty clay	5.71	1700	42708.51	0.350	0.543	4.62	78614.46	24	35
Fractured rock	21.43	2000	34346.08	0.576	0.894	17.31	38395.32	38	30

Where, $\sigma^t = 0$ for $q_\phi = 0$ otherwise $\sigma^t = k_\phi / q_\phi$

Table 3.16 Summary of D-P soil parameters for Nib bank building project

Property keyword	bulk	density	Kshear	qdil	qvol	Shear	tension		
Symbol	elastic bulk modulus, K	mass density, ρ	material parameter, K_ϕ	material parameter, q_ψ	material parameter, q_ϕ	elastic shear modulus, G	tension limit, σ^t	friction angle, φ	cohesion, c
Unit	$[MN/m^2]$	$[kg/m^3]$	$[kN/m^2]$	$[-]$	$[-]$	$[MN/m^2]$	$[kN/m^2]$	[°]	$[kN/m^2]$
Highly fractured rock	14.57	1700	-	0.63	0.98	11.77	-	41.4	-
Slightly weathered weak rock	47.62	2400	109298.3	0.66	1.02	38.46	107259.1	43	100
Highly facture rock with cavities	14.28	1800	-	0.64	0.99	11.54	-	42	-
Slightly to weathered highly fractured rock	23.81	2200	54938.31	0.65	1.01	19.23	54574.4	42.5	50
Slightly weathered basalt	71.43	2400	106828.3	0.69	1.07	57.69	100000	45	100

Table 3.17 Summary of D-P soil parameters for Zemen bank building project

Property keyword	bulk	density	Kshear	qdil	qvol	shear	tension		
Symbol	elastic bulk modulus, K	mass density, ρ	material parameter, K_ϕ	material parameter, q_ψ	material parameter, q_ϕ	elastic shear modulus, G	tension limit ,σ^t	friction angle, φ	cohesion, c
Unit	$[MN/m^2]$	$[kg/m^3]$	$[kN/m^2]$	$[-]$	$[-]$	$[MN/m^2]$	$[kN/m^2]$	$[^\circ]$	$[kN/m^2]$
Stiff sandy clayey silt	5.14	1961	30616.9	0.273	0.422	4.15	72603.88	19	25
Fine grained highly fractured to fragmented basalt	23.81	1820	-	0.625	0.970	19.23	-	41	-
Highly plastic clayey silt	3.43	1700	35515.6	0.273	0.422	2.77	84220.50	19	29
Stiff silty clay with some sand	4.14	1800	39175.1	0.258	0.398	3.35	98480.02	18	32
Stiff sandy clay silt	4.43	2400	31829.8	0.258	0.398	3.59	80015.02	18	26
Highly plastic clayey silt	23.81	2600	35515.6	0.273	0.422	19.23	84220.50	19	29
Clayey silty sand	4.14	1660	12233.8	0.319	0.494	3.35	24750.88	22	10

3.3.5.2 Mohr-Coulomb Model

For geomaterials like soils and rocks, in principal both the cohesion and the friction angle may increase (hardening) or decrease (softening) with the progress of plastic deformation, and that the hardening/softening behaviors are related only to the increase/decrease of the cohesion while the internal friction remains constant during the plastic deformation (Chen, 2012).

Mohr-Coulomb model is an elastic-perfectly plastic model that is often used to model soil behavior in general and serves as a first-order model. In general stress state, the model's stress-strain behaves linearly in the elastic range, with two defining parameters from Hooke's law (bulk modulus, K, and shear modulus, G). The failure criteria is defined by two parameters, namely cohesion, c and friction angle, φ and a flow rule which is defined by dilatency angle, ψ comes from the use of non-associated flow rule which is used to model a realistic irreversible change in volume due to shearing.

The Mohr-Coulomb model is the conventional model used to represent a shear failure in soils and rocks. The failure envelope for this model corresponds to a Mohr-Coulomb criterion (shear yield function) with tension cutoff (tension yield function). The position of a stress point on this envelope is controlled by a non-associated flow rule for shear failure and an associated rule for tension failure.

Generalized Stress and Strain Components

The Mohr-Coulomb criterion in FLAC^{3D} is expressed in terms of the principal stresses σ_1, σ_2 and σ_3, which are the three components of the generalized stress vector for this model (n = 3). The components of the corresponding generalized strain vector are the principal strains ϵ_1, ϵ_2 and ϵ_3.

Composite Failure Criterion and Flow Rule

The failure criterion used in the FLAC^{3D} model is a composite Mohr-Coulomb criterion with tension cutoff. In labeling the three principal stresses so that,

$$\sigma_1 \leq \sigma_2 \leq \sigma_3 \tag{3.43}$$

This criterion may be represented in the plane (σ_1, σ_3) as illustrated in Figure 3.11. The failure envelope $f(\sigma_1, \sigma_3) = 0$ is defined from point A to B by the Mohr-Coulomb failure criterion $f^s = 0$ with

$$f^s = \sigma_1 - \sigma_3 N_\phi + 2c\sqrt{N_\phi} \tag{3.44}$$

and from B to C by a tension failure criterion of the form $f^t = 0$ with

$$f^t = \sigma_3 - \sigma^t \tag{3.45}$$

Where φ is the friction angle, c is the cohesion, σ^t is the tensile strength, and

$$N_\phi = \frac{1 + \sin\varphi}{1 - \sin\varphi} \tag{3.46}$$

Note that the tensile strength of the material cannot exceed the value of σ_3 corresponding to the intersection point of the straight lines $f^t = 0$ and $\sigma_1 = \sigma_3$ in the $f(\sigma_1, \sigma_3)$ plane. This maximum value is given by

$$\sigma^t_{max} = \frac{c}{\tan\varphi} \tag{3.47}$$

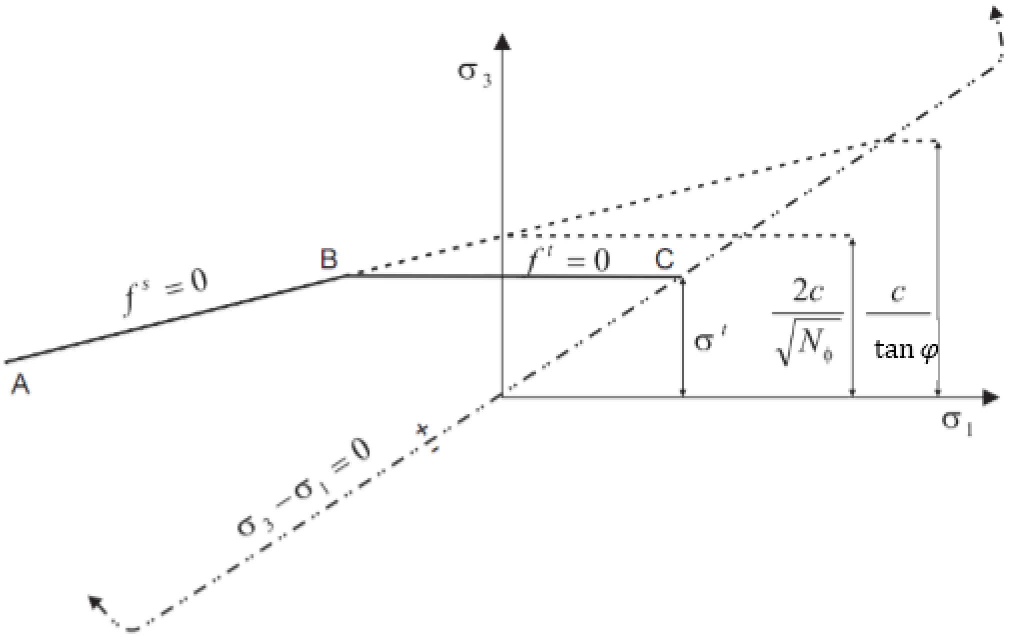

Figure 3.11 FLAC^{3D} Mohr-Coulomb failure criterion (Itasca Consulting Group, 2012)

The potential function is described by means of two functions, g^s and g^t , used to define shear plastic flow and tensile plastic flow, respectively. The function g^s corresponds to a non-associated law and has the form

$$g^s = \sigma_1 - \sigma_3 N_\psi \tag{3.48}$$

Where ψ is dilation angle and

$$N_\psi = \frac{1 + \sin\psi}{1 - \sin\psi} \tag{3.49}$$

The function g^t corresponds to an associated flow rule and is written

$$g^t = -\sigma_3 \tag{3.50}$$

The flow rule is given a unique definition by application of the following technique. A function $h(\sigma_1, \sigma_3) = 0$, which is represented by the diagonal between the representation of $f^s = 0$ and $f^t = 0$ in the (σ_1, σ_3)-plane (Figure 3.12), is defined. The function is selected with its positive and negative domains, as indicated on the figure, and has the form

$$h = \sigma_3 - \sigma^t + a^p(\sigma_1 - \sigma^p) \tag{3.51}$$

Where a^p and σ^p are constants defined as

$$a^p = \sqrt{1 + N_\phi^2} + N_\phi \tag{3.52}$$

$$\sigma^p = \sigma^t N_\phi - 2c\sqrt{N_\phi} \tag{3.53}$$

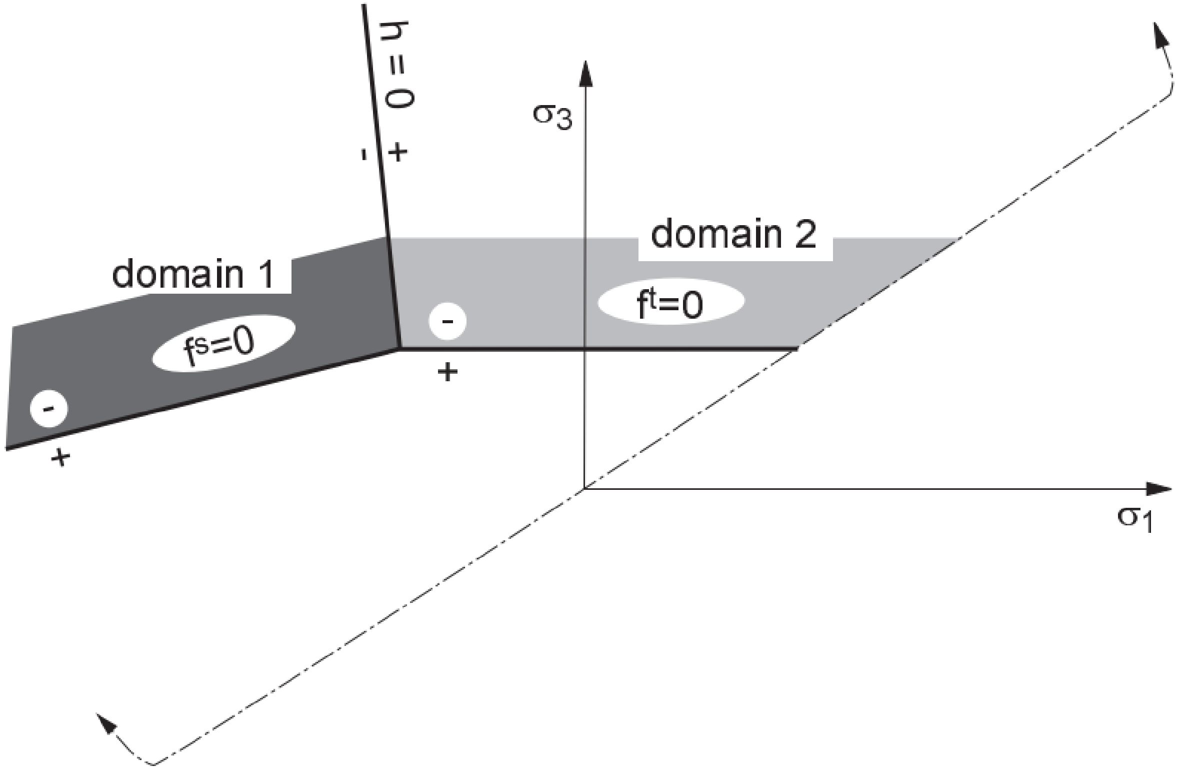

Figure 3.12 Mohr-Coulomb model – domains used in the definition of the flow rule (Itasca Consulting Group, 2012)

An elastic guess violating the composite yield function is represented by a point in the (σ_1, σ_3)-plane, located either in domain 1 or 2, corresponding to negative or positive domains of h =0, respectively (Figure 3.12). If the stress point falls within domain 1, shear failure is declared, and the stress point is placed on the curve $f^s = 0$ using a flow rule derived using the potential function g^s. If the point falls within domain 2, tensile failure takes place, and the new stress point conforms to $f^t = 0$ using a flow rule derived using g^t.

Note that by ordering the stresses as in Eq. (3.43), the case of a shear-shear edge is automatically handled by a variation on this technique. The technique, applicable for small-strain increments, is simple to implement: at each step, only one flow rule and corresponding stress correction is involved in the case of plastic flow. In particular, when a stress point follows an edge, it receives stress corrections alternating between two criteria. In this process, the two yield criteria are fulfilled to an accuracy, which depends on the magnitude of the strain increment.

3.4 Finite difference method based Numerical model

3.4.1 General

The governing equations for the analysis of geotechnical problems are formulated based on the fundamental requirements of equilibrium, compatibility, material behavior, and boundary conditions. For an exact theoretical solution, all of the aforementioned conditions must be satisfied.

FLAC^{3D} is an explicit finite difference program to study, numerically, the mechanical behavior of a continuous three-dimensional medium as it reaches equilibrium or steady plastic flow. The response observed derives from a particular mathematical model on one hand and a specific numerical implementation on the other. These two topics are addressed below.

3.4.1.1 Mathematical Model Description

The mechanics of the medium are derived from general principles (definition of strain, laws of motion), and the use of constitutive equations defining the idealized material. The resulting mathematical expression is a set of partial differential equations, relating mechanical (stress) and kinematic (strain rate, velocity) variables, which are to be solved for particular geometries and properties, given specific boundary and initial conditions (Itasca Consulting Group, 2012).

The state of stress at a given point of the medium is characterized by the symmetric stress tensor σ_{ij}. The traction vector $[t]$ on a face with unit normal $[n]$ is given by Cauchy's formulae (tension positive):

$$t_i = \sigma_{ij} n_j \tag{3.54}$$

Equations of Motion is derived by applying the continuum form of the momentum principle which yields Cauchy's equations of motion:

$$\sigma_{ij,j} + \rho b_i = \rho \frac{dv_i}{dt} \tag{3.55}$$

Where ρ is the mass-per-unit volume of the medium, [b] is the body force per unit mass, and $d[v]/dt$ is the material derivative of the velocity. These laws govern, in the mathematical model, the motion of an elementary volume of the medium from the forces applied to it. Note that, in the case of static equilibrium of the medium, the acceleration $d[v]/dt$ is zero, and Eq. (3.55) reduces to the partial differential equations of equilibrium:

$$\sigma_{ij,j} + \rho b_i = 0 \tag{3.56}$$

The above equation represent three scalar relations called the equilibrium equation

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial x} + \frac{\partial \tau_{zx}}{\partial x} + F_x = 0$$

$$\frac{\partial \sigma_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial x} + \frac{\partial \tau_{zy}}{\partial x} + F_y = 0 \tag{3.57}$$

$$\frac{\partial \sigma_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial x} + \frac{\partial \sigma_z}{\partial x} + F_z = 0$$

Where F is body force

The boundary conditions consist of imposed boundary tractions (Eq. (3.54)) and/or velocities (to induce given displacements). In addition, body forces may be present. Moreover, the initial stress state of the body needs to be specified.

Let the particles of the medium move with velocity $[v]$. In an infinitesimal time, $dt,$ the medium experiences an infinitesimal strain determined by the translations, $v_i\ dt,$ and the corresponding components of the strain-rate tensor may be written as

$$\xi_{ij} = \frac{1}{2}\left(v_{i,j} + v_{j,i}\right) \tag{3.58}$$

Where $\xi_{ij}$the strain rate tensor and partial derivatives are taken with respect to components of the current position vector$[x]$.

The developed theoretical solution obtained from the equation of motion Eq. (3.55) together with the definition of the rate of strain Eq. (3.58) forms only nine equations and constitutes fifteen unknowns (i.e. six components of stress, six components of strain-rate tensor, and three components of velocity vector). Therefore, to make the solution determinate six additional relations are provided by the constitutive equations which define the nature of the particular material.

3.4.1.2 Numerical model description

In a finite difference approach (First-order space and time derivatives of a variable are approximated by finite differences, assuming linear variations of the variable over finite space and time intervals, respectively.)

The governing equations are written in finite difference equations and the solutions are computed at specific discrete points called nodes. The process of subdividing the domain into discrete points is called discretization. Starting from $x = 0$ the domain is subdivided into steps until $x = 1$ the chosen step is equal to step size and is denoted by Δx.

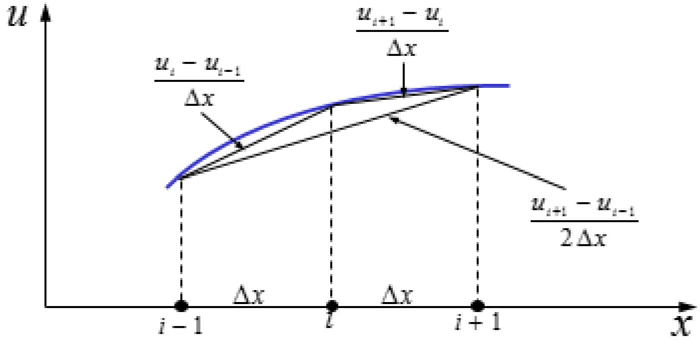

Figure 3.13 Finite difference equation derivation

The derivation of the finite difference equation based on the above figure yields,

Forward difference,

$$\left(\frac{du}{dx}\right)_i \approx \frac{u_{i+1} - u_i}{\Delta x}, \tag{3.59}$$

Backward difference,

$$\left(\frac{du}{dx}\right)_i \approx \frac{u_i - u_{i-1}}{\Delta x}, \tag{3.60}$$

Central difference

$$\left(\frac{du}{dx}\right)_i \approx \frac{u_{i+1} - u_{i-1}}{2\Delta x}, \tag{3.61}$$

The second-order derivative in finite difference equations is given by,

$$\left(\frac{d^2u}{dx^2}\right)_i = \frac{d}{dx}\left(\frac{du}{dx}\right)_i \cong \frac{1}{\Delta x}\left(\left(\frac{du}{dx}\right)_{i+1} - \left(\frac{du}{dx}\right)_i\right)$$

$$= \frac{1}{\Delta x}\left(\frac{u_{i+1} - u_i}{\Delta x} - \frac{u_i - u_{i-1}}{\Delta x}\right) = \frac{u_{i+1} - 2u_i - u_{i-1}}{\Delta x^2} \tag{3.62}$$

The laws of motion for the continuum are, by means of these approaches, transformed into discrete forms of Newton's law at the nodes. The resulting system of ordinary differential equations is then solved numerically using an explicit finite difference approach in time. The spatial derivatives involved in the derivation of the equivalent medium are those appearing in the definition of strain rates in terms of velocities.

3.4.1.3 Main calculation step

FLAC^{3D} uses an explicit "time-marching" finite difference solution scheme; for every time step, the calculation sequence can be summarized:

1. New strain rates are derived from nodal velocities.

2. Constitutive equations are used to calculate new stresses from the strain rates and stresses at the previous time.

3. The equations of motion are invoked to derive new nodal velocities and displacements from stresses and forces.

The sequence is repeated at every time step, and the maximum out-of-balance force in the model is monitored. This force will either approach zero, indicating that the system is reaching an equilibrium state, or it will approach a constant, nonzero value, indicating that a portion (or all) of the system is at steady-state (plastic) flow of material.

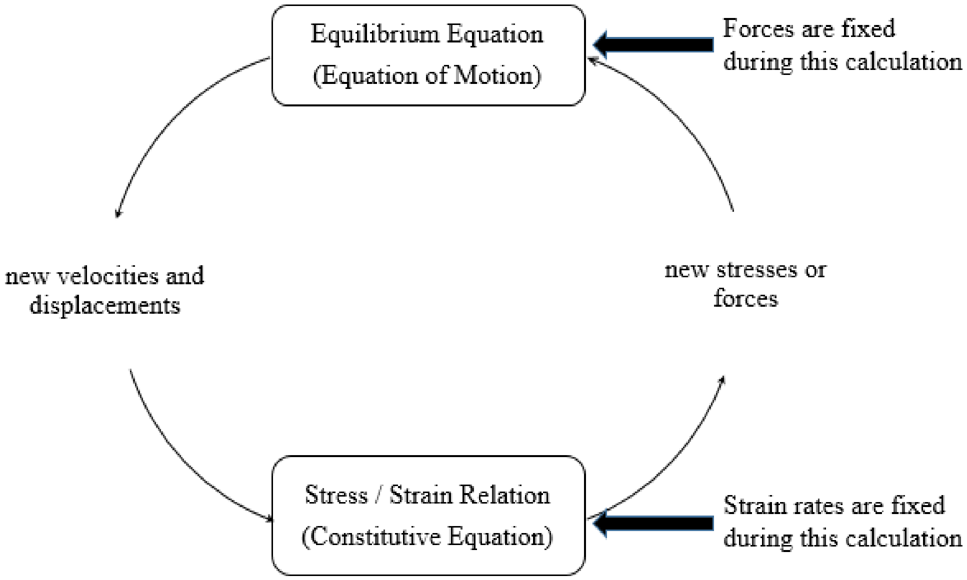

Figure 3.14 Calculation cycle (Roger Hart and Varun, 2012)

3.4.2 Pile-Soil/Rock Interaction Modeling

Problems in foundation engineering involve direct contact between soil/rock and structure. When external forces act on these systems both the structural displacement and the soil displacement show dependence on each other. The load-deformation of soil-structure systems is significantly influenced by the behavior of the soil-structure interface and its stress-displacement relationship. This is particularly important for pile-soil systems axially loaded, where one part of the pile total resistance (shaft resistance) is given by frictional resistance between the pile and the soil (S. D’Aguiar et al., 2008).

Interfaces are used when modeling soil/rock-structure interaction. Interfaces will be required to simulate the finite frictional resistance between the pile and adjacent soil. It allows relative displacement and separation between the structure and soil mass (Yi, 2004).

Interface elements simulate the normal and shear direction interactions of pile shaft with surrounding soil mass via shear coupling spring and normal coupling spring. Parameters of coupling spring are cohesion (c_n & c_s), friction (φ_s & φ_n) and spring stiffness (k_s & k_n) (Lin, Liu, & Chou, 2016). In this study, interfaces are introduced along the pile shaft and under the pile tip. Furthermore, interfaces along the pile shaft are also subdivided based on the adjacent soil layer, so interface parameters are derived from properties of the soil/rock in contact.

(FLA^{3D} manual, 2012) recommends a maximum value of shear and stiffness parameters to be set to ten times the equivalent stiffness of the stiffest neighboring zone.

$$\mathrm{k_n}, \mathrm{k_s} = 10\mathrm{max}\left[\frac{\mathrm{k} + \frac{4}{3} * \mathrm{G}}{\Delta \mathrm{Z_{min}}}\right] \tag{3.63}$$

Where, $\Delta \mathrm{Z_{min}} -$ the smallest width of an adjoining zone in the normal direction

Interface cohesion and friction values have been set to 2/3 to 0.9 times the adjacent soil. The computed interface parameters are listed in the table below

Table 3.18 Interface parameters for United bank

Layer	Δz_{min}	K(MPa)	G(MPa)	k_s, k_n
1	0.069	14.28	11.54	429951691
2	0.069	5.71	4.62	172028986
3	0.069	16.67	13.46	501690821
4	0.069	5.71	4.62	172028986
5	0.069	16.67	13.46	5.02E+08
Pile	0.1333	1.39E+04	10400	2.083E+11
Above	0.525	1.39E+04	10400	5.29E+10
Below	1.125	19.05	15.38	
			Along pile [Max*10]	5.02E+09
			Pile tip [Max*10]	5.29E+11

Table 3.19 Interface parameters for Nib bank project

Layer	Δz_{min}	K(MPa)	G(MPa)	k_s, k_n
1	0.069	14.57	11.8	4.39E+08
2	0.069	47.62	38.5	1.43E+09
3	0.069	14.28	11.5	4.3E+08
4	0.069	23.81	19.2	7.17E+08
5	0.069	71.43	57.7	2.15E+09
Pile	0.1333	1.39E+04	1.04E+04	2.08E+11
Above	0.512	1.39E+04	1.04E+04	5.42E+10
Below	1	71.43	57.7	
			Along pile [Max*10]	2.15E+10
			Pile tip [Max*10]	5.42E+11

Table 3.20 Interface parameters for Zemen bank project

Layer	Δz_{min}	K(MPa)	G(MPa)	k_s, k_n
1	0.03931	5.14	4.15	2.72E+08
2	0.03931	23.8	19.2	1.26E+09
3	0.03931	3.43	2.77	1.81E+08
4	0.03931	4.14	3.35	2.19E+08
Pile	0.1333	1.39E+04	1.04E+04	2.08E+11
Above	0.525	1.39E+04	1.04E+04	5.29E+10
Below	0.8333	4.14	3.35	
			Along pile [Max*10]	1.26E+10
			Pile tip [Max*10]	5.29E+11

The contribution of the pile tip and skin friction resistance were examined. As shown in Figure 3.15, the resistance offered by the pile tip is 87.5% of the total resistance, while the rest percentage of capacity is owed to the skin friction.

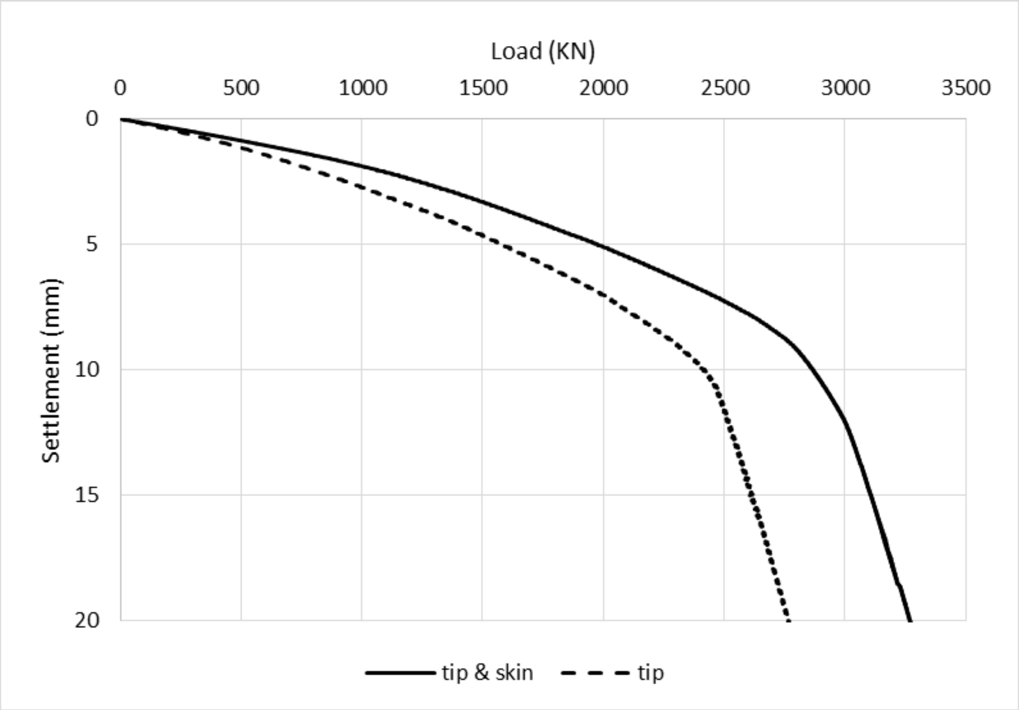

Figure 3.15 Comparison between pile tip and total resistance (Nib bank project pile no. 1)

3.4.3 Finite Difference Model Construction

3.4.3.1 Model Geometry and Discretization

This study is composed of circular concrete piles of different diameter embedded in a layered soil subjected to vertical loading under undrained condition. A vertical plane through the pile axis is a plane of symmetry for this analysis. As a result, only half of the pile-soil system is modeled to reduce computational time. According to Randolph and Wroth (1978), a minimum distance at which the deflections in the soil are assumed to become vanishingly small were found to be greater than the length of the pile L, therefore the distance from the axisymmetric axis to the horizontal boundary of 1.5L was adopted. The vertical boundary depth equal to 2L was chosen, which is reported as proper by numerous authors (D'Aguiar, 2008; Mascarucci et al., 2013, and others) for this type of problem. The sides` boundaries were restrained horizontally, while the bottom was prevented from moving either vertically and horizontally.

The soil strata were modeled with eight noded-brick shape elements and the pile using six-noded cylindrical shape elements.

The soil-pile interface is modeled by the standard FLAC^{3D} interface consisting of triangular elements. By default, two triangular interface elements are defined for each quadrilateral zone face. Interface nodes are created automatically at every interface element vertex. When another grid surface meets an interface element, the contact is detected at the interface node and is characterized by normal and shear stiffnesses, and sliding properties. Each interface element distributes its area to its nodes in a weighted fashion. Each interface node has an associated representative area. The entire interface is thus divided into active interface nodes representing the total area of the interface. The interfaces are one-sided and attached to the soil. It is sensitive to interpenetration with the other surface, denoted as the target face (pile). Figure 5.5 illustrates the relationship between interface elements and interface nodes, and the representative area associated with an individual node (Itasca Consulting Group, 2012).

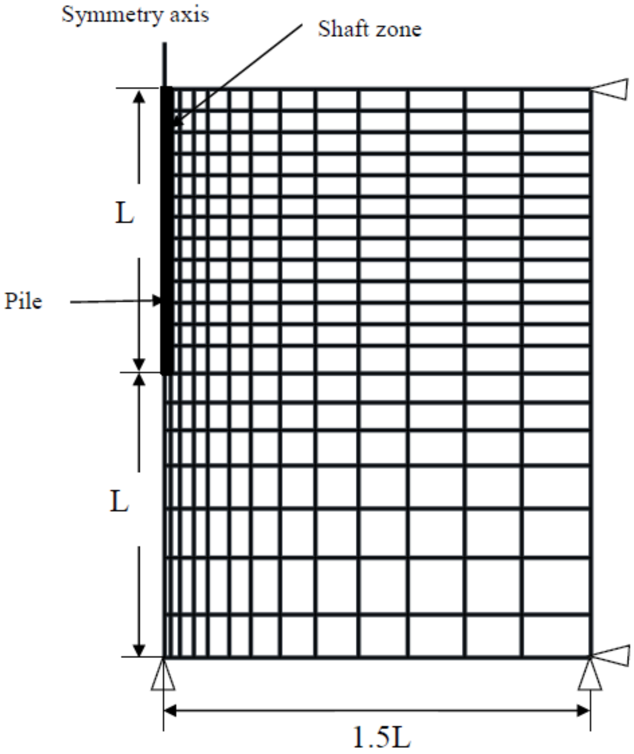

Figure 3.16 Typical discretization of computational model (Zeleke, 2015)

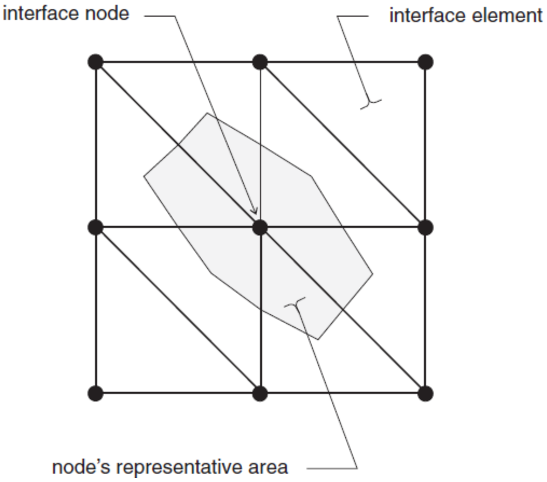

Figure 3.17 Interface consisting of eight triangular interface elements (Itasca Consulting Group, 2012)

3.4.4 Effect of Mesh Quality

Mesh quality is crucial for the stability, accuracy, and fast convergence of numerical simulations. Aspect ratio, orthogonality (interior angle), face planarity, skew, and taper are the most common measures of the quality of structured elements. Over the past decade, aspect ratio and orthogonally have become more important and are widely used quality measures for 2-dimensional and 3-dimensional elements (B. Abbasi et al., 2008). The integrity measurement of gridded models according to (Itasca Consulting Group, 2012) are described below.

Aspect ratio measures the height to width ratio of zones (i.e. edges of each internal tetrahedron for a given zone the smallest aspect ratio between the edges of each internal tetrahedron for a given zone, considering FLAC^{3D} sub-zone discretization). For all zones types, it is assumed that the ideal shape is the one for which all edges are equal in length, and angles between the edges are 90 degrees for a brick, 60 degrees and 90 degrees for a wedge or a degenerate brick, 60 degrees for a pyramid or a tetrahedron. For an ideal tetrahedron, the aspect ratio is 1. However, for the internal tetrahedrons composing zones with the ideal shapes, the smallest aspect ratio is $1/\sqrt{3}$ for those internal tetrahedrons in an ideal brick and $1/\sqrt{2}$ for an ideal pyramid and wedge. Thus the result is required to be normalized to the range between 0 and 1 by multiplying a factor of $\sqrt{3}$ for bricks, and $\sqrt{2}$ for pyramids and wedges.

Orthogonality measurement shows how "well" sides of the zones (for each face) are inclined relative to each other. If the test returns a value close to zero, it means that the zone is most likely very elongated or badly deformed. For ideal brick, this minimum value is 1, but for other zone types with ideal shape, the minimum value is $\sqrt{2}$. Thus this measure has to be multiplied by $1\sqrt{2}$ for zones except for bricks.

Face Planarity for hexahedrons having 6 quadrilateral faces, just like a cube's square faces. However, it is possible that the four vertices of a quadrilateral polygon will not be coplanar in 3D. FLAC^{3D} allows faces to be non-planar, but the greater the deviation, the less accurate the solution process will be. There is no clear singular method of measuring planarity. A method that compares the volume of a tetrahedron filling the 4 vertices and the area of the quadrilateral face, computing the area by adding a central point, m = (A + B + C + D)/4, and computing the 4 triangle areas, ABC, ABD, ACD, BCD, is chosen. The equation is, therefore, given by,

$$= \frac{\sqrt[3]{V}}{\sqrt{A}}, \tag{3.64}$$

Where A is the area and V is the volume

This value is zero if planar, and positive if non-planar. This test can be “scaled” by a constant because there is no fundamental limit on how non-planar a face can be. (Values should be << 1.0 for a face to be good.) Each zone has its six faces tested, and the worst value is reported.

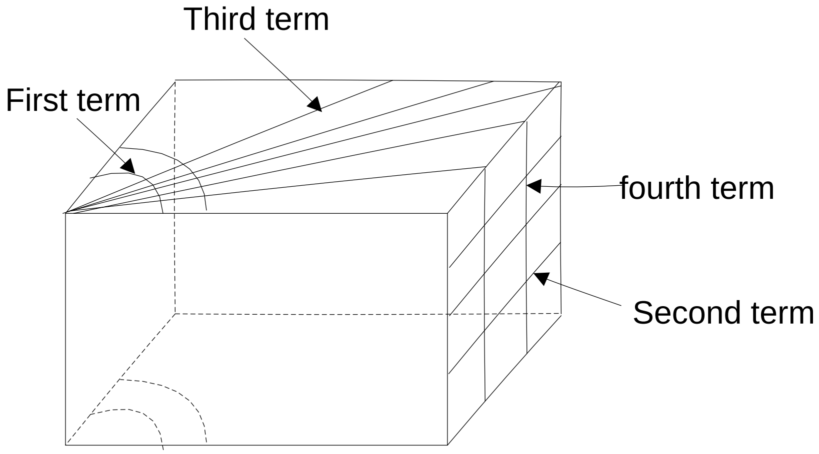

Figure 3.18 The order of numbers that grid lines are represented to i.e. size (1st, 2nd, 3rd, 4th)

In this study, the mesh was formed using ‘size’ and ‘ratio’ keywords. The ‘size’ specifies the number of zones for each shape, while the ‘ratio’ is used to space zones with an increasing/decreasing geometric ratio. The four terms in the size keyword were chosen based on the relation in order of, size (3, z-dimension, ¾*y-dimension, 2*x-dimension).

The study had compared two meshing types only by varying the last term of the ratio. Table 3.21 and 3.22 shows the mesh condition for the two types of meshing techniques.

Table 3.21 Measured mesh condition for ratio (1, 1, 1, 1.15)

Range	Number of zone		
	Orthogonality	Aspect ratio	Face planarity
0-0.1	0	7458	31218
0.1-0.2	1386	4984	0
0.2-0.3	0	5154	0
0.3-0.4	0	3604	0
0.4-0.5	0	2694	0
0.5-0.6	0	2544	0
0.6-0.7	0	2264	0
0.7-0.8	6068	1770	0
0.8-0.9	7216	746	0
0.9-1.0	16548	0	0
Worst values	0.142315	0.0190238	9.1067E-06

Table 3.22 Measured mesh conditions for ratio (1, 1, 1, 1)

Range	number of zone		
	Orthogonality	Aspect ratio	Face planarity
0-0.1	0	4994	31218
0.1-0.2	1368	3512	0
0.2-0.3	0	8972	0
0.3-0.4	0	4762	0
0.4-0.5	0	2968	0
0.5-0.6	0	2250	0
0.6-0.7	0	1980	0
0.7-0.8	9266	1244	0
0.8-0.9	7462	536	0
0.9-1.0	13104	0	0
worst values	0.142315	0.0190238	9.85317E-06

Figure 3.19 and 3.20 shows the two types of meshing techniques

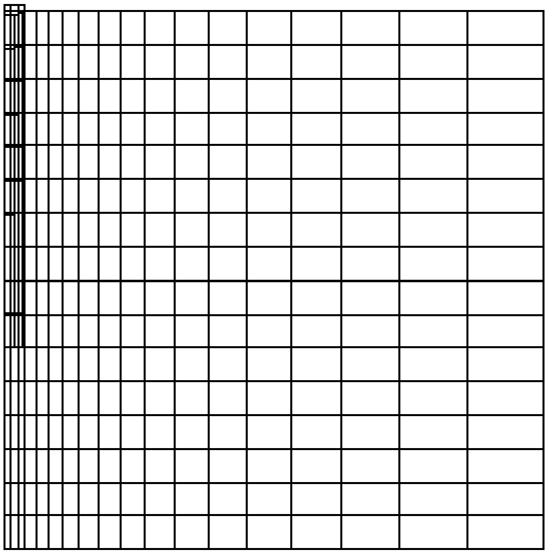

Figure 3.19 Finer mesh near the pile (high stress gradient zone)

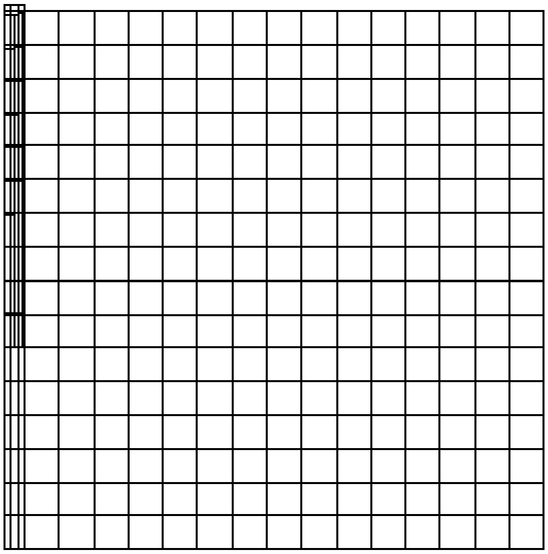

Figure 3.20 Coarser mesh near the pile

In this section of the study, an attempt has been made to compare two types of meshing techniques. The numerical model of Nib International Bank project is chosen to study the effect of applying different types of meshing. The meshing provided for Figure 3.19 is a finer mesh near the pile (i.e. high-stress gradient zone) and becomes coarser towards the boundary. However, the meshing provided for figure 3.20 is uniform coarse mesh throughout the whole geometry of the model. Comparison between table 3.19 and 3.20 depicts that as the number of zones closer to the ideal zone shape increases the quality of the simulation will also improve.

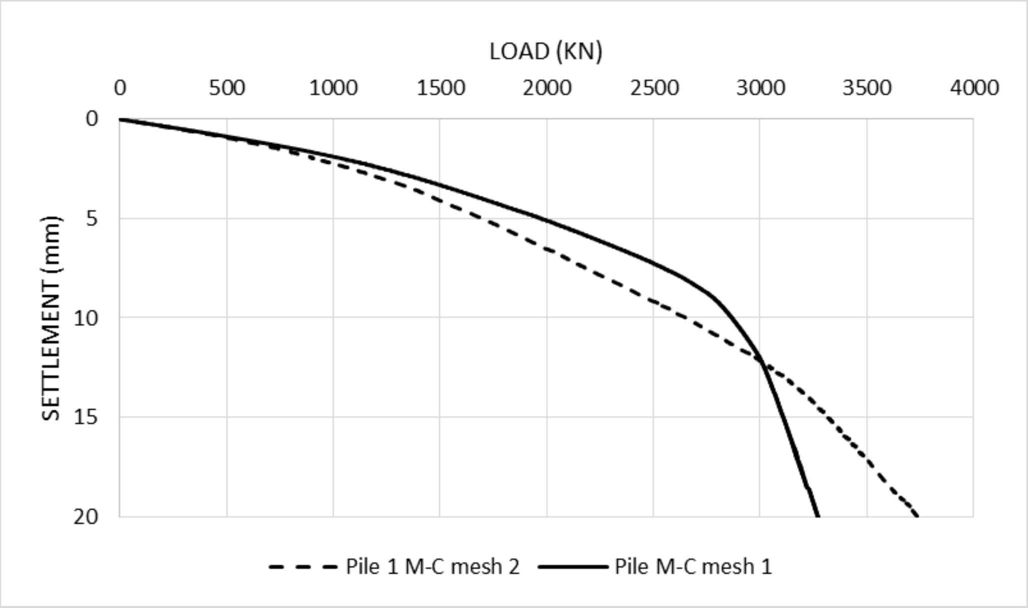

Figure 3.21 Effect of mesh type in load settlement curve

Figure 3.21 shows that the provision of coarser mesh leads to numerical instability and unclear distinct plunging load capacity. However, the finer mesh application near the pile yields a better result in terms of showing the plunging curve clearly, so as the ultimate capacity of the pile is easily determined. Most importantly, finer mesh lead to results that are more accurate in that they provide a better representation of high-stress gradient.

3.4.5 Modeling Procedure

To set up a model to simulate with FLAC 3D, three fundamental components of a problem must be specified: (FLAC 3D manual, 2012)

(1) A finite difference grid;

(2) Constitutive behavior and material properties; and

(3) Boundary and initial conditions.

The grid defines the geometry of the problem. The constitutive behavior and associated material properties dictate the type of response the model will display upon disturbance (e.g., deformational response due to excavation). Boundary and initial conditions define the in-situ state (i.e., the condition before a change or disturbance in the problem state is introduced).

After these conditions are defined in FLAC 3D, the initial equilibrium state is calculated for the model. An alteration is then made (e.g., excavate material or change boundary conditions), and the resulting response of the model is calculated. FLAC 3D uses an explicit time-marching method to solve the algebraic equations. The solution is reached after a series of computational steps. The number of steps required to reach a solution can be controlled automatically by the code or manually by the user.

The procedures followed for this particular study are discussed below:

1. Grid generation
   Basic grid generation is performed via the ‘Generate’ command and associated keywords that both define the number of zones in a model and shape the grid to fit a specified problem region. Built-in primitive shapes are used to speed up mesh generation. Accordingly, radcylinder and cylinder primitive shape are used to construct soil and pile, respectively. At this stage, interfaces between the two materials are also introduced.
2. Constitutive model and material behavior assignment
3. Initial stress

   In geological materials, there is an in-situ state of stress in the ground before any excavation or construction is started. By setting the initial conditions in a FLAC^{3D} grid, an attempt is made to reproduce this in-situ state because it will influence the subsequent behavior of the model. At this stage, the model must be brought to equilibrium before installing the pile. For this thesis, ‘initial’ and ‘grad’ commands are used to install initial stress. Since grad has been used, a value according to the following relation,

$$\mathrm{S} = \mathrm{S}^{(o)} + \mathrm{g_x x} + \mathrm{g_y y} + \mathrm{g_z} z \tag{3.65}$$

Where, $S^{(o)}$ the value immediately following Szz in the command, g_x, g_y and g_z specify the variations of the value in x, y, and z directions and are the numbers following the gradient keyword.

4. Pile installation

   In this stage of analysis, the model is brought into equilibrium after the installation of the pile. The installation is modeled by changing the properties of the pile zones from the properties representing the soil material to those representing the concrete pile material.

5. Loading

   To simulate the actual pile load test procedure the load is applied in an incremental manner using a ‘ramp’ (FISH function). In numerical analysis, the load is applied in various ways (by applying new stress or displacement, by applying velocity) If, however, a velocity loading condition is specified at the boundary of a body, it is sometimes beneficial to initialize the velocities throughout the body to minimize the shock to the system.

   In this study, the FISH function ‘ramp’ is used to apply velocity to the pile top grid points. The FISH function ‘zs_top’ calculates the axial stress at the top of the pile and stores the value as history. The velocity is applied as a ramp from 0.0 to 10−8 over 30,000 steps.

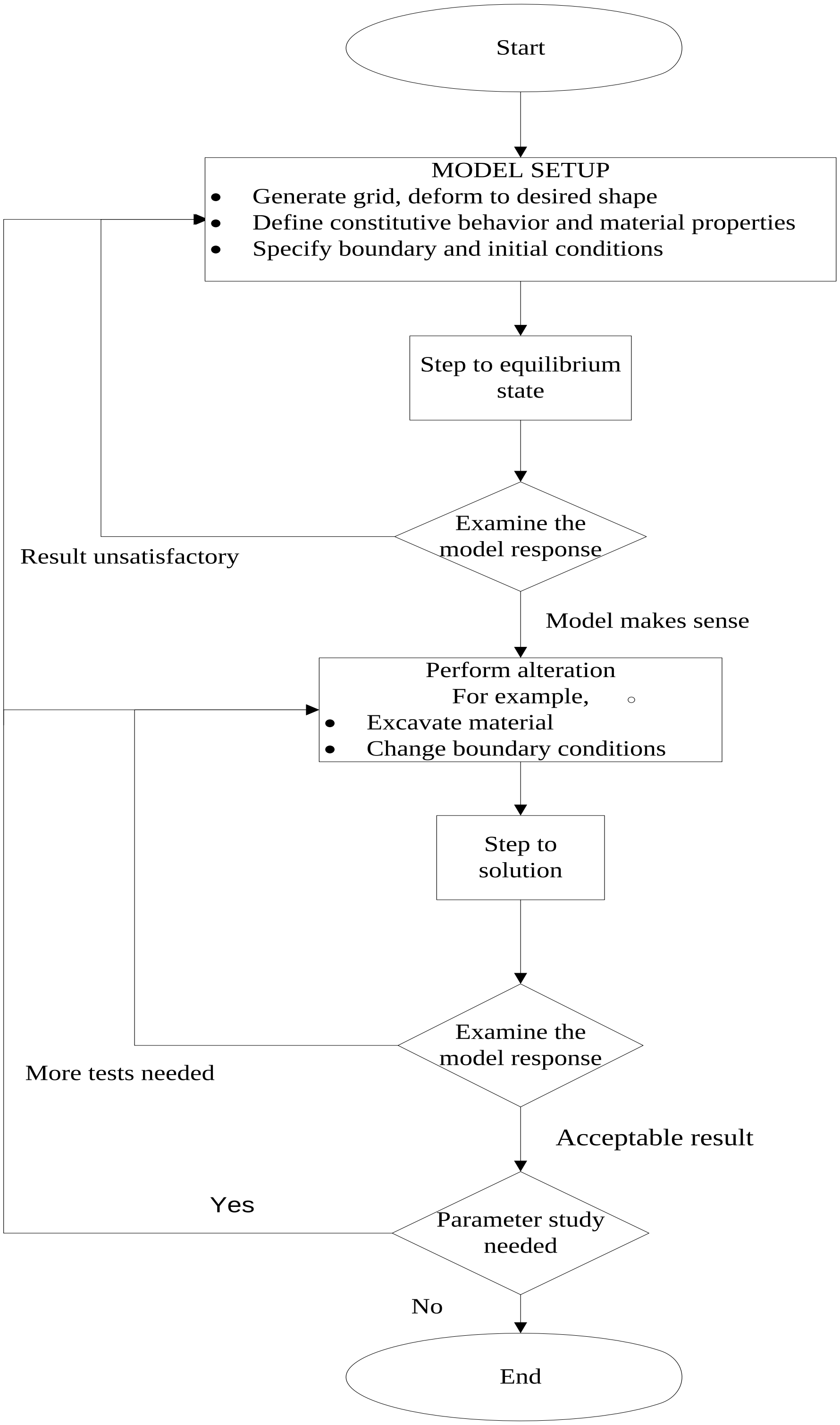

Figure 3.22 General solution procedure

4. Results and Discussion

4.1 Validation and Verification of FDM Simulated Results

The potential use of the results from the numerical simulation is related to the confidence that one might have in the simulation. An approach to establishing the credibility of the numerical simulation is through the verification and validation process. Verification is the process of determining that a model implementation meets an accurate representation of the proposed conceptual model and the solution to the model. While validation is the assessment of the accuracy of numerical simulation in comparison with experimental data.

For this study, the verification is performed by assessing the implementation of the initial stress condition of the model and the stress increment after the load-application. The initial stress state, which is given as an input for the numerical model is checked if it is accurately implemented in the FDM simulation. Then, the stress increment at the bottom of the model is assessed by comparing it with the theoretical approximations (based on Westergaard's solution). Hence, the theoretical approximation is not considered as an accurate benchmark, rather it is just used as a reference to demonstrate that the model is performing stress increment properly when subjected to a certain load.

Most importantly, the numerical model of the study was validated by comparing it with the actual pile load test results. Besides, a calibration was also performed by comparing the proposed numerical model in reference to a published study, and a factor was determined for the proposed model (Figure 4.1).

$$\sigma_z = \frac{{}^{c}\!/_{2\pi}}{\left(c^2 + ({}^{r}\!/_{cz})^2\right)^{3/2}} * \frac{Q}{z^2} \tag{4.1}$$

Where, $c = \sqrt{{}^{(1-2\nu)}\!/_{(2-2\nu)}}$

$Q -$ is applied load in KN

$z -$ is the depth to which the stress is computed

$r -$ is the horizontal distance between point of load application and point of the required stress and $\nu -$ is the Poisson's ratio

Table 4.1 Comparison of total vertical stress for Nib bank building project

Model type	Analysis steps	Vertical stress(pa)		Difference (%)
		Theoretical	FDM	
Mohr coulomb	Initial stress	936600	936670	0.007
	Final loading	936631.6	936740	0.01
Drucker Prager	Initial stress	936600	936670	0.004
	Final loading	936631.6	936700	0.007

Table 4.2 Comparison of total vertical stress for Zemen bank project

Model type	Analysis steps	Vertical stress (pa)		Difference (%)
		Theoretical	FDM	
Mohr coulomb	Initial stress	741231.5	741260	0.0038
	Final loading	741266.32	752000	1.448
Drucker Prager	Initial stress	741231.5	741260	0.0038
	Final loading	741266.32	741840	0.077

Table 4.3 Comparison of total vertical stress for United bank project

Model type	Analysis steps	Vertical stress (pa)		Difference (%)
		Theoretical	FDM	
Mohr coulomb	Initial stress	784500	784530	0.0038
	Final loading	784531.6	797350	1.63
Drucker Prager	Initial stress	784500	784530	0.0038
	Final loading	784531.6	786040	0.192

To check the validity of the proposed FD model, the reported actual test results calculated during the pile test (AbdelMohsen et al., 2011) were used. The bored cast-in-situ pile considered for validation is 1 meter in diameter and 25 meters long.

The parameters of the soil strata considered in this section are shown in the table below.

Table 4.4 Soil design parameters for the simulation

Layer	Fill	Clay	sand	clay	sand	Clay
Thick (m)	1.4	9.1	9.5	4.5	3.5	9
γ $(\mathrm{KN/m^3})$	16	18	18.5	18.5	19	18.5
Cu $(\mathrm{KN/m^2})$	-	125	-	135	-	125
φ (degree)	32	-	36	-	38	-
Es (MPa)	20	87.5	75	10.3	112	87.5
ν	0.30	0.35	0.30	0.35	0.30	0.35

In the FLAC 3D model, eight nodded bricks and six nodded cylindrical-shaped elements were chosen for soil and pile, respectively. The pile is made of reinforced concrete, the behavior was assumed linear elastic, and the M-C model has been used to approximate the behavior of the soil strata.

Field test results were compared with the FLAC 3D model and shown in Figure 4.1.

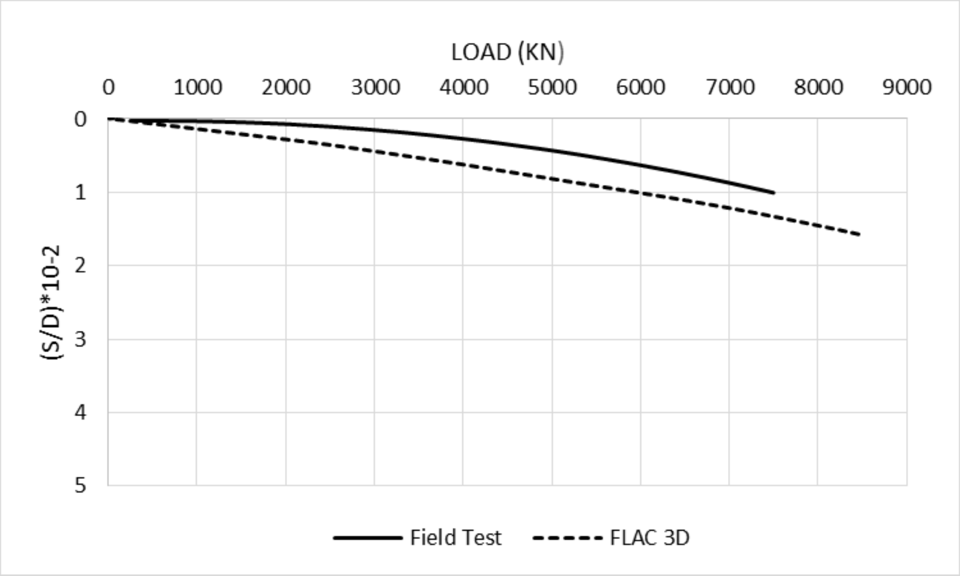

Figure 4.1 Comparison of load-settlement graph

In the above figure, the curve obtained from the FLAC 3D simulation is shown to be comparable to field test results with a small deviation. The deviation may have resulted from an unsuitable estimate of soil-pile interface strength parameters. Furthermore, the extent to which the simulation deviates from the actual result has been computed and a factor of 0.76 was found.

In general, the FLAC 3D analysis can capture the deformation behavior of a single pile fairly well. Therefore, the above simulation procedures are considered valid for further analysis.

4.2 Comparison between Numerical Analysis and Actual Pile Load Test Results

4.2.1 Load Settlement analysis result

The Load-Settlement relationship generated from the numerical analysis was plotted on the same graph with the field load-settlement curve for two different types of constitutive models (i.e. Mohr-Coulomb and Drucker-Prager) in Figures 4.2 through 4.6.

Figures 4.2 and 4.3 present the load-settlement curves of field data and numerical results. When comparing the curves, it is observed that the numerical analysis underpredicts field test results. This disagreement may be resulted due to the generalized parameters used for the soil profile in the numerical simulation for the piles under consideration. However, the curve obtained using the M-C model outperforms the D-P model in terms of exhibiting closeness to the field measurements.

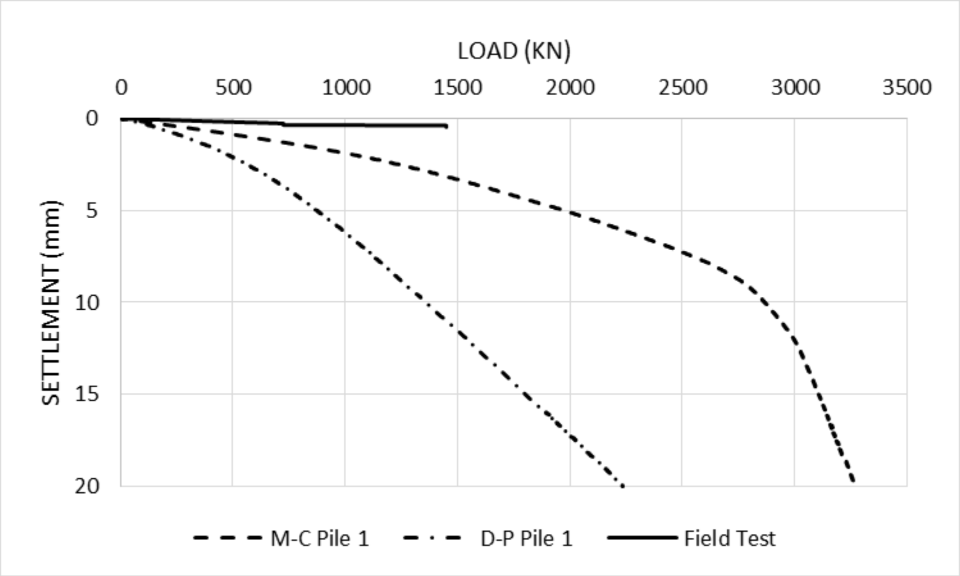

Figure 4.2 Comparison of finite difference method with pile load test (Nib pile no. 1)

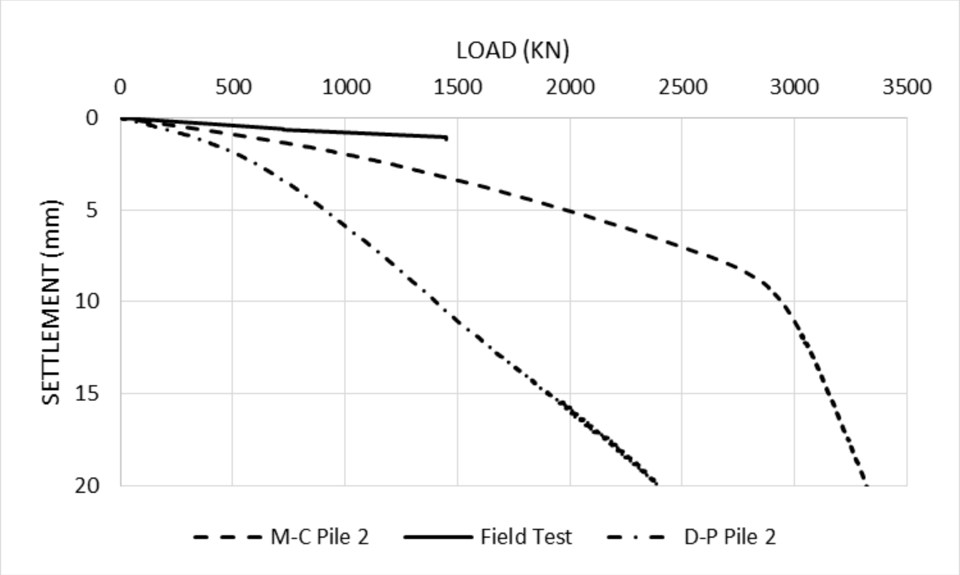

Figure 4.3 Comparison of finite difference method with pile load test (Nib pile no. 2)

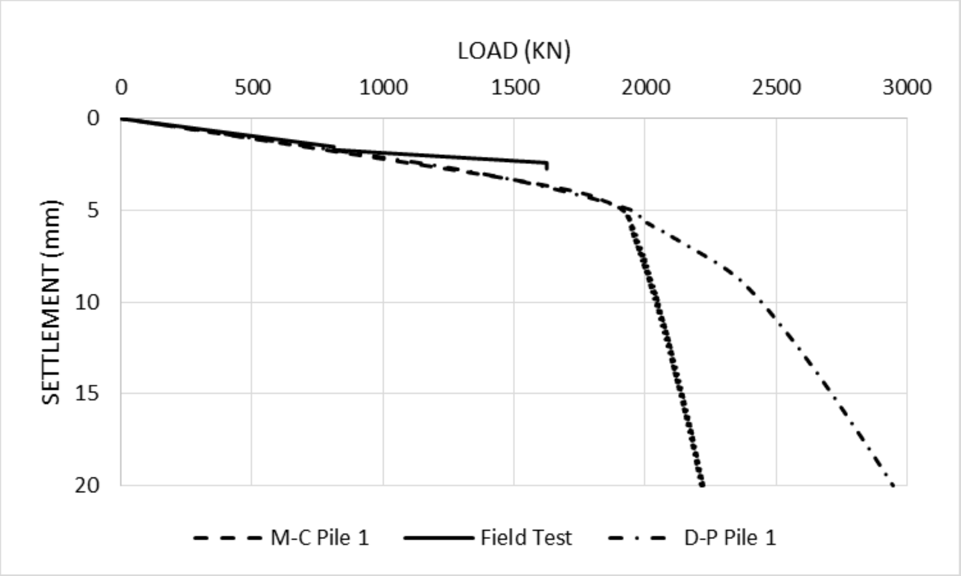

Figure 4.4 Comparison of finite difference method with pile load test (United pile no. 1)

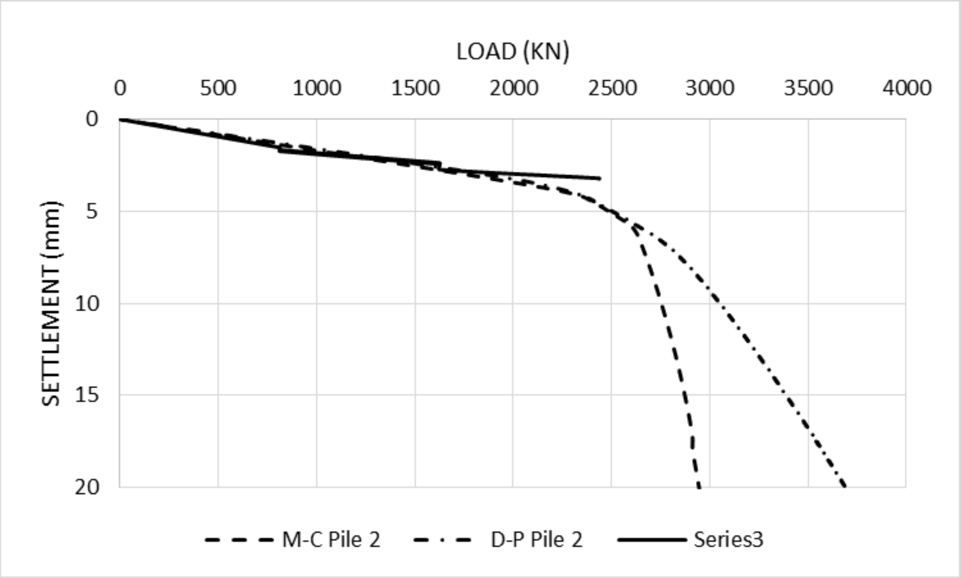

Figure 4.5 Comparison of finite difference method with pile load test (United pile no. 2)

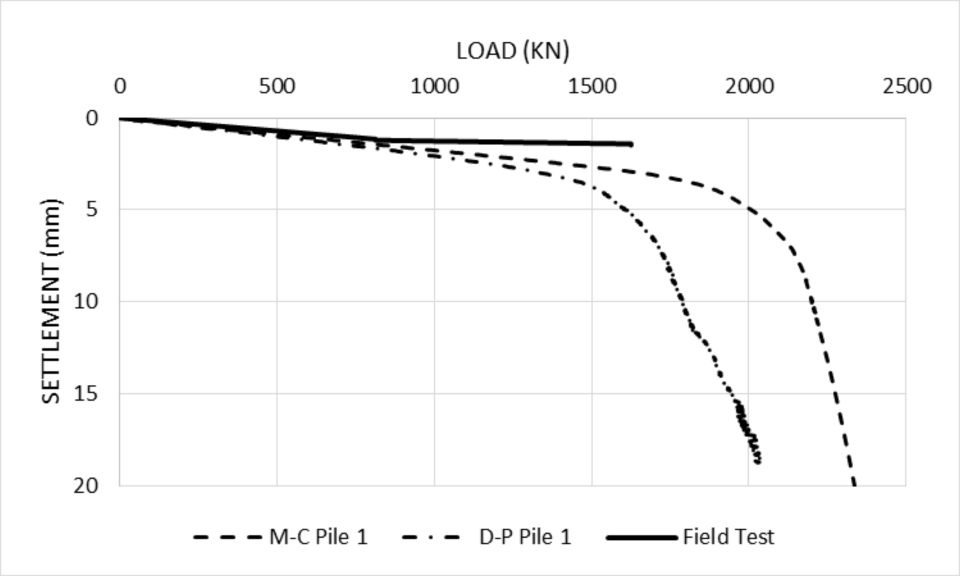

Figure 4.6 Comparison of finite difference method with pile load test (Zemen pile no. 1)

As shown in Figures 4.4, 4.5, and 4.6 the simulated results are almost identical to the ones given by the field test results at smaller loads. However, as the load increases the field test curve begins to get stiffer and starts to deviate from FLAC 3D prediction. These figures additionally indicate that the simulation results obtained using M-C and D-P models display the same behavior.

The Drucker-Prager model curves in Figures 4.2 and 4.3 do not show any distinct failure load point. However, all curves obtained using the M-C model displayed a distinct failure load in which the plastic deformation will start to takes place (i.e. the ultimate pile capacity).

Overall, the simulated results matched the curves of the field test fairly well, but the ultimate pile capacity was underestimated. Hence, in the next section, an attempt was made to explain the reason by examining the effect of various parameters on the load settlement curve.

4.3 Parametric Study

4.3.1 Young’s Modulus

In this section, a parametric study had been conducted for Nib bank building project Pile 1 (M-C model) by varying the values of Young’s modulus values along the pile shaft and at the pile tip. The results are shown in Figures 4.7 through 4.10.

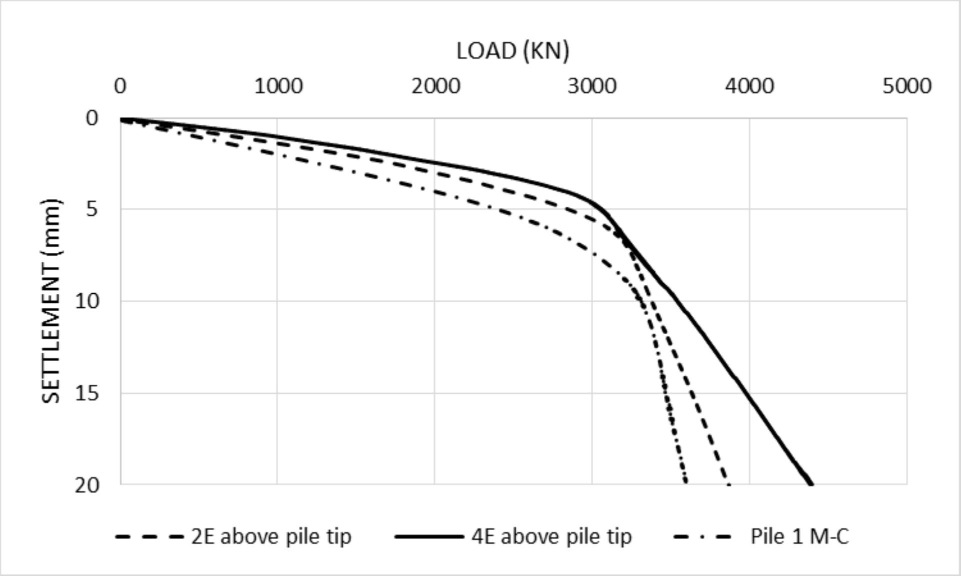

Figure 4.7 Effect of young’s modulus of the soil along the pile

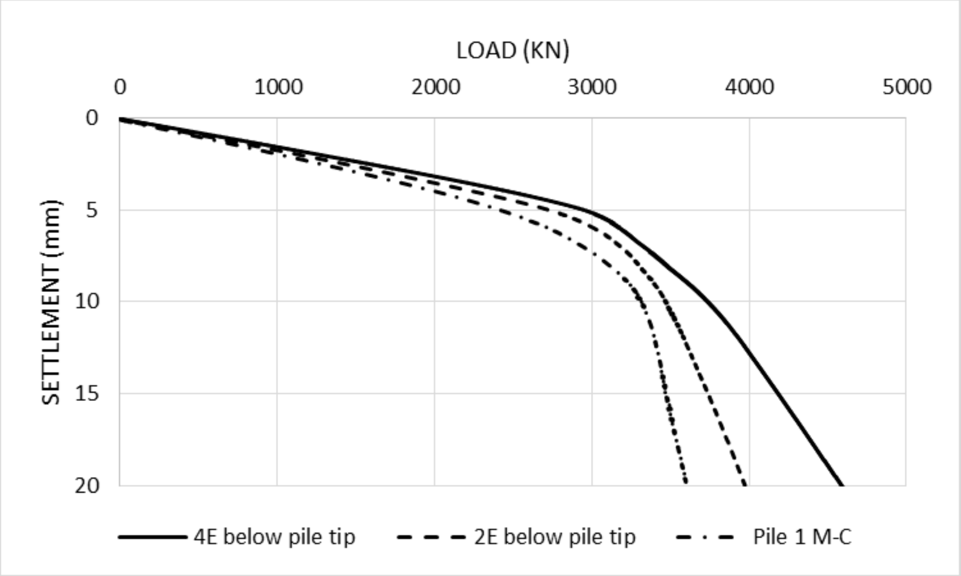

Figure 4.8 Effect of young’s modulus of the soil below the pile tip

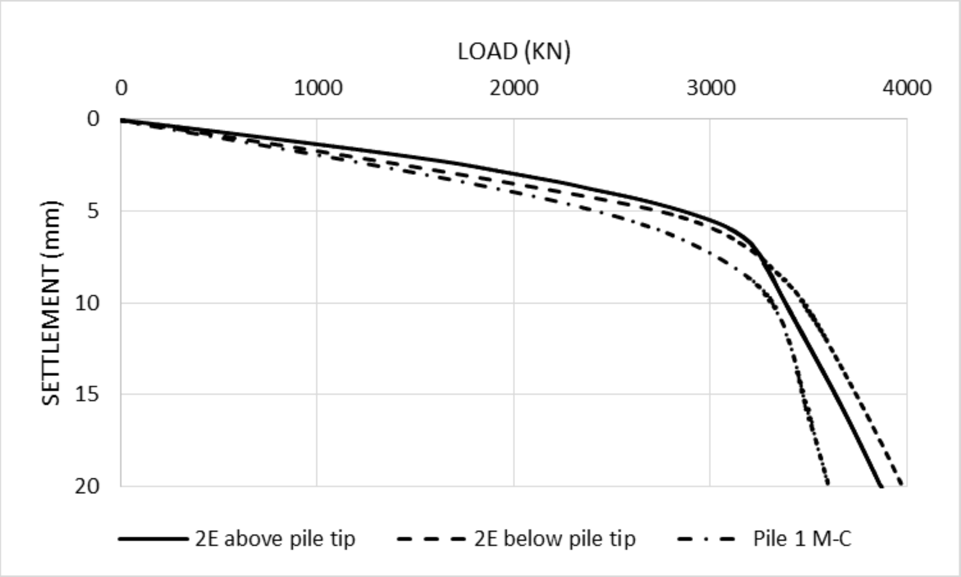

Figure 4.9 Effect of young’s modulus of the soil above and below the base of the pile (2E)

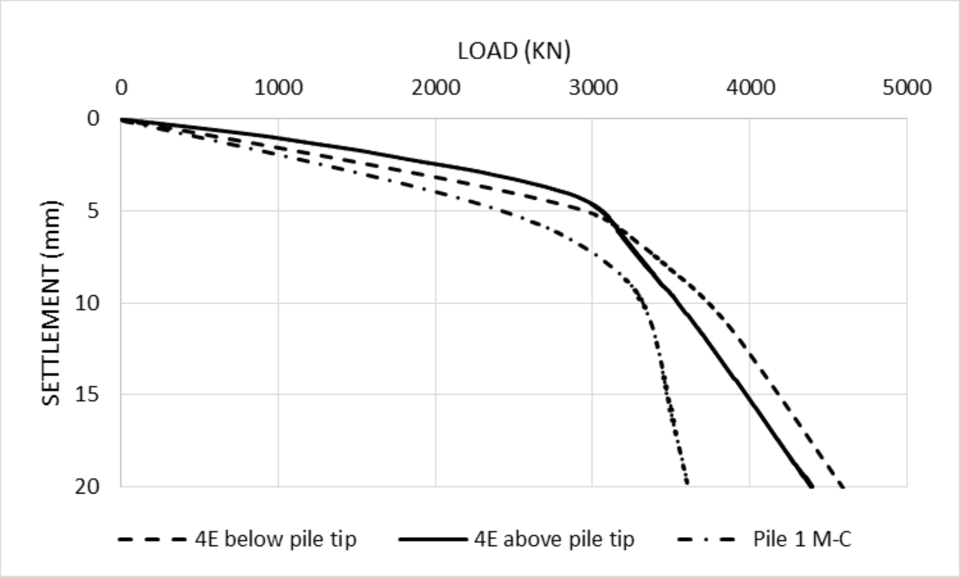

Figure 4.10 The effect of young’s modulus along the pile (4E)

The study results obtained by varying Young's modulus showed that, as the soil stiffness increases, the settlement decreases for the same loading (i.e. less settlement occurs for higher values of E).

4.3.2 Soil Dilatancy

Dilatancy is the change in volume that occurs with shear distortion of material. It is used to describe the tendency of soil to expand or contract in volume upon shearing. It is characterized by a dilation angle, ψ, which is related to the ratio of plastic volume change to plastic shear strain.

In this section, the effect of varying soil dilatancy, (ψ) on the load-settlement curve had been examined. For this analysis, the M-C Nib Bank building project pile no. 1 were selected and the values of ψ were varied from $\psi = 0$ to $\psi = \varphi$.

The curves obtained for different values of ψ are presented in Figure 4.11. In the non-dilatant soil ($\psi = 0$), pile capacity apparently reached its yield and an excessively unconservative nearly elastic response was predicted by the associative flow rule.

When considering associated flow rule with higher dilatancy values, the model will predict continued plastic dilatancy in sustained plastic loading without reaching a clear plunging point. The simulation conducted by setting $\psi = 0$ yields relatively suitable results.

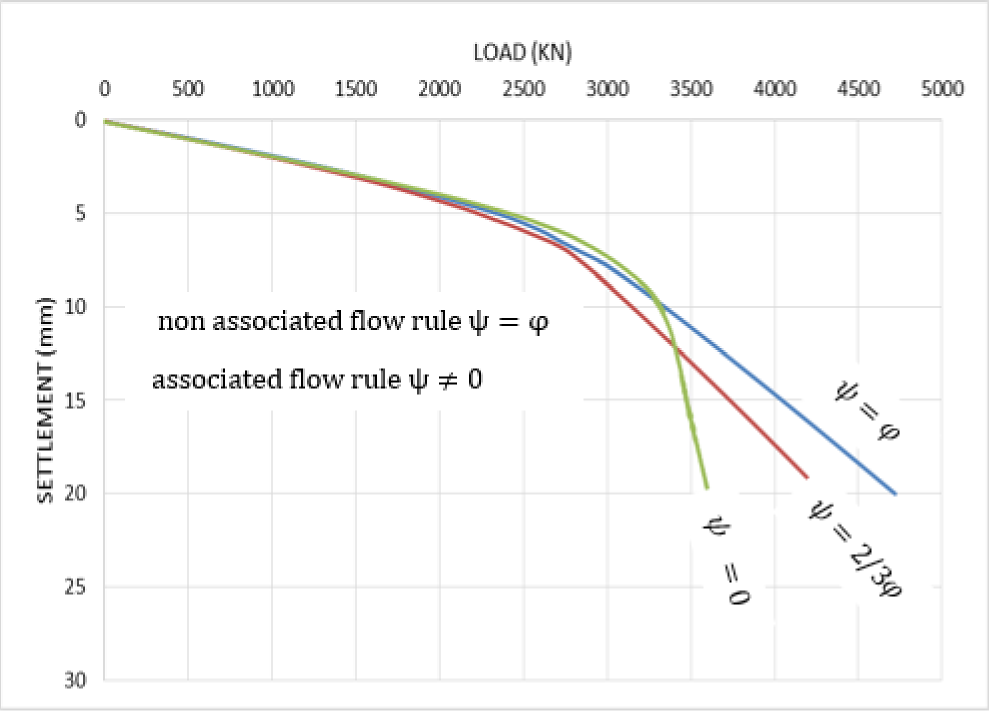

Figure 4.11 The effect of dilation angle on load settlement curve (Nib pile no. 1)

4.3.3 Angle of Internal Friction

The influence of the angle of internal friction on the load-settlement curve was studied by varying the values from φ to 1.5 of φ. The result is presented in Figure 4.12 below.

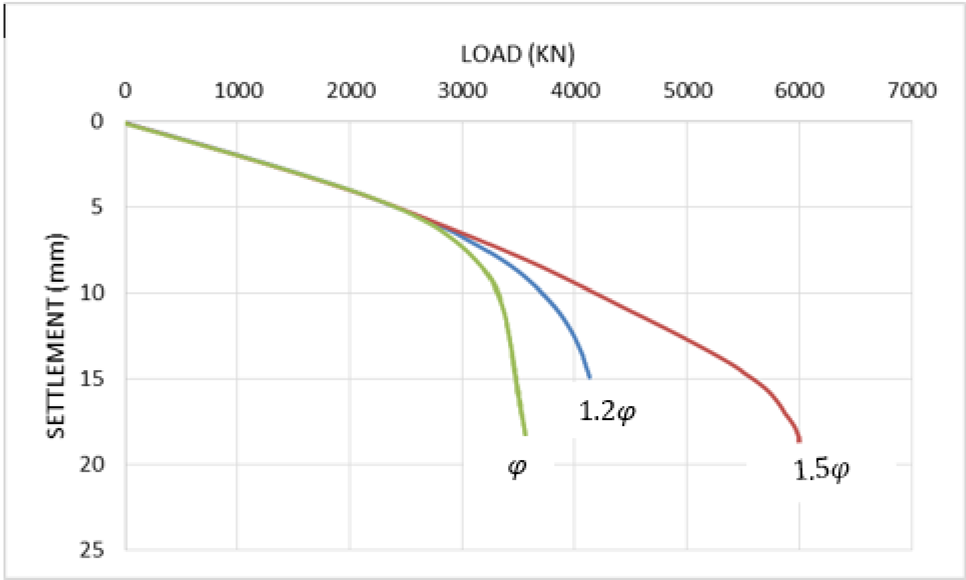

Figure 4.12 The effect of angle of internal friction on load settlement curve(Nib pile no. 1)

For the case in Figure 4.12, no significant change had been shown in the load-settlement behavior in terms of approaching to the actual pile load settlement curve except for ultimate capacity increment. It can be seen that the ultimate pile capacity increases with the increments of shear strength parameter, φ. Thus, using a reasonably higher shear strength parameter value improves the ultimate pile capacity. Since the higher possible values of φ were used in the simulation, the next section further investigates additional parameters that could improve the estimation of ultimate pile capacity.

4.3.4 Coefficient of Lateral Earth Pressure (K)

Figure 4.13 illustrates load-settlement curves generated for different values of lateral earth pressure coefficients. A comparison between the results for different values of the lateral earth pressure coefficient shows similar curves with different plunging points. From previous curves, it was noticed that the ultimate capacity was under predicted. However, the selection of suitable values of K can give better pile capacity estimation (Bowles, 1996). As shown in the figure, the best-matched results are for K of 3 times the value of the lateral pressure coefficient at rest (K_O).

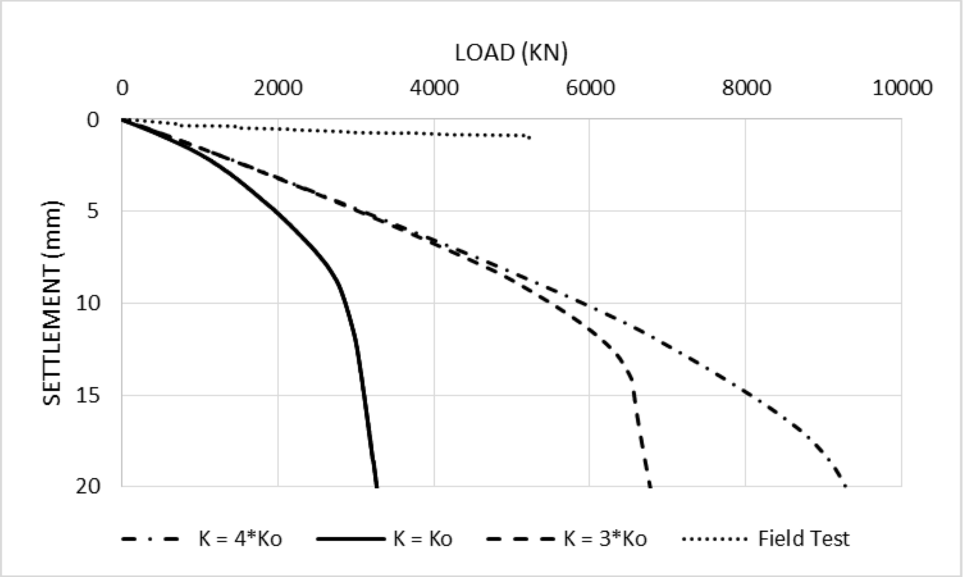

Figure 4.13 Simulation for different values of lateral pressure coefficient (K)

5. Conclusions and Recommendations

5.1 Conclusion

The purpose of the research was to understand the simulation of axially loaded piles in layered soil using a finite difference method based software and to determine the load-settlement relation. Besides, a parameter study has also been performed. The parameter study could help in identifying the extent to which the load settlement curve is affected by various parameters. A comparison was also made between the actual load test results (i.e. considering the loading part only) and the numerical simulation.

The following conclusions were drawn from this research:

- The proposed numerical model has been validated with field data and published results provided by other studies. The latter produced good results with a minor deviation, which might be induced by unspecified interface values in the study and unsuitable young's modulus values.
- In the Numerical simulation, the pile was modeled using a linear elastic model and the soil was modeled using D-P and M-C constitutive models. For this particular study, the results had shown that the M-C model performed well in terms of showing proximity to the actual load-settlement curve.
- The simulation results obtained for united and zemen bank piles correspond well to the field results. However, the discrepancy regarding the load-settlement curves of nib bank piles are due to the generalized soil parameter values employed in the analysis.
- The study confirmed that using finer mesh near high-stress gradient zones yields improved accuracy with less computation time. In addition, the study provides an optimum mesh generation technique for the size and ratio keywords based on the relation: size (3, z-dimension, ¾*y-dimension, 2*x-dimension) and ratio (1, 1, 1, 1.15).
- In this study, nib bank pile 1 was analyzed to determine the transfer of load, and the load carried by the pile base was found to be greater than the skin friction resistance (i.e. 87.5 % of the total pile capacity).

- The ultimate pile capacity was underestimated and using a lateral pressure coefficient of three times the at-rest pressure for nib bank pile 1 produces close results to the actual field data.
- It was noted that the generalized estimates of nib bank pile 1 material parameters were found to underpredict the real behavior, hence further increasing the stiffness values could produce a better load-settlement curve.
- For the type of soil/rock simulated in this particular study, the use of a non-zero dilation angle does not help much in getting a comparable load-settlement curve with the actual field result.

5.2 Recommendation

- Nowadays, numerical analysis is becoming the preferred computational tool in geotechnical structures. Even new codes of practice, like Eurocode 7 encourage the use of numerical design methods. Therefore, finite difference method based tool (FLAC 3D) can be used as an alternative method to simulate pile-loading test with good quality soil data, appropriate constitutive model, and boundary conditions for the determination of load-settlement curve.
- Since FLAC 3D is the ideal tool for parametric analysis, the developed numerical model stored in a notepad can be used to perform an in-depth parametric study.
- The developed 3D numerical model can be compared with the 2D model and the computational time can be further reduced.
- The developed numerical model can be used to compare the relative performance of various pile design alternatives.

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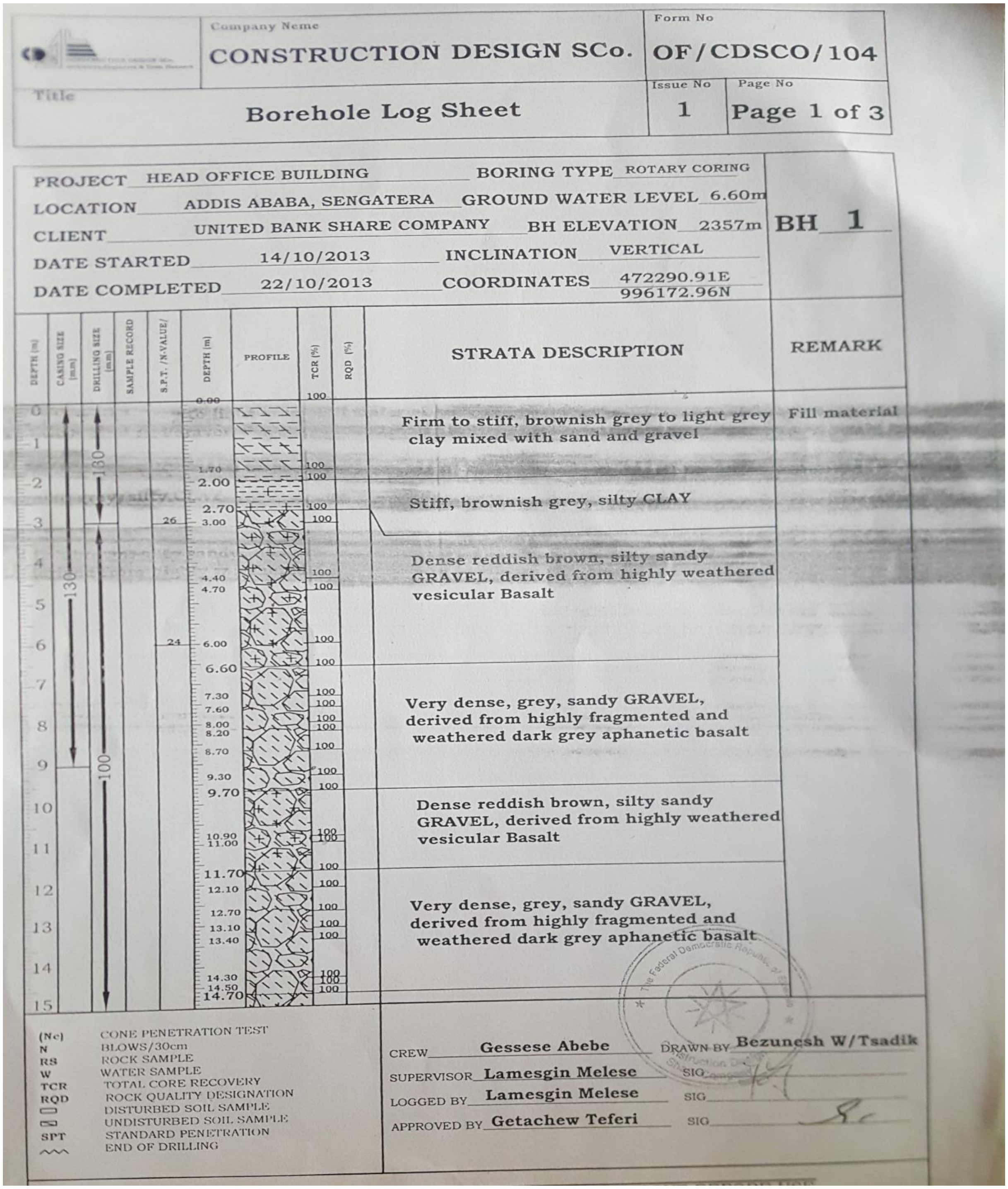
Appendices

Appendix A – Bore hole log data

United bank new headquarter building project



Company Neme

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 6.60m

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

CASING SIZE (mm): 130 (0 – 9 m)

DRILLING SIZE (mm): 180 (0 – 3 m); 100 (3 – 15 m)

DEPTH (m)	S.P.T. /N-VALUE/	TCR (%)	STRATA DESCRIPTION	REMARK
0.00		100	Firm to stiff, brownish grey to light grey clay mixed with sand and gravel	Fill material
1.70		100		
2.00		100	Stiff, brownish grey, silty CLAY	
2.70		100	Dense reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt	
3.00	26	100		
4.40		100		
4.70		100		
6.00	24	100		
6.60		100	Very dense, grey, sandy GRAVEL, derived from highly fragmented and weathered dark grey aphanetic basalt	
7.30		100		
7.60		100		
8.00		100		
8.20		100		
8.70		100		
9.30		100		
9.70		100	Dense reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt	
10.90		100		
11.00		100		
11.70		100	Very dense, grey, sandy GRAVEL, derived from highly fragmented and weathered dark grey aphanetic basalt	
12.10		100		
12.70		100		
13.10		100		
13.40		100		
14.30		100		
14.50		100		
14.70		100		

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

CREW Gessese Abebe

DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Lamesgin Melese

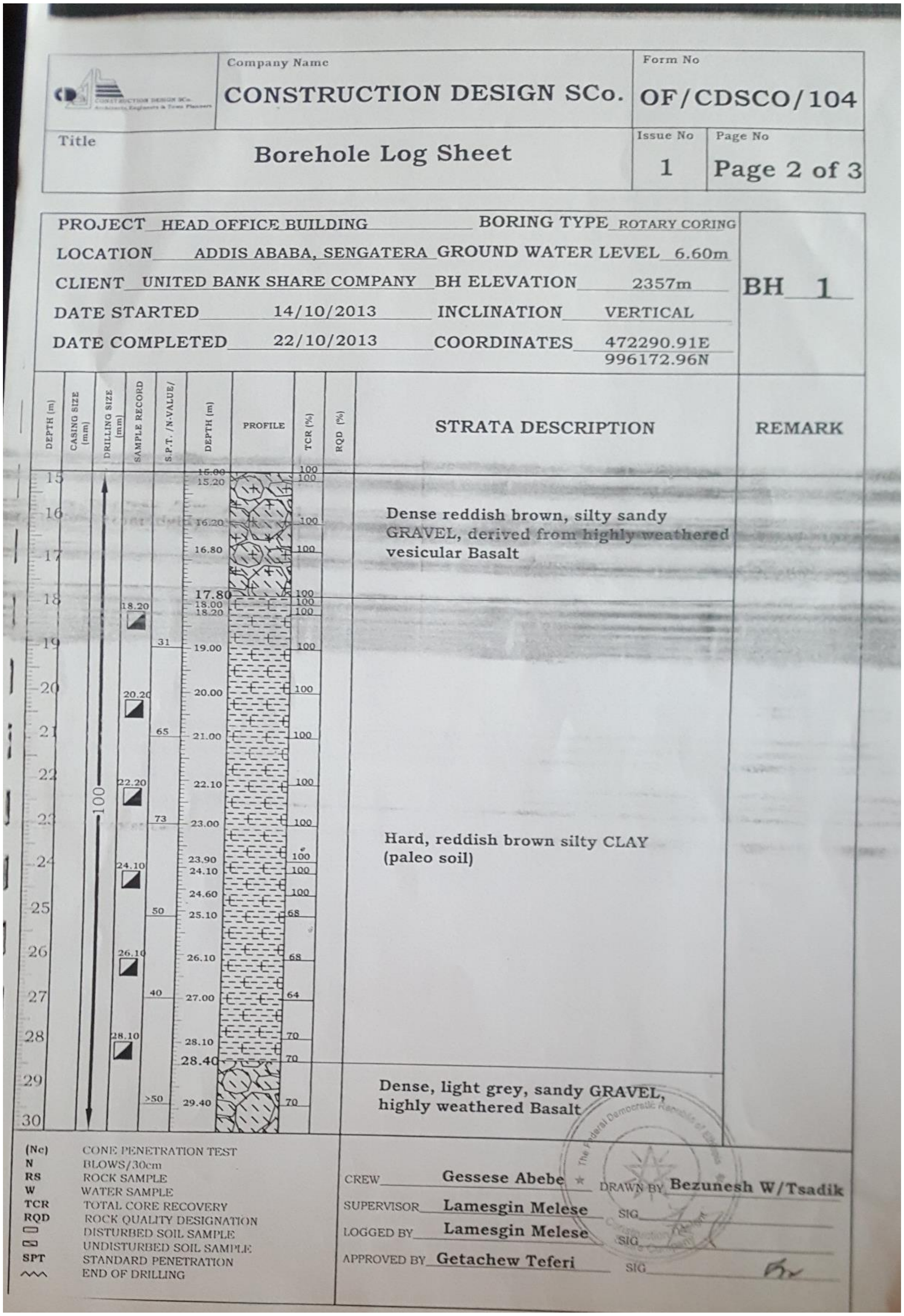
SIG

LOGGED BY Lamesgin Melese

SIG

APPROVED BY Getachew Teferi

SIG



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

Title	Issue No	Page No
Borehole Log Sheet	1	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

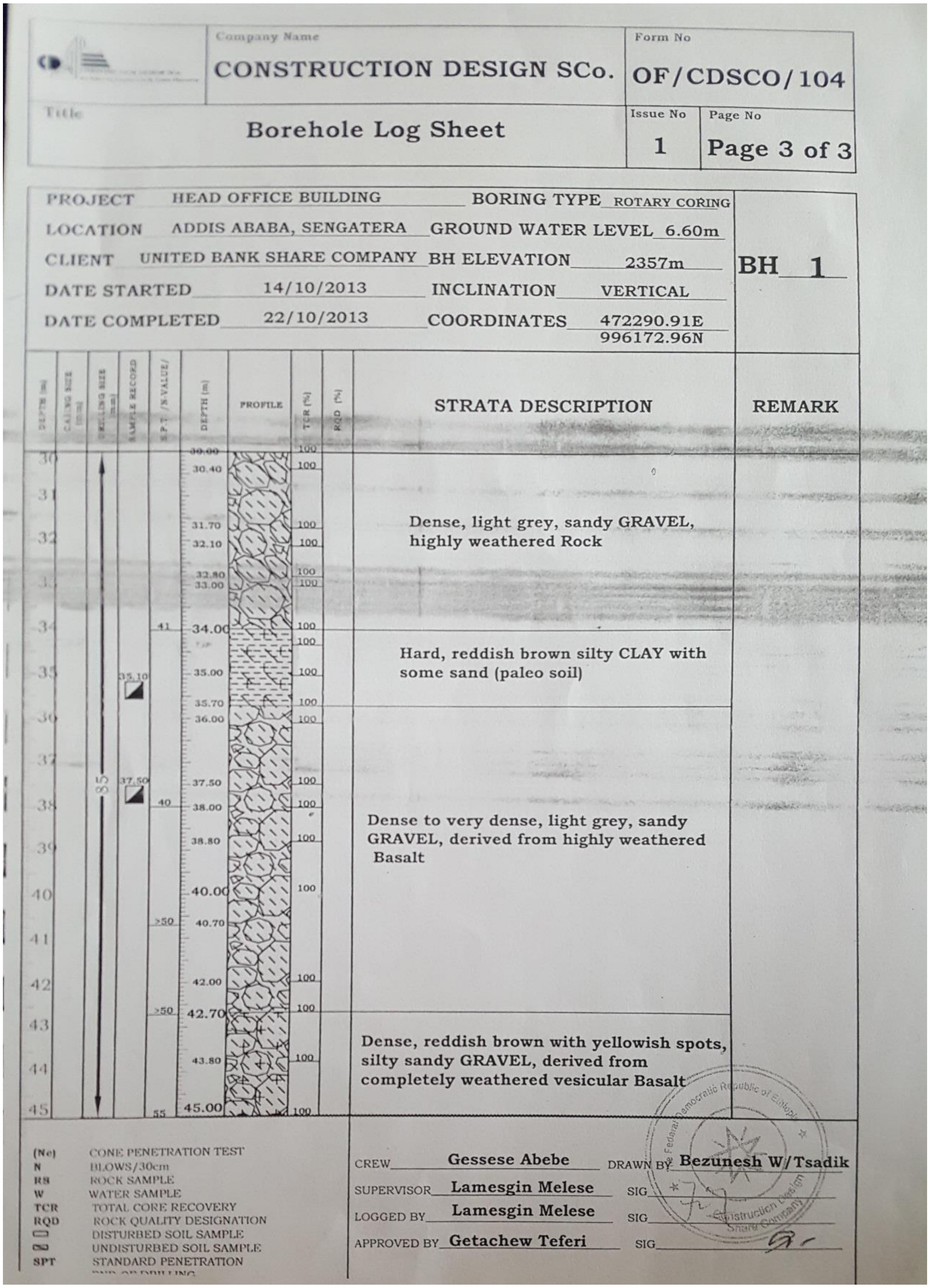
BH 1

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	TCR (%)	RQD (%)
15		100			15.00	100	
					15.20	100	
16					16.20	100	
17					16.80	100	
18					17.80	100	
					18.00	100	
			18.20		18.20	100	
19				31	19.00	100	
20			20.20		20.00	100	
21				65	21.00	100	
22			22.20		22.10	100	
23				73	23.00	100	
					23.90	100	
24			24.10		24.10	100	
					24.60	100	
25				50	25.10	68	
26			26.10		26.10	68	
27				40	27.00	64	
28			28.10		28.10	70	
					28.40	70	
29				>50	29.40	70	
30							

DEPTH (m)	STRATA DESCRIPTION	REMARK
15.00 – 17.80	Dense reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt	
17.80 – 28.40	Hard, reddish brown silty CLAY (paleo soil)	
28.40 –	Dense, light grey, sandy GRAVEL, highly weathered Basalt	

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

CREW Gessese Abebe
DRAWN BY Bezunesh W/Tsadik
SUPERVISOR Lamesgin Melese
SIG
LOGGED BY Lamesgin Melese
SIG
APPROVED BY Getachew Teferi
SIG



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

Title	Issue No	Page No
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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

DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T /N-VALUE/	DEPTH (m)	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
85			30.00	100		Dense, light grey, sandy GRAVEL, highly weathered Rock	
			30.40	100			
			31.70	100			
			32.10	100			
			32.80	100			
			33.00	100			
		41	34.00	100 100		Hard, reddish brown silty CLAY with some sand (paleo soil)	
	35.10		35.00	100			
			35.70	100			
			36.00	100		Dense to very dense, light grey, sandy GRAVEL, derived from highly weathered Basalt	
	37.50		37.50	100			
		40	38.00	100			
			38.80	100			
			40.00	100			
		>50	40.70				
			42.00	100			
		>50	42.70	100		Dense, reddish brown with yellowish spots, silty sandy GRAVEL, derived from completely weathered vesicular Basalt	
			43.80	100			
		55	45.00	100			

(Ne)	CONE PENETRATION TEST
N	BLOWS/30cm
RS	ROCK SAMPLE
W	WATER SAMPLE
TCR	TOTAL CORE RECOVERY
RQD	ROCK QUALITY DESIGNATION
▭	DISTURBED SOIL SAMPLE
▭	UNDISTURBED SOIL SAMPLE
SPT	STANDARD PENETRATION

CREW Gessese Abebe

DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Lamesgin Melese

SIG

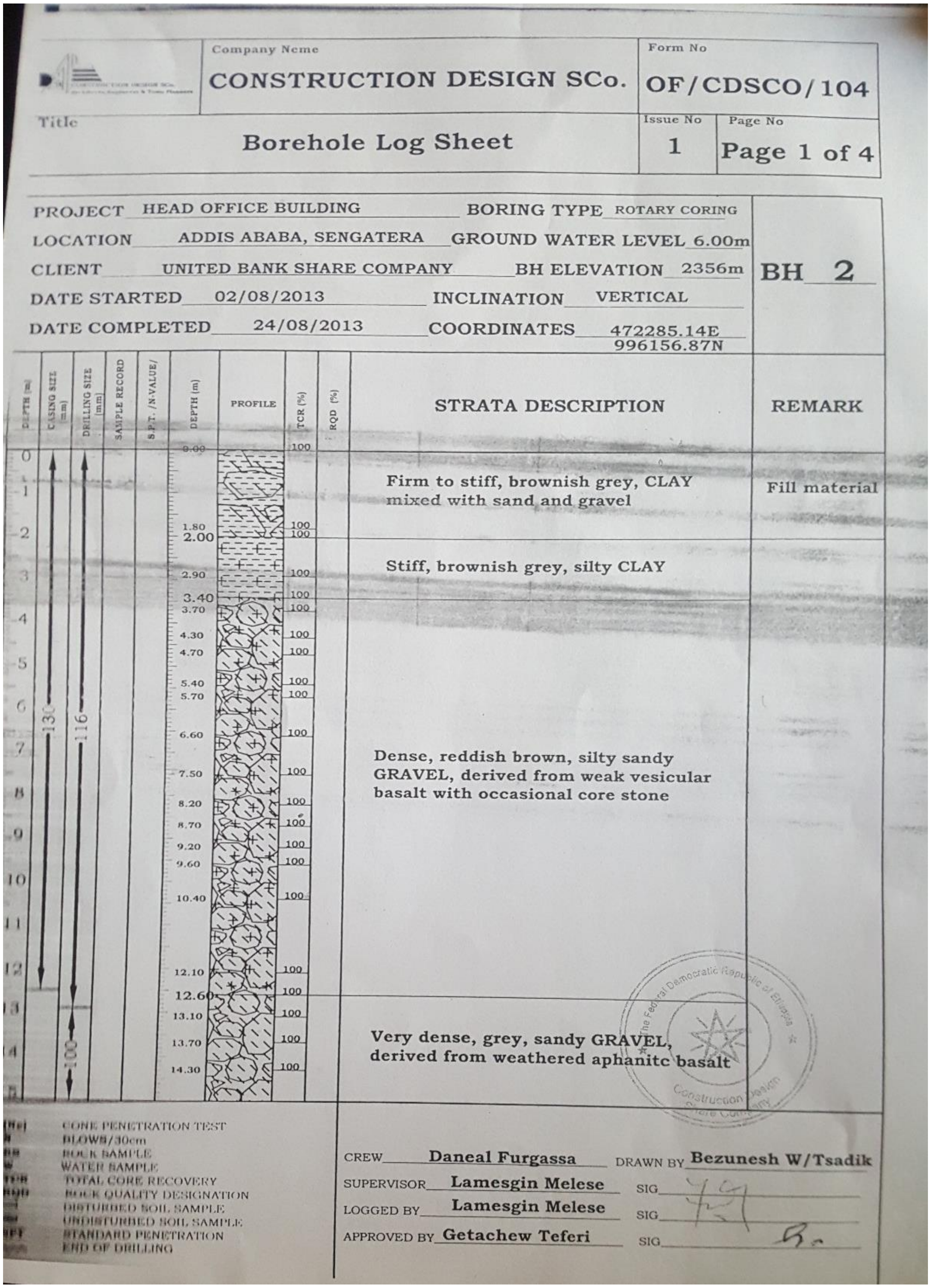
LOGGED BY Lamesgin Melese

SIG

APPROVED BY Getachew Teferi

SIG

Construction Design Share Company



Company Neme **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 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

CASING SIZE (mm): 130

DRILLING SIZE (mm): 116, 100

DEPTH (m)	TCR (%)	STRATA DESCRIPTION	REMARK
0.00	100	Firm to stiff, brownish grey, CLAY mixed with sand and gravel	Fill material
1.80	100		
2.00	100	Stiff, brownish grey, silty CLAY	
2.90	100		
3.40	100	Dense, reddish brown, silty sandy GRAVEL, derived from weak vesicular basalt with occasional core stone	
3.70	100		
4.30	100		
4.70	100		
5.40	100		
5.70	100		
6.60	100		
7.50	100		
8.20	100		
8.70	100		
9.20	100		
9.60	100		
10.40	100		
12.10	100		
12.60	100	Very dense, grey, sandy GRAVEL, derived from weathered aphanitc basalt	
13.10	100		
13.70	100		
14.30	100		

CONE PENETRATION TEST
BLOWS/30cm
ROCK SAMPLE
WATER SAMPLE
TOTAL CORE RECOVERY
ROCK QUALITY DESIGNATION
DISTURBED SOIL SAMPLE
UNDISTURBED SOIL SAMPLE
STANDARD PENETRATION
END OF DRILLING

CREW Daneal Furgassa

DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Lamesgin Melese

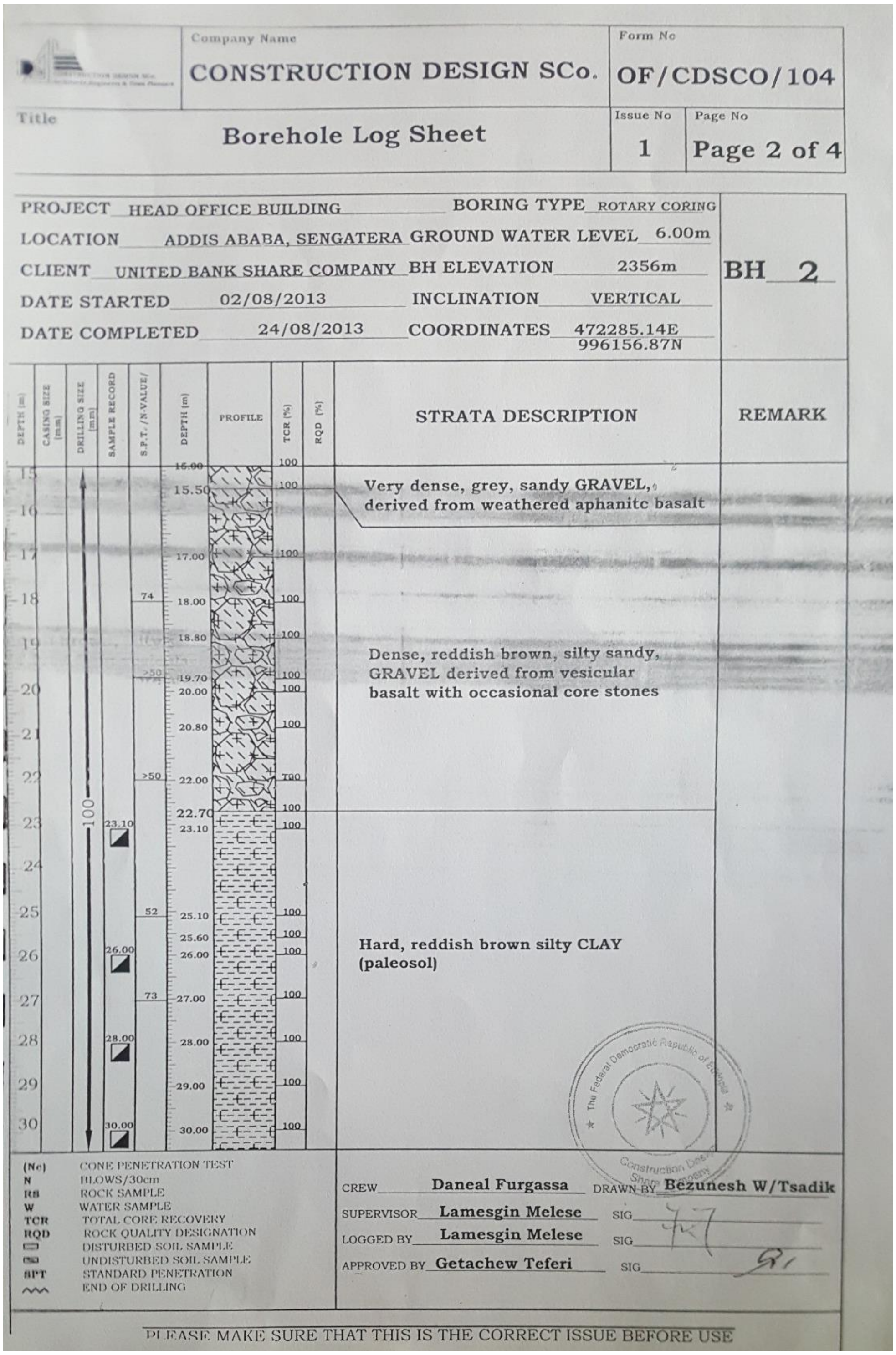
SIG

LOGGED BY Lamesgin Melese

SIG

APPROVED BY Getachew Teferi

SIG



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

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 (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
							100			
15		100			16.00		100			
					15.50		100		Very dense, grey, sandy GRAVEL, derived from weathered aphanitc basalt	
16										
17					17.00		100			
18				74	18.00		100			
19					18.80		100		Dense, reddish brown, silty sandy, GRAVEL derived from vesicular basalt with occasional core stones	
				>50	19.70		100			
20					20.00		100			
21					20.80		100			
22				>50	22.00		100			
					22.70		100			
23			23.10 ◩		23.10		100		Hard, reddish brown silty CLAY (paleosol)	
24										
25				52	25.10		100			
					25.60		100			
26			26.00 ◩		26.00		100			
27				73	27.00		100			
28			28.00 ◩		28.00		100			
29					29.00		100			
30			30.00 ◩		30.00		100			

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

CREW Daneal Furgassa

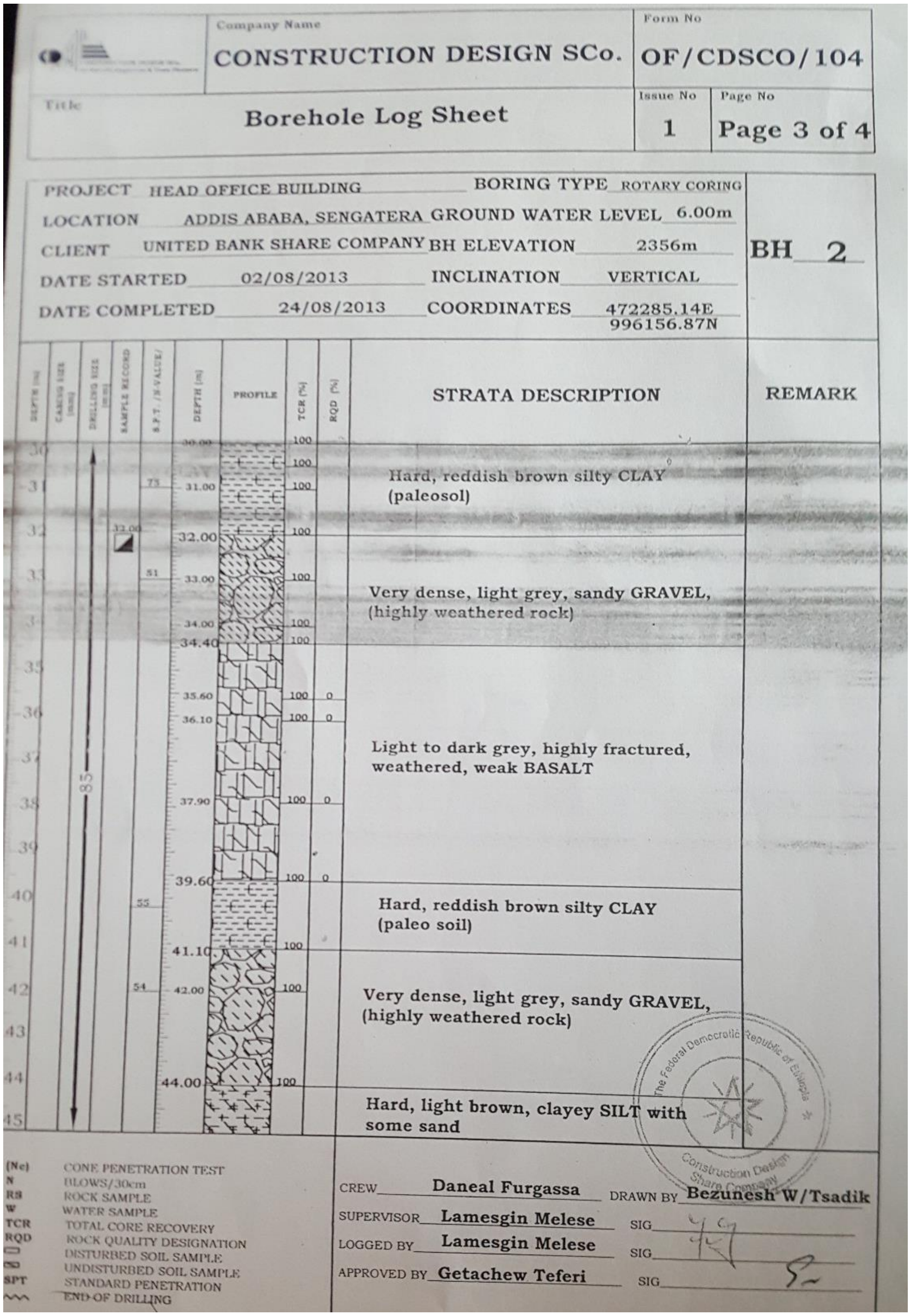
DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Lamesgin Melese SIG

LOGGED BY Lamesgin Melese SIG

APPROVED BY Getachew Teferi SIG

PLEASE MAKE SURE THAT THIS IS THE CORRECT ISSUE BEFORE USE



Company Name
CONSTRUCTION DESIGN SCo.

Form No
OF/CDSCO/104

Title
Borehole Log Sheet

Issue No **1**
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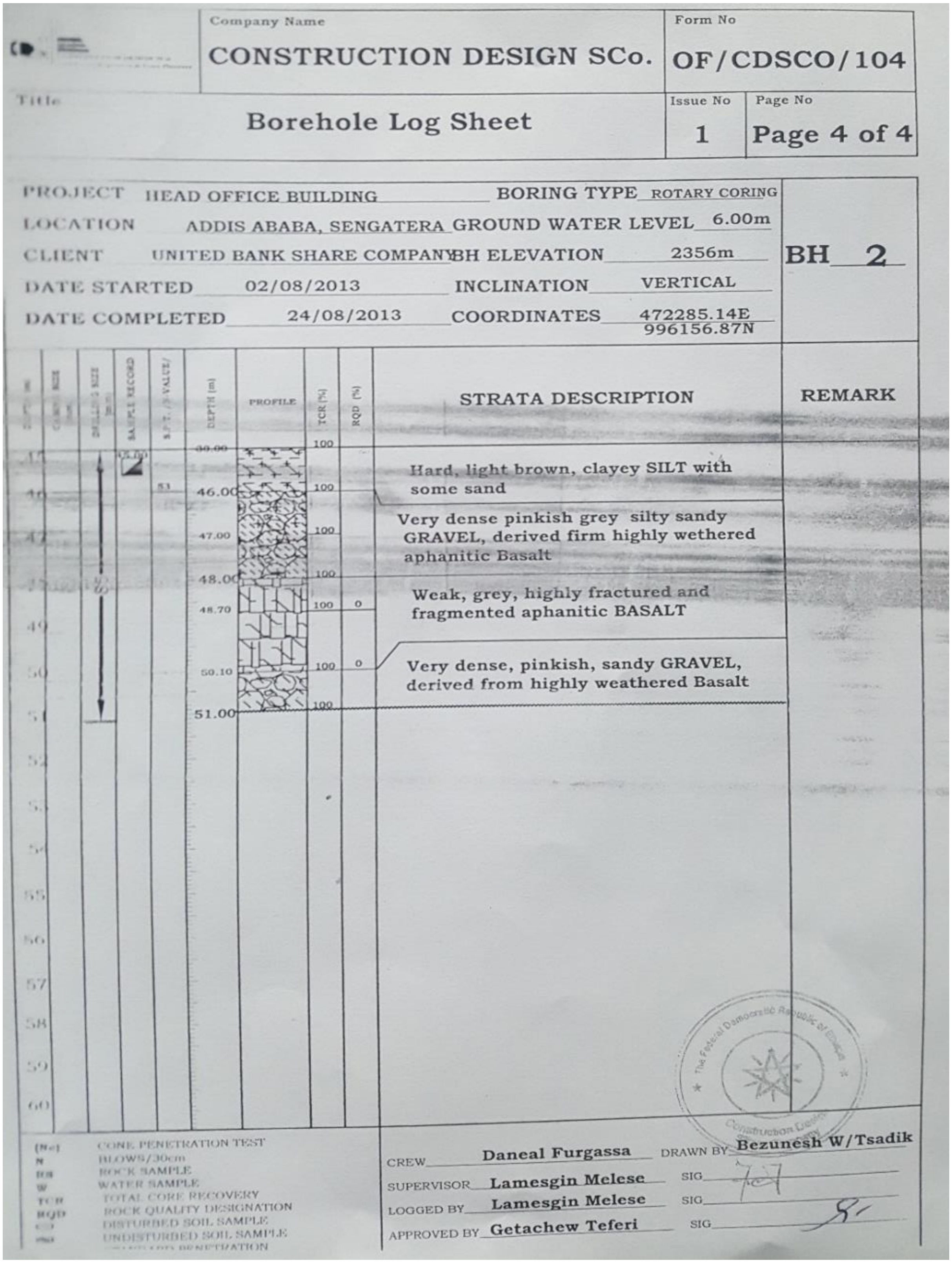
PROJECT HEAD OFFICE BUILDING
BORING TYPE ROTARY CORING
LOCATION ADDIS ABABA, SENGATERA
GROUND WATER LEVEL 6.00m
CLIENT UNITED BANK SHARE COMPANY
BH ELEVATION 2356m
DATE STARTED 02/08/2013
INCLINATION VERTICAL
DATE COMPLETED 24/08/2013
COORDINATES 472285.14E 996156.87N

BH 2

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
30		85			30.00		100			
							100			
31				73	31.00		100		Hard, reddish brown silty CLAY (paleosol)	
32			32.00 ◪		32.00		100			
33				51	33.00		100		Very dense, light grey, sandy GRAVEL, (highly weathered rock)	
34					34.00		100			
					34.40		100			
35										
					35.60		100	0		
36					36.10		100	0		
37									Light to dark grey, highly fractured, weathered, weak BASALT	
38					37.90		100	0		
39										
					39.60		100	0		
40				55					Hard, reddish brown silty CLAY (paleo soil)	
41					41.10		100			
42				54	42.00		100		Very dense, light grey, sandy GRAVEL, (highly weathered rock)	
43										
44					44.00		100			
45									Hard, light brown, clayey SILT with some sand	

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

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



Company Name

CONSTRUCTION DESIGN SCo.

Form No **OF/CDSCO/104**

Title

Borehole Log Sheet

Issue No **1**

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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 (m)	S.P.T. (N VALUE)	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
45.00		100		Hard, light brown, clayey SILT with some sand	
46.00	51	100		Very dense pinkish grey silty sandy GRAVEL, derived firm highly wethered aphanitic Basalt	
47.00		100			
48.00		100		Weak, grey, highly fractured and fragmented aphanitic BASALT	
48.70		100	0		
50.10		100	0	Very dense, pinkish, sandy GRAVEL, derived from highly weathered Basalt	
51.00		100			

(N_c) CONE PENETRATION TEST
N BLOWS/30cm
RS ROCK SAMPLE
W WATER SAMPLE
TCR TOTAL CORE RECOVERY
RQD ROCK QUALITY DESIGNATION
DISTURBED SOIL SAMPLE
UNDISTURBED SOIL SAMPLE
PENETRATION

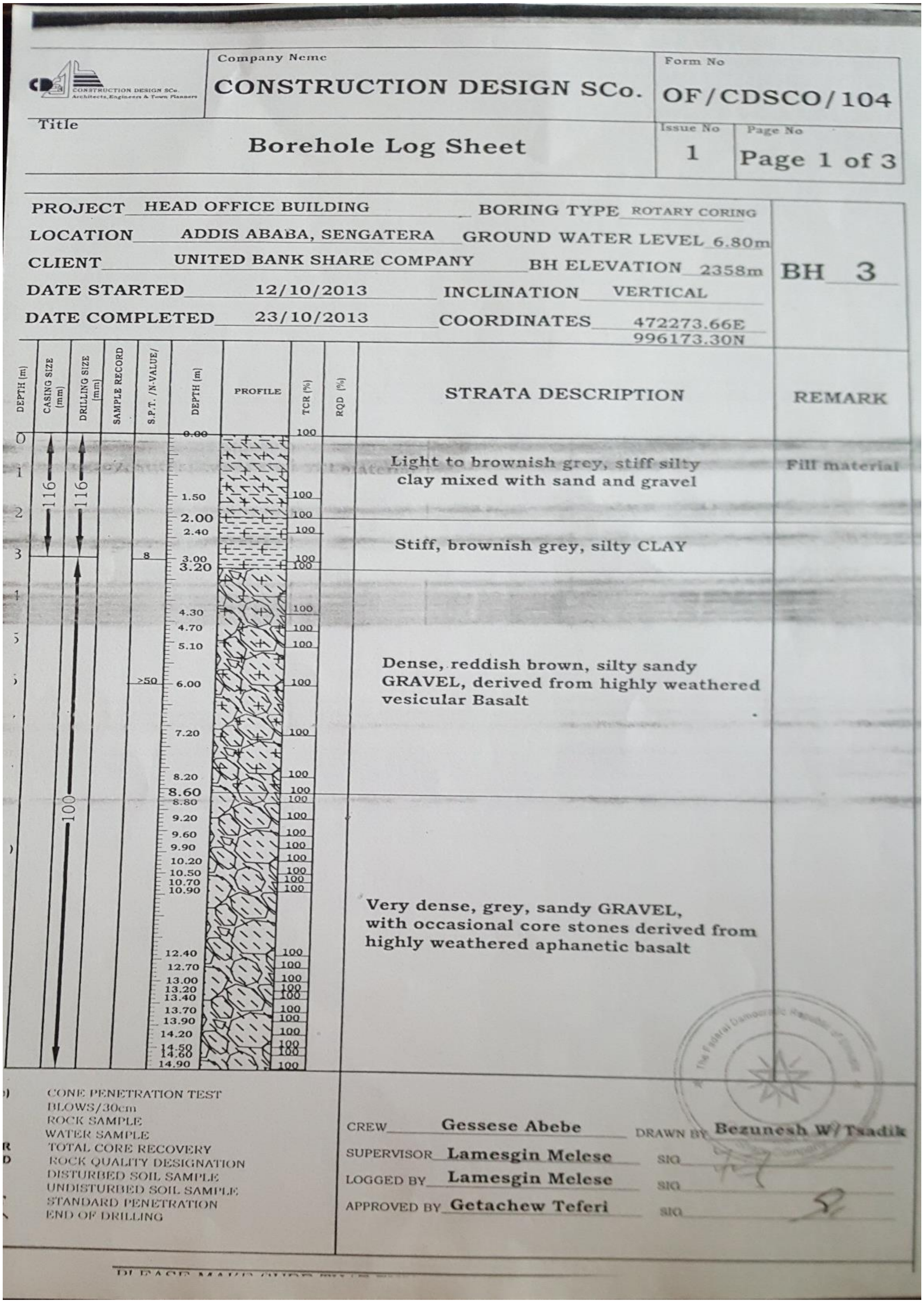
CREW Daneal Furgassa

DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Lamesgin Melese SIG

LOGGED BY Lamesgin Melese SIG

APPROVED BY Getachew Teferi SIG



CONSTRUCTION DESIGN SCo.
Architects, Engineers & Town Planners

Company Neme: **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

BH 3

DATE STARTED: 12/10/2013

INCLINATION: VERTICAL

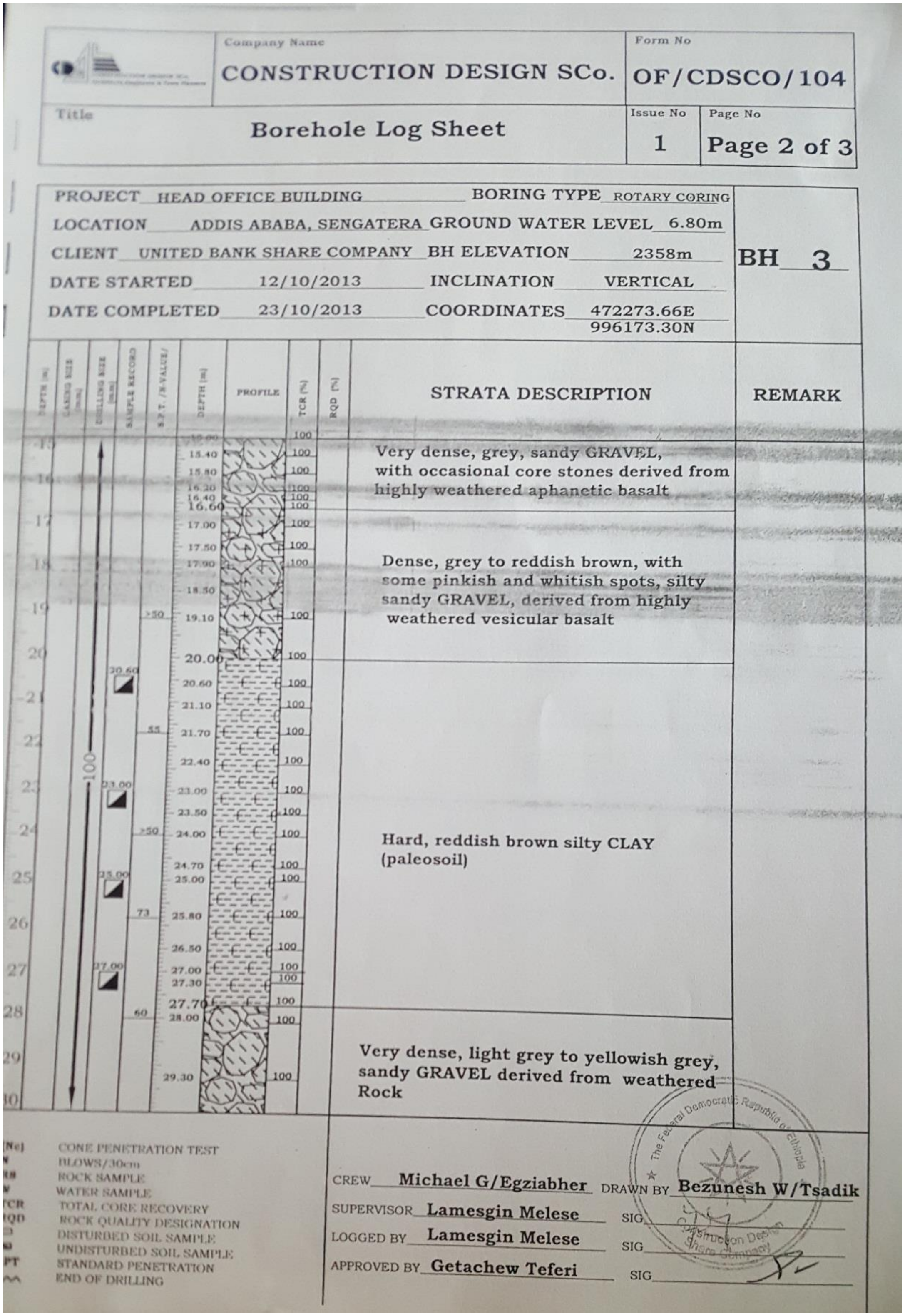
DATE COMPLETED: 23/10/2013

COORDINATES: 472273.66E
996173.30N

CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
116	116			0.00	100		Light to brownish grey, stiff silty clay mixed with sand and gravel	Fill material
				1.50	100			
				2.00	100		Stiff, brownish grey, silty CLAY	
				2.40	100			
	100		8	3.00	100			
				3.20	100		Dense, reddish brown, silty sandy GRAVEL, derived from highly weathered vesicular Basalt	
				4.30	100			
				4.70	100			
				5.10	100			
			>50	6.00	100			
				7.20	100			
				8.20	100			
				8.60	100		Very dense, grey, sandy GRAVEL, with occasional core stones derived from highly weathered aphanetic basalt	
				8.80	100			
				9.20	100			
				9.60	100			
				9.90	100			
				10.20	100			
				10.50	100			
				10.70	100			
				10.90	100			
				12.40	100			
				12.70	100			
				13.00	100			
				13.20	100			
				13.40	100			
				13.70	100			
				13.90	100			
				14.20	100			
				14.50	100			
				14.60	100			
				14.90	100			

CONE PENETRATION TEST
BLOWS/30cm
ROCK SAMPLE
WATER SAMPLE
TOTAL CORE RECOVERY
ROCK QUALITY DESIGNATION
DISTURBED SOIL SAMPLE
UNDISTURBED SOIL SAMPLE
STANDARD PENETRATION
END OF DRILLING

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



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

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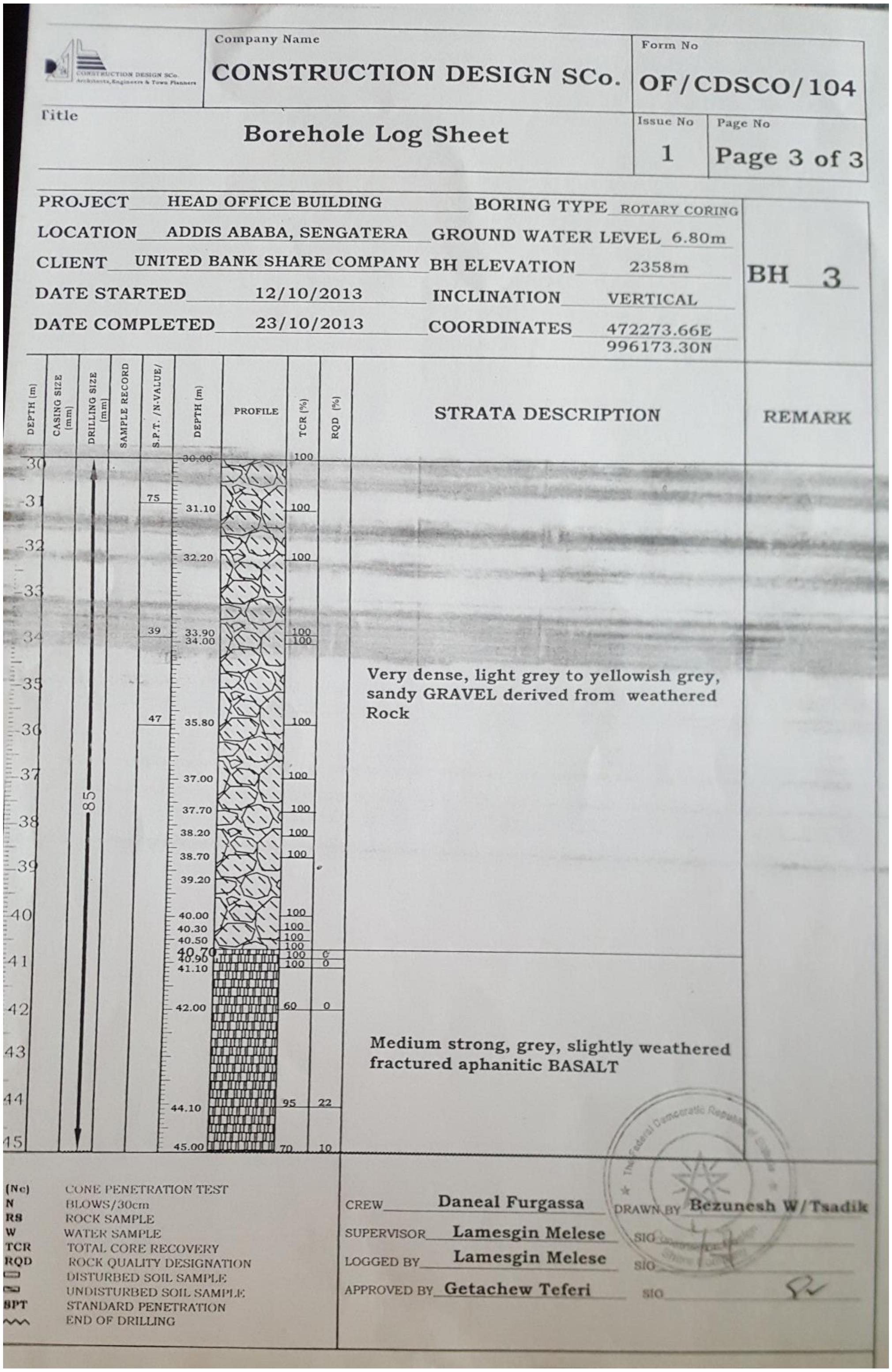
PROJECT HEAD OFFICE BUILDING
BORING TYPE ROTARY CORING
LOCATION ADDIS ABABA, SENGATERA
GROUND WATER LEVEL 6.80m
CLIENT UNITED BANK SHARE COMPANY
BH ELEVATION 2358m
DATE STARTED 12/10/2013
INCLINATION VERTICAL
DATE COMPLETED 23/10/2013
COORDINATES 472273.66E
996173.30N

BH 3

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
15		100			15.00		100		Very dense, grey, sandy GRAVEL, with occasional core stones derived from highly weathered aphanetic basalt	
					15.40		100			
					15.80		100			
16										
					16.20		100			
					16.40		100			
					16.60		100		Dense, grey to reddish brown, with some pinkish and whitish spots, silty sandy GRAVEL, derived from highly weathered vesicular basalt	
17					17.00		100			
					17.50		100			
					17.90		100			
18										
					18.50					
19				>50	19.10		100			
20					20.00		100		Hard, reddish brown silty CLAY (paleosoil)	
			20.60		20.60		100			
21					21.10		100			
				55	21.70		100			
22					22.40		100			
23			23.00		23.00		100			
					23.50		100			
24				>50	24.00		100			
					24.70		100			
25			25.00		25.00		100			
				73	25.80		100			
26										
					26.50		100			
27			27.00		27.00		100			
					27.30		100			
					27.70		100		Very dense, light grey to yellowish grey, sandy GRAVEL derived from weathered Rock	
28				60	28.00		100			
29					29.30		100			
30										

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

CREW Michael G/Egziabher
DRAWN BY Bezunesh W/Tsadik
SUPERVISOR Lamesgin Melese
SIG
LOGGED BY Lamesgin Melese
SIG
APPROVED BY Getachew Teferi
SIG



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

CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
	85			30.00	100		Very dense, light grey to yellowish grey, sandy GRAVEL derived from weathered Rock	
			75	31.10	100			
				32.20	100			
			39	33.90	100			
				34.00	100			
			47	35.80	100			
				37.00	100			
				37.70	100			
				38.20	100			
				38.70	100			
				39.20				
				40.00	100			
				40.30	100			
				40.50	100			
				40.70	100		Medium strong, grey, slightly weathered fractured aphanitic BASALT	
				40.90	100	0		
				41.10	100	0		
				42.00	60	0		
				44.10	95	22		
				45.00	70	10		

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

CREW Daneal Furgassa

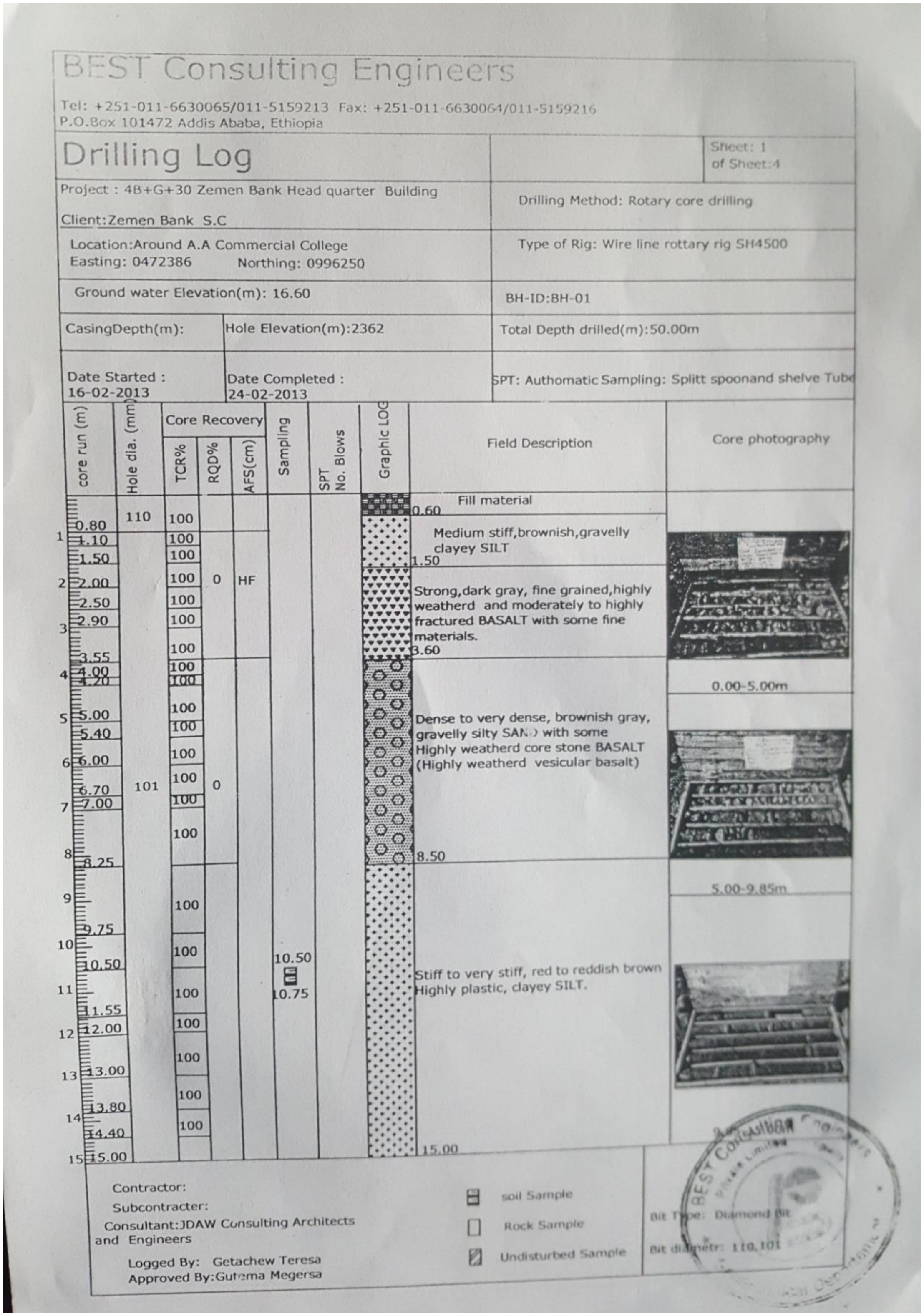
DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Lamesgin Melese SIG

LOGGED BY Lamesgin Melese SIG

APPROVED BY Getachew Teferi SIG

Zemen bank new headquarter building project



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
Client: Zemen Bank S.C
Drilling Method: Rotary core drilling
Location: Around A.A Commercial College
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 Tub

core run (m)	Hole dia. (mm)	Core Recovery TCR%	Core Recovery RQD%	Core Recovery AFS(cm)	Sampling	SPT No. Blows
0.80	110	100				
1.10	101	100	0	HF		
1.50		100				
2.00		100				
2.50		100				
2.90		100				
3.55		100				
4.00		100	0			
4.20		100				
5.00		100				
5.40		100				
6.00		100				
6.70		100				
7.00		100				
8.25		100				
9.75		100				
10.50		100				
11.55		100			10.50 ⊟ 10.75	
12.00		100				
13.00		100				
13.80		100				
14.40		100				
15.00						

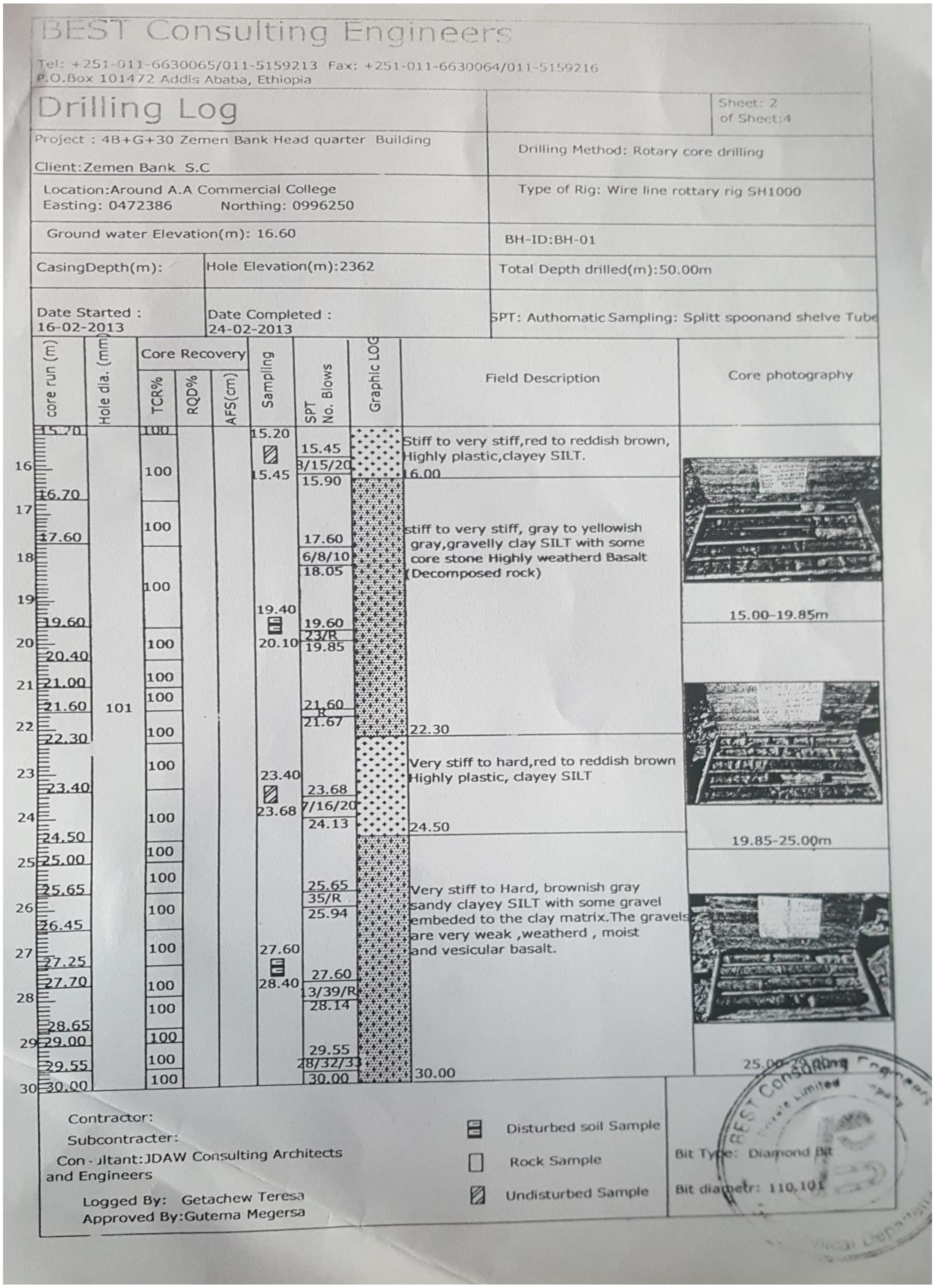
Graphic LOG	Field Description
0.60	Fill material
1.50	Medium stiff,brownish,gravelly clayey SILT
3.60	Strong,dark gray, fine grained,highly weatherd and moderately to highly fractured BASALT with some fine materials.
8.50	Dense to very dense, brownish gray, gravelly silty SAND with some Highly weatherd 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:
Subcontracter:
Consultant:JDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By:Gutema Megersa

⊟ soil Sample
☐ Rock Sample
▨ Undisturbed Sample

Bit Type: Diamond Bit
Bit diametr: 110,101



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Drilling Log

Sheet: 2
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: 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

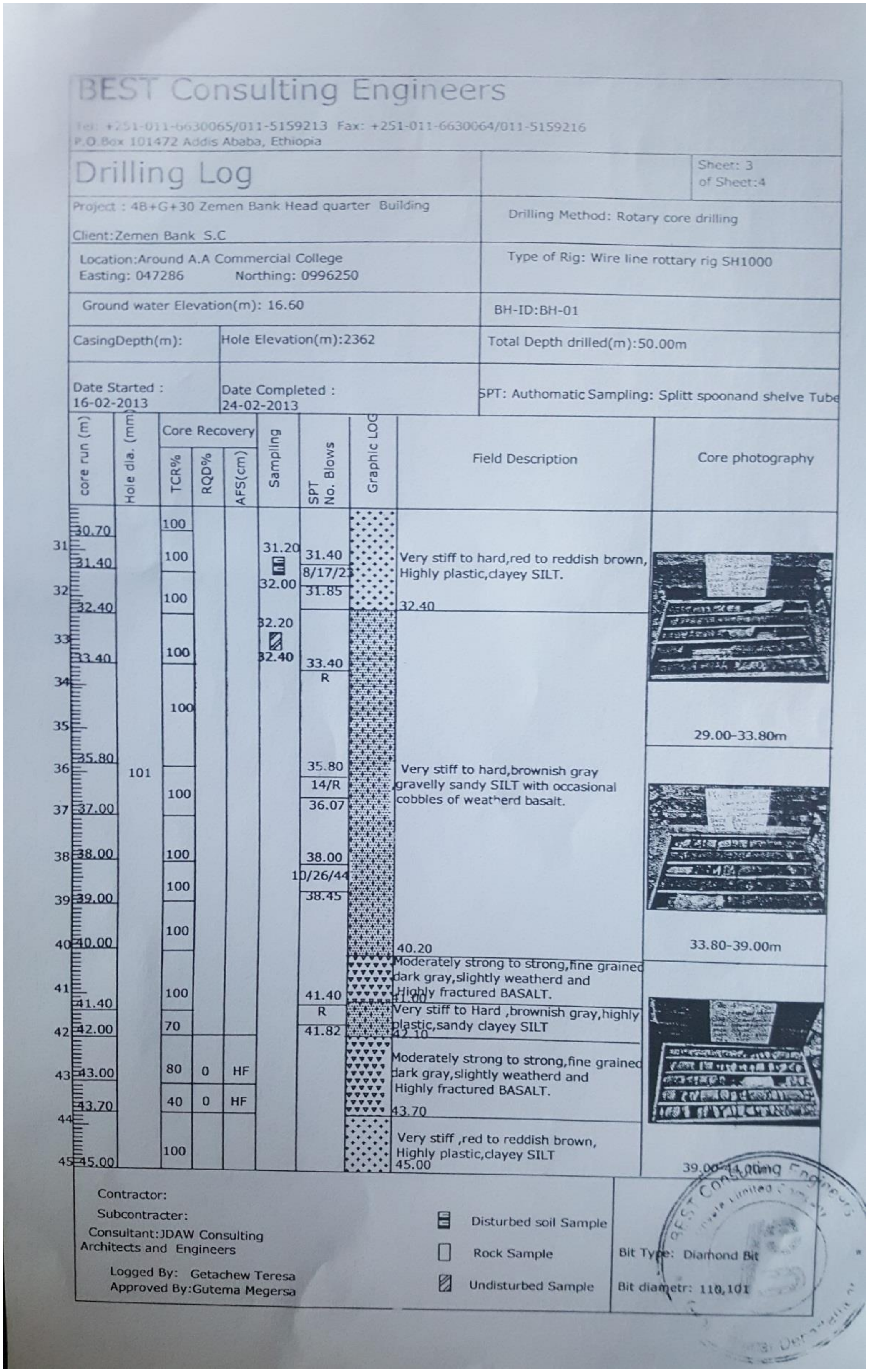
SPT: Authomatic Sampling: Splitt spoonand shelve Tube

core run (m)	Hole dia. (mm)	Core Recovery TCR%	RQD%	AFS(cm)	Sampling	SPT No. Blows	Graphic LOG	Field Description	Core photography
15.20		100			15.20				
16.70		100			▨ 15.45	15.45 3/15/20 15.90		Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. 16.00	
17.60		100							
19.60		100			19.40 ▤	17.60 6/8/10 18.05		stiff to very stiff, gray to yellowish gray,gravelly clay SILT with some core stone Highly weatherd Basalt (Decomposed rock)	15.00-19.85m
20.40		100			20.10	19.60 23/R 19.85			
21.00		100							
21.60	101	100							
22.30		100				21.60 R 21.67		22.30	
23.40		100			23.40			Very stiff to hard,red to reddish brown Highly plastic, clayey SILT	
24.50		100			▨ 23.68	23.68 7/16/20 24.13		24.50	19.85-25.00m
25.00		100							
25.65		100							
26.45		100				25.65 35/R 25.94		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.	
27.25		100							
27.70		100			27.60 ▤				
28.65		100			28.40	27.60 13/39/R 28.14			
29.00		100							
29.55		100							
30.00		100				29.55 28/32/3 30.00		30.00	25.00-[illegible]

Contractor:
Subcontracter:
Con ultant: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 diametr: 110,101



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Drilling Log

Sheet: 3
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: 047286 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

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
30.70		100							
31.40		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
32.40		100							
33.40		100			32.20 ▨ 32.40	33.40 R			
35.80		100							
37.00	101	100				35.80 14/R 36.07		Very stiff to hard,brownish gray gravelly sandy SILT with occasional cobbles of weatʰerd basalt.	
38.00		100				38.00 10/26/44 38.45			33.80-39.00m
39.00		100							
40.00		100						40.20	
41.40		100				41.40 R 41.82		Moderately strong to strong,fine grained dark gray,slightly weatherd and Highly fractured BASALT. 41.00 Very stiff to Hard ,brownish gray,highly plastic,sandy clayey SILT 42.10	
42.00		70							
43.00		80	0	HF				Moderately strong to strong,fine grained dark gray,slightly weatherd and Highly fractured BASALT. 43.70	
43.70		40	0	HF					
45.00		100						Very stiff ,red to reddish brown, Highly plastic,clayey SILT 45.00	39.00-44.00m

Contractor:

Subcontracter:

Consultant:JDAW Consulting Architects and Engineers

Logged By: Getachew Teresa
Approved By:Gutema Megersa

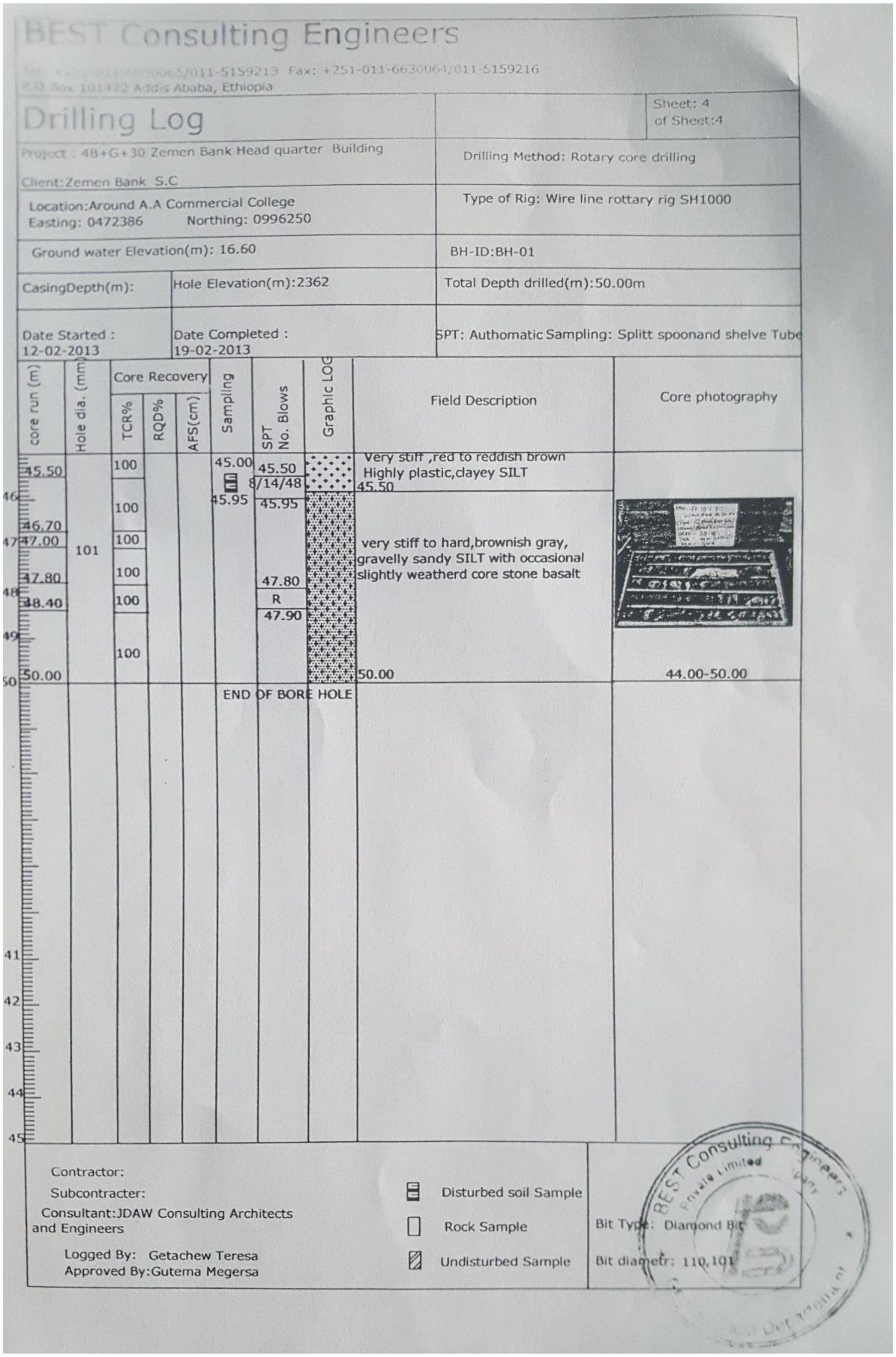
▤ Disturbed soil Sample

☐ Rock Sample

▨ Undisturbed Sample

Bit Type: Diamond Bit

Bit diametr: 110,101



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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: 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: Splitt spoonand shelve Tube

core run (m)	Hole dia. (mm)	Core Recovery			Sampling	SPT No. Blows	Graphic LOG	Field Description	Core photography
		TCR%	RQD%	AFS(cm)					
45.50	101	100			45.00 ▤ 45.95	45.50 8/14/48 45.95		Very stiff ,red to reddish brown Highly plastic,clayey SILT 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 R 47.90			
48.40		100							
50.00		100						50.00	44.00-50.00

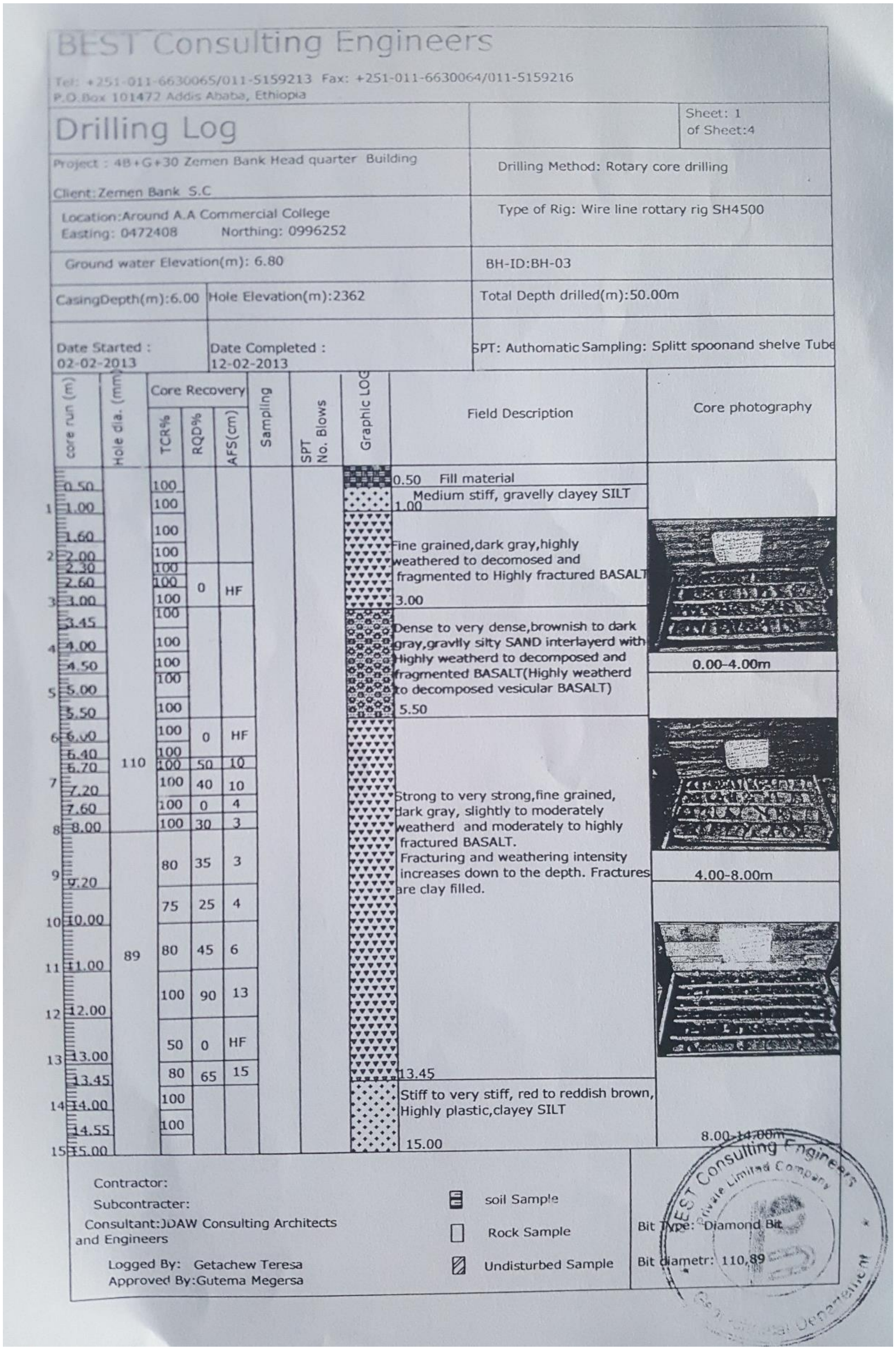
END OF BORE HOLE

Contractor:
Subcontracter:
Consultant:JDAW Consulting Architects and Engineers

Logged By: Getachew Teresa
Approved By:Gutema Megersa

▤	Disturbed soil Sample
▯	Rock Sample
▨	Undisturbed Sample

Bit Type: Diamond Bit
Bit diametr: 110,101



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Drilling Log

Sheet: 1
of Sheet:4

Project : 4B+G+30 Zemen Bank Head quarter Building

Client:Zemen Bank S.C

Location:Around A.A Commercial College
Easting: 0472408 Northing: 0996252

Drilling Method: Rotary core drilling

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

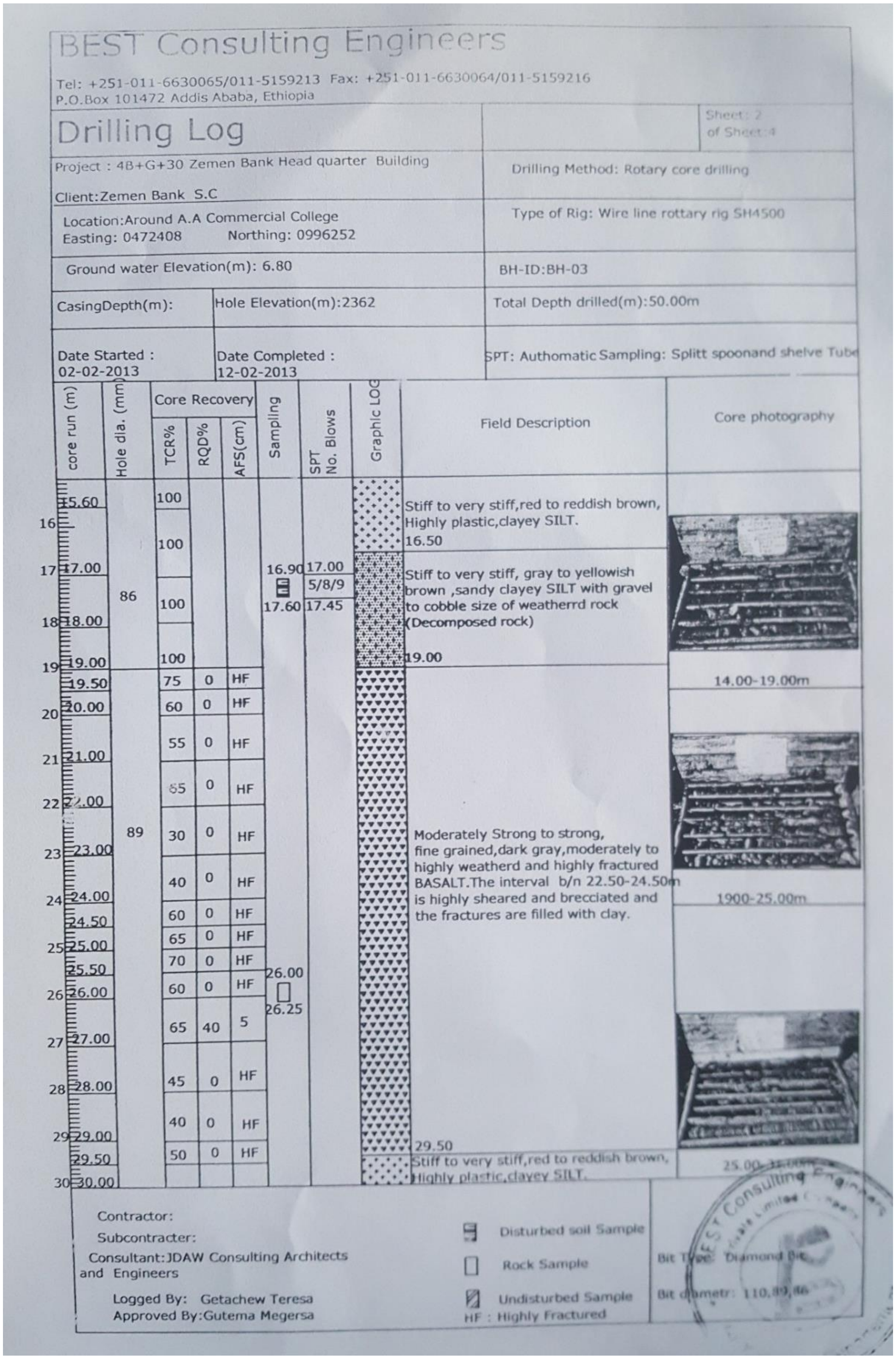
SPT: Authomatic Sampling: Splitt spoonand shelve Tube

core run (m)	Hole dia. (mm)	Core Recovery			Sampling	SPT No. Blows	Graphic LOG	Field Description	Core photography
		TCR%	RQD%	AFS(cm)					
0.50	110	100						0.50 Fill material	
1.00		100						Medium stiff, gravelly clayey SILT 1.00	
1.60		100						Fine grained,dark gray,highly weathered to decomosed and fragmented to Highly fractured BASALT	
2.00		100							
2.30		100	0	HF					
2.60		100							
3.00		100						3.00	
3.45		100						Dense to very dense,brownish to dark gray,gravlly silty SAND interlayerd with Highly weatherd to decomposed and fragmented BASALT(Highly weatherd to decomposed vesicular BASALT)	
4.00		100							
4.50		100							0.00-4.00m
5.00		100							
5.50		100						5.50	
6.00		100	0	HF					
6.40		100							
6.70		100	50	10					
7.20		100	40	10				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.	
7.60		100	0	4					
8.00		100	30	3					
9.20	89	80	35	3					4.00-8.00m
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				13.45	
14.00		100						Stiff to very stiff, red to reddish brown, Highly plastic,clayey SILT	
14.55		100							
15.00								15.00	8.00-14.00m

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

▤ soil Sample
☐ Rock Sample
▨ Undisturbed Sample

Bit Type: Diamond Bit
Bit diametr: 110,89



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Drilling Log

Sheet: 2
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: 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			Sampling	SPT No. Blows	Graphic LOG	Field Description	Core photography
		TCR%	RQD%	AFS(cm)					
15.60	86	100						Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT. 16.50	
17.00		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 weatherrd rock (Decomposed rock)	14.00-19.00m
18.00		100							
19.00		100						19.00	
19.50	89	75	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
20.00		60	0	HF					
21.00		55	0	HF					
22.00		65	0	HF					
23.00		30	0	HF					
24.00		40	0	HF					
24.50		60	0	HF					
25.00		65	0	HF					
25.50		70	0	HF					
26.00		60	0	HF	26.00 □ 26.25				
27.00		65	40	5					
28.00		45	0	HF					
29.00		40	0	HF					
29.50		50	0	HF				29.50 Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT.	25.00-[illegible]
30.00									

Contractor:

Subcontracter:

Consultant:JDAW Consulting Architects and Engineers

Logged By: Getachew Teresa
Approved By:Gutema Megersa

▤ Disturbed soil Sample

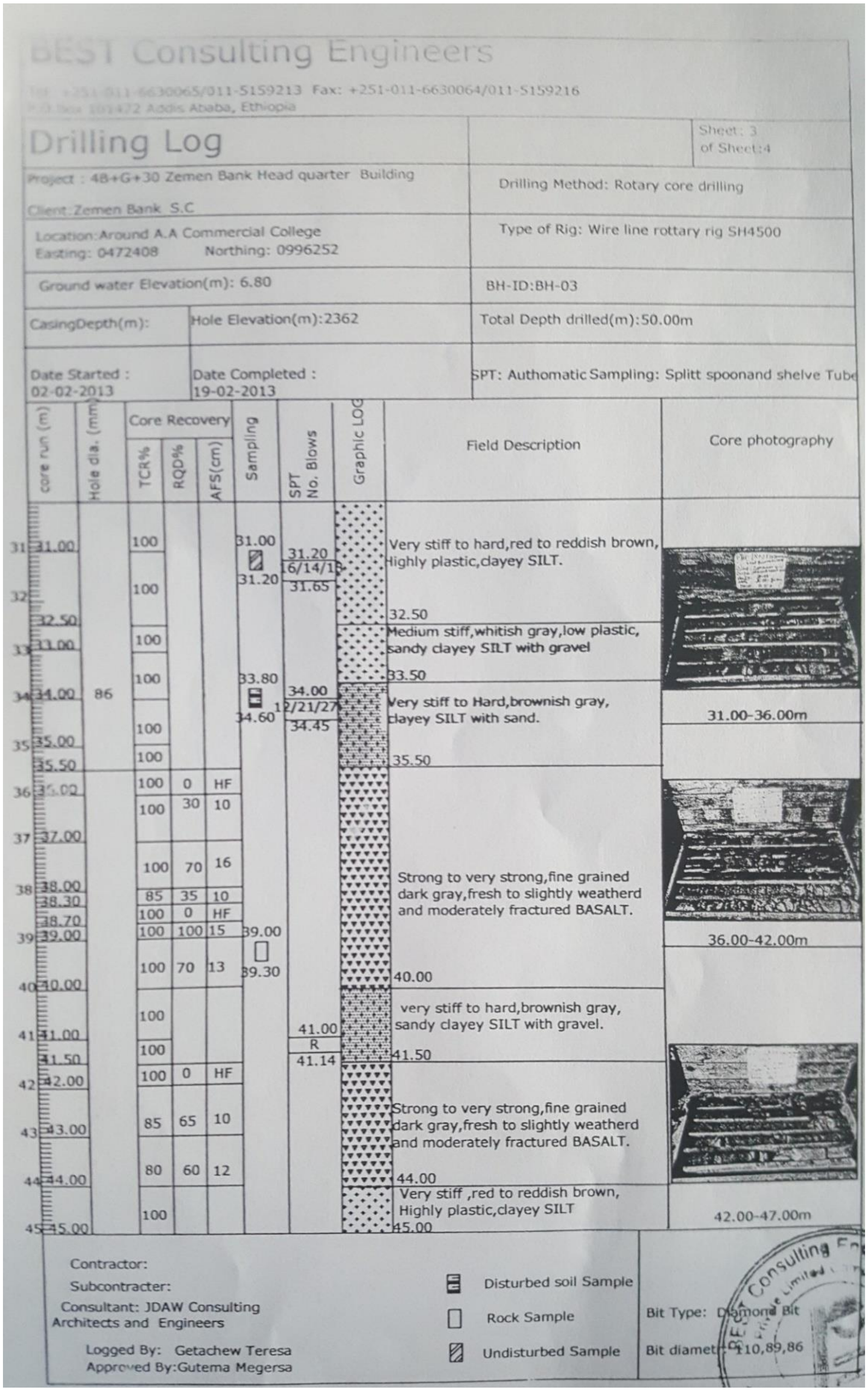
□ Rock Sample

▨ Undisturbed Sample

HF : Highly Fractured

Bit Type: Diamond Bit

Bit diametr: 110,89,86



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Drilling Log

Sheet: 3
of Sheet:4

Project : 48+G+30 Zemen Bank Head quarter Building

Client:Zemen Bank S.C

Drilling Method: Rotary core drilling

Location:Around A.A Commercial College
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 : 19-02-2013

SPT: Authomatic Sampling: Splitt spoonand shelve Tube

	core run (m)	Hole dia. (mm)	Core Recovery TCR%	RQD%	AFS(cm)	Sampling	SPT No. Blows	Field Description
31	31.00		100			31.00 ▨ 31.20	31.20 16/14/1 31.65	Very stiff to hard,red to reddish brown, Highly plastic,clayey SILT. 32.50
32	32.50		100					Medium stiff,whitish gray,low plastic, sandy clayey SILT with gravel 33.50
33	33.00		100					
34	34.00	86	100			33.80 ▤ 34.60	34.00 12/21/27 34.45	Very stiff to Hard,brownish gray, clayey SILT with sand. 35.50
35	35.00		100					
	35.50		100					
36	36.00		100	0	HF			Strong to very strong,fine grained dark gray,fresh to slightly weatherd and moderately fractured BASALT. 40.00
37	37.00		100	30	10			
38	38.00		100	70	16			
	38.30		85	35	10			
	38.70		100	0	HF			
39	39.00		100	100	15	39.00 ☐ 39.30		
40	40.00		100	70	13			very stiff to hard,brownish gray, sandy clayey SILT with gravel. 41.50
41	41.00		100				41.00 R 41.14	
	41.50		100					Strong to very strong,fine grained dark gray,fresh to slightly weatherd and moderately fractured BASALT. 44.00
42	42.00		100	0	HF			
43	43.00		85	65	10			
44	44.00		80	60	12			Very stiff ,red to reddish brown, Highly plastic,clayey SILT 45.00
45	45.00		100					

Core photography

31.00-36.00m

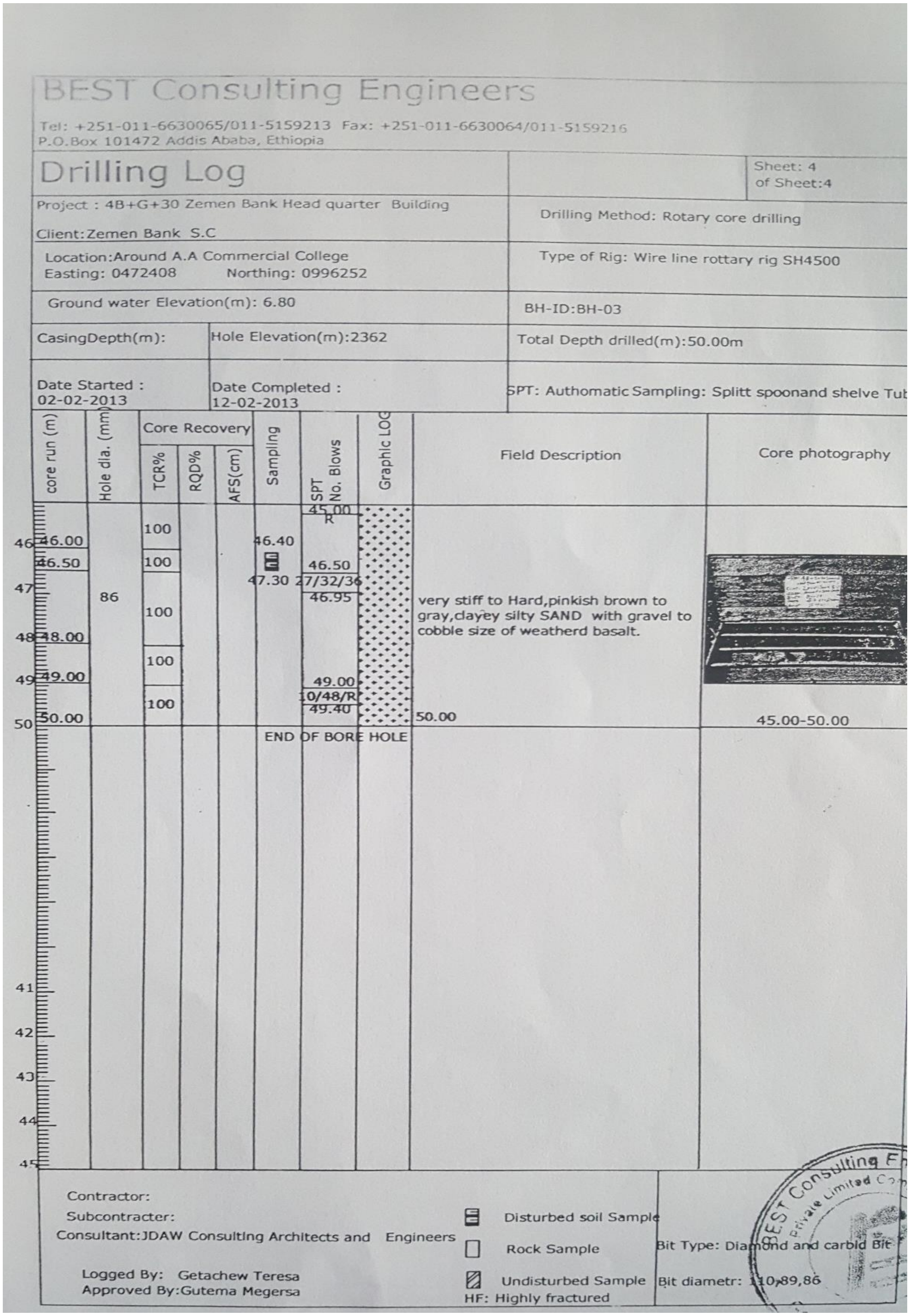
36.00-42.00m

42.00-47.00m

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



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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: 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 Tu

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
						45.00 R			
46.00	86	100			46.40			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 27/32/36 46.95			
48.00		100							
49.00		100				49.00 10/48/R 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

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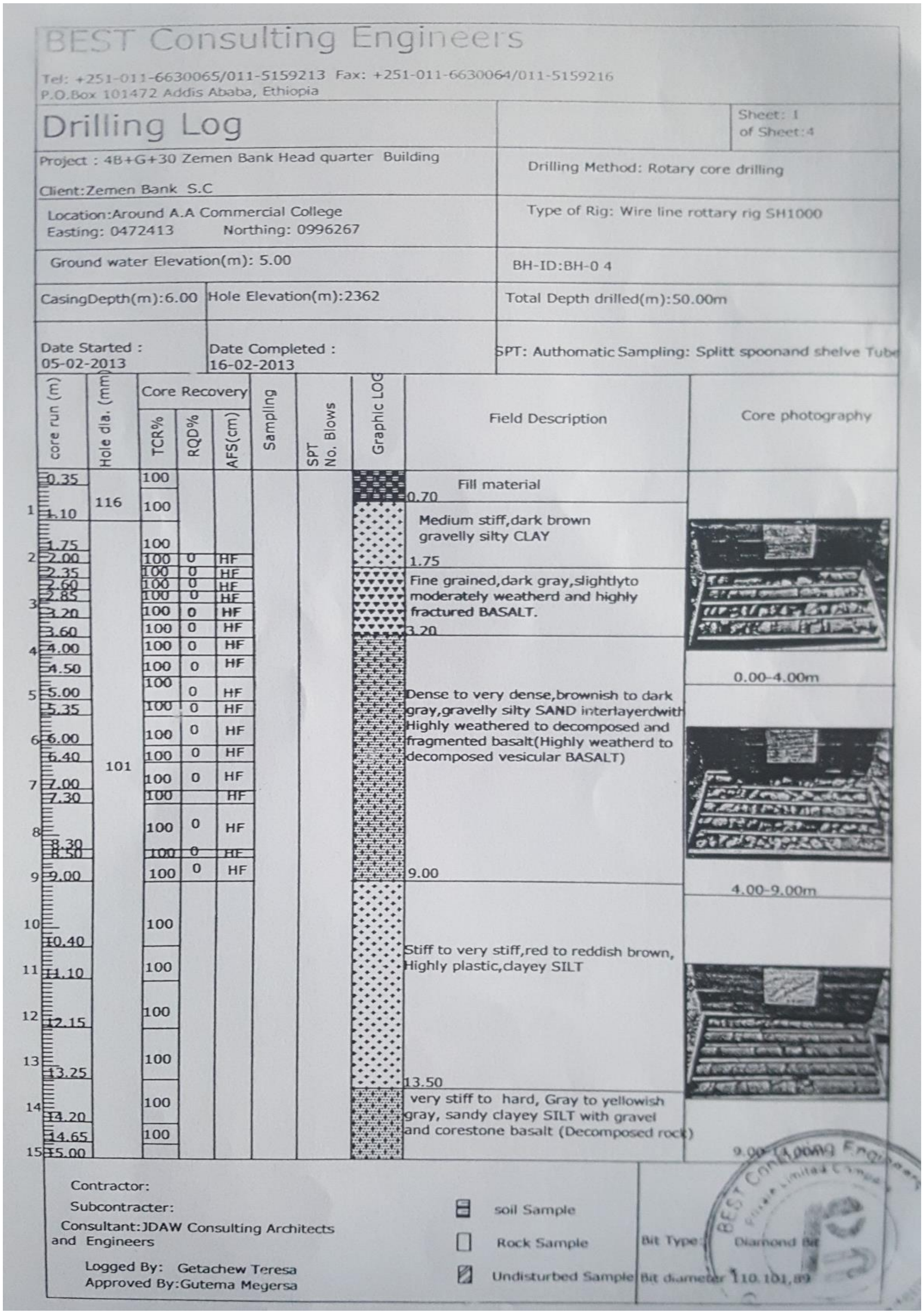
Private Limited Co



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Drilling Log

Sheet: 1
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 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

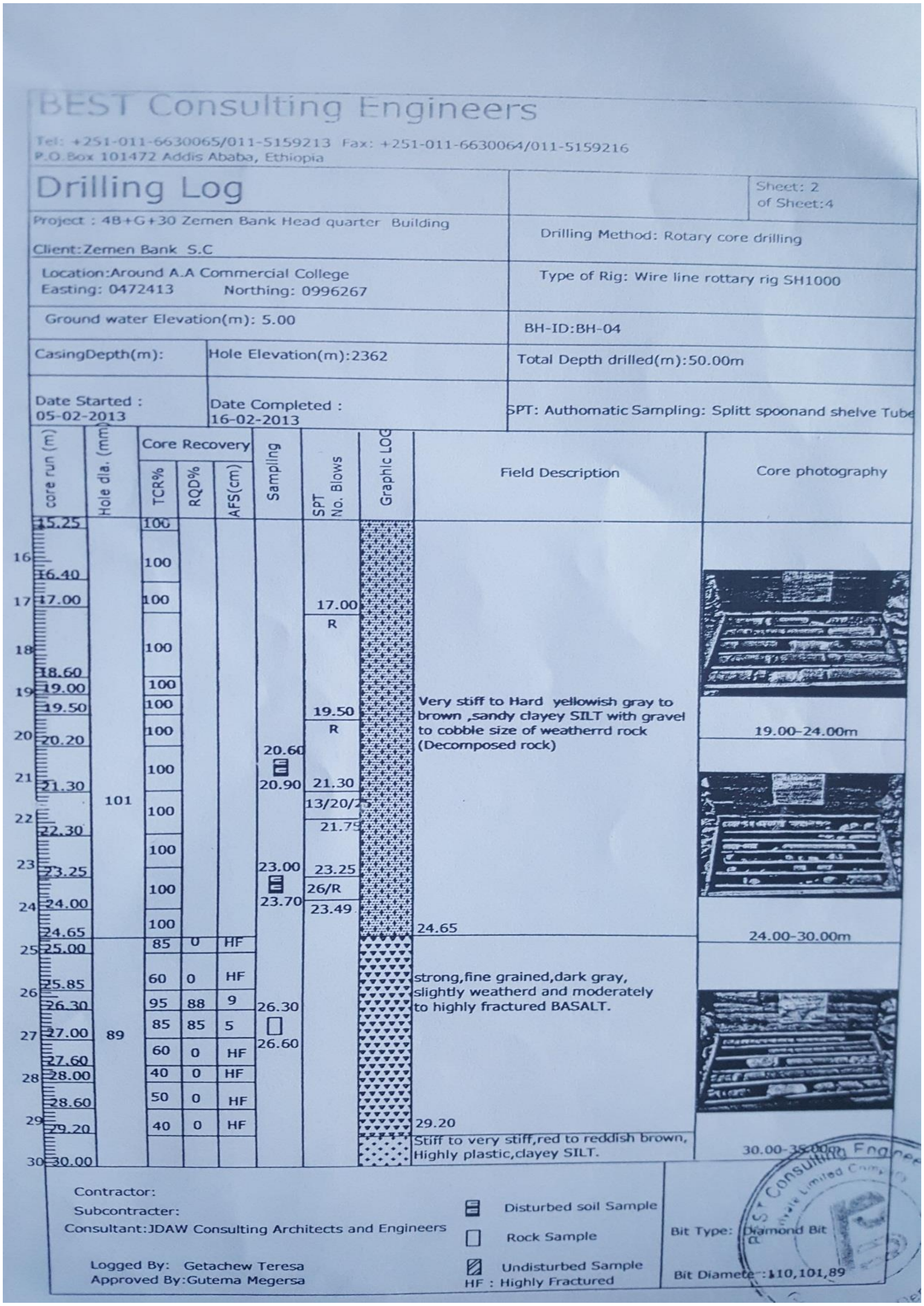
core run (m)	Hole dia. (mm)	Core Recovery TCR%	Core Recovery RQD%	Core Recovery AFS(cm)	Sampling	SPT No. Blows	Graphic LOG	Field Description	Core photography
0.35	116	100						Fill material	
1.10		100					0.70	Medium stiff,dark brown gravelly silty CLAY	
1.75	101	100					1.75		
2.00		100	0	HF				Fine grained,dark gray,slightlyto moderately weatherd and highly fractured BASALT.	
2.35		100	0	HF					
2.60		100	0	HF					
2.85		100	0	HF					
3.20		100	0	HF			3.20		
3.60		100	0	HF				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)	0.00-4.00m
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			9.00		4.00-9.00m
10.40		100						Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT	
11.10		100							
12.15		100							
13.25		100					13.50		
14.20		100						very stiff to hard, Gray to yellowish gray, sandy clayey SILT with gravel and corestone basalt (Decomposed rock)	
14.65		100							
15.00									9.00-14.00m

Contractor:
Subcontracter:
Consultant:JDAW 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

BEST Consulting Engineers



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Drilling Log

Sheet: 2
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 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

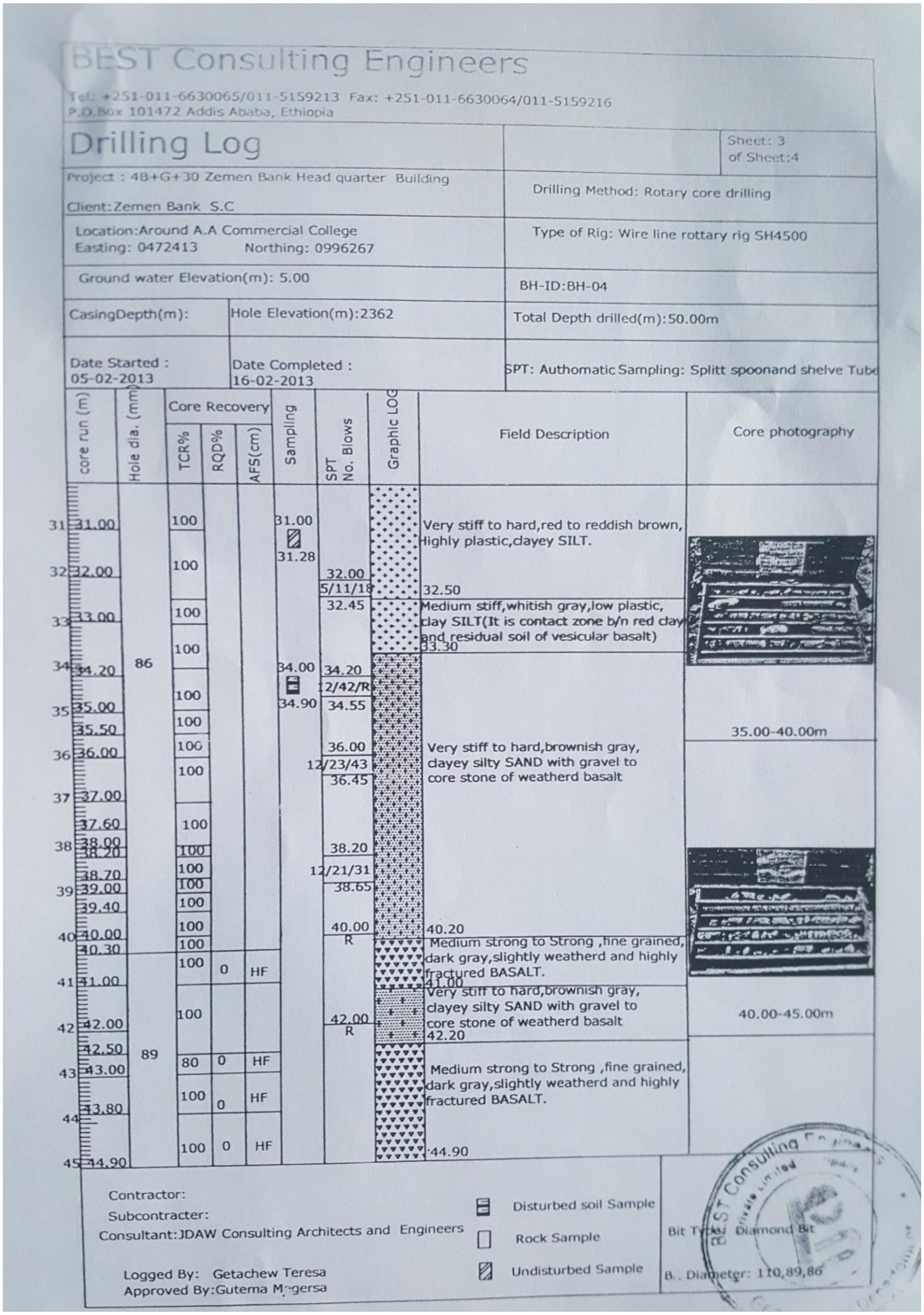
	core run (m)	Hole dia. (mm)	Core Recovery TCR%	RQD%	AFS(cm)	Sampling	SPT No. Blows	Graphic LOG	Field Description	Core photography
	15.25		100							
16	16.40		100							
17	17.00		100				17.00			
18	18.60		100				R			
19	19.00		100							
	19.50		100				19.50		Very stiff to Hard yellowish gray to brown ,sandy clayey SILT with gravel to cobble size of weatherrd rock (Decomposed rock)	19.00-24.00m
20	20.20		100				R			
21	21.30		100			20.60 ▤ 20.90	21.30			
22	22.30	101	100				13/20/2 21.7			
23	23.25		100			23.00	23.25			
24	24.00		100			▤ 23.70	26/R 23.49			
	24.65		100						24.65	24.00-30.00m
25	25.00		85	0	HF					
26	25.85		60	0	HF				strong,fine grained,dark gray, slightly weatherd and moderately to highly fractured BASALT.	
	26.30		95	88	9	26.30				
27	27.00	89	85	85	5	☐ 26.60				
	27.60		60	0	HF					
28	28.00		40	0	HF					
	28.60		50	0	HF					
29	29.20		40	0	HF				29.20	
30	30.00								Stiff to very stiff,red to reddish brown, Highly plastic,clayey SILT.	30.00-35.00m

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 Diamete :110,101,89



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
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 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: 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
31	31.00		100			31.00 ▨ 31.28			Very stiff to hard,red to reddish brown, Highly plastic,clayey SILT. 32.50	
32	32.00		100				32.00 5/11/18 32.45			
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	
			100							
34	34.20	86	100			34.00 ▤ 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 40.20	
37	37.00									
	37.60		100							
38	38.00									
	38.20		100				38.20 12/21/31 38.65			
	38.70		100							
39	39.00		100							
	39.40		100							
40	40.00		100				40.00 R		Medium strong to Strong ,fine grained, dark gray,slightly weatherd and highly fractured BASALT. 41.00	
	40.30		100	0	HF					
41	41.00		100						Very stiff to hard,brownish gray, clayey silty SAND with gravel to core stone of weatherd basalt 42.20	40.00-45.00m
42	42.00						42.00 R			
	42.50	89	80	0	HF				Medium strong to Strong ,fine grained, dark gray,slightly weatherd and highly fractured BASALT. 44.90	
43	43.00		100	0	HF					
44	43.80		100	0	HF					
45	44.90									

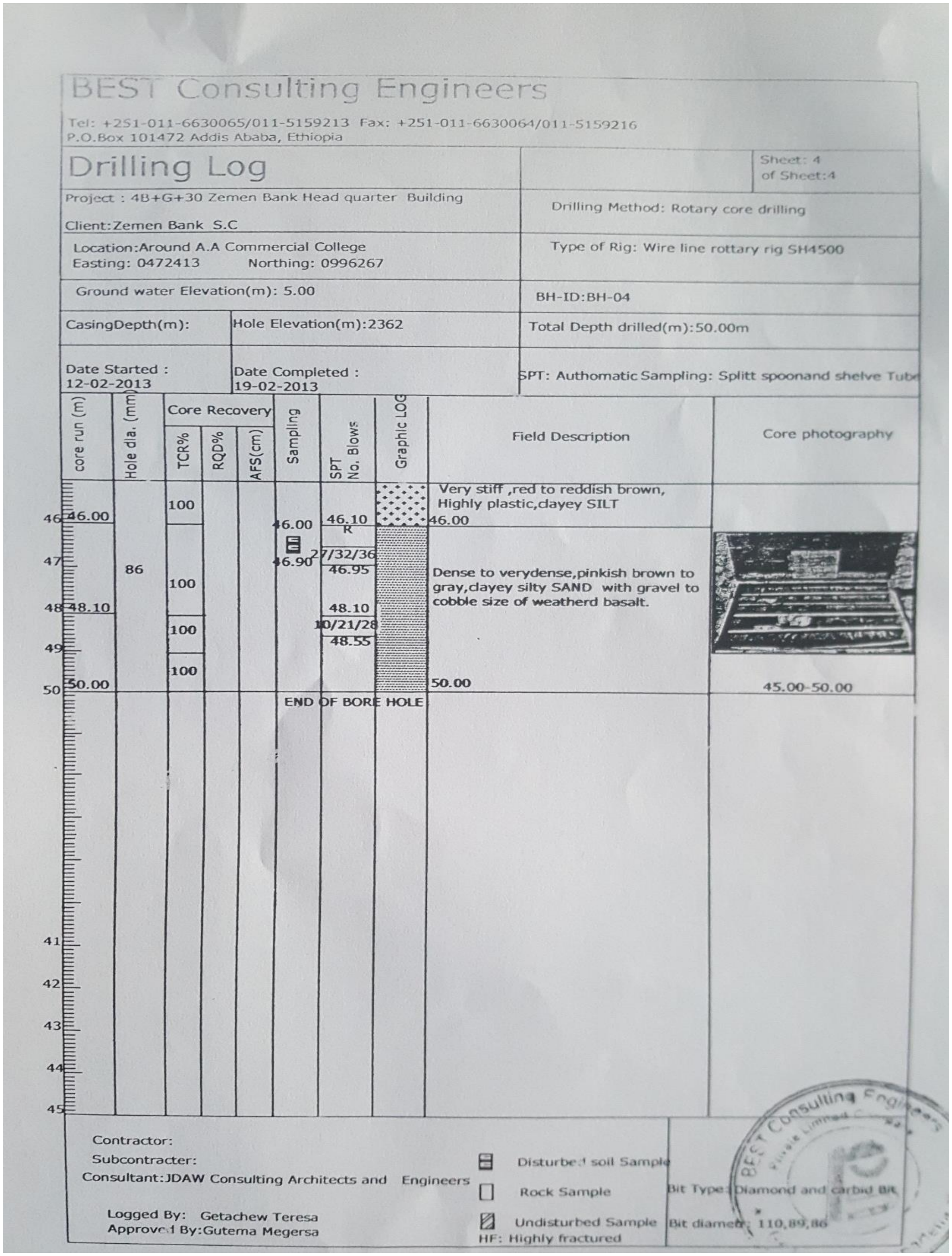
Contractor:
Subcontracter:
Consultant:JDAW Consulting Architects and Engineers

▤ Disturbed soil Sample
☐ Rock Sample
▨ Undisturbed Sample

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

Logged By: Getachew Teresa
Approved By:Gutema M·gersa

BEST Consulting Engineers Private Limited



BEST Consulting Engineers

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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 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 : 12-02-2013
Date Completed : 19-02-2013
SPT: Authomatic Sampling: Splitt spoonand shelve Tub

core run (m)	Hole dia. (mm)	Core Recovery TCR%	RQD%	AFS(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 46.95		Dense to verydense,pinkish brown to gray,clayey silty SAND with gravel to cobble size of weatherd basalt.	45.00-50.00
		100				48.10 10/21/28 48.55			
50.00		100						50.00	

END OF BORE HOLE

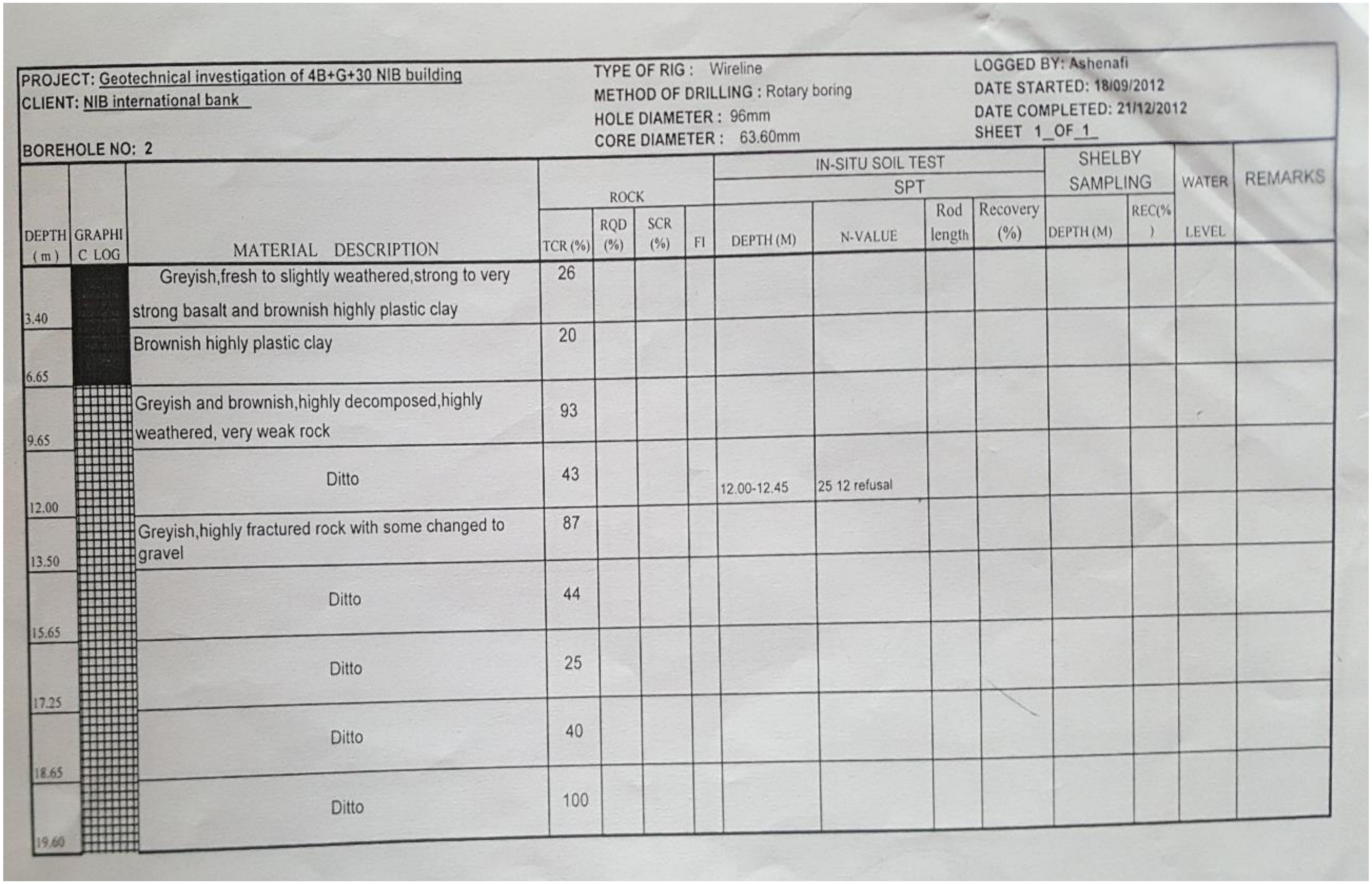
Contractor:
Subcontracter:
Consultant:JDAW Consulting Architects and Engineers

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 diameter: 110,89,86

Nib bank new headquarter building project

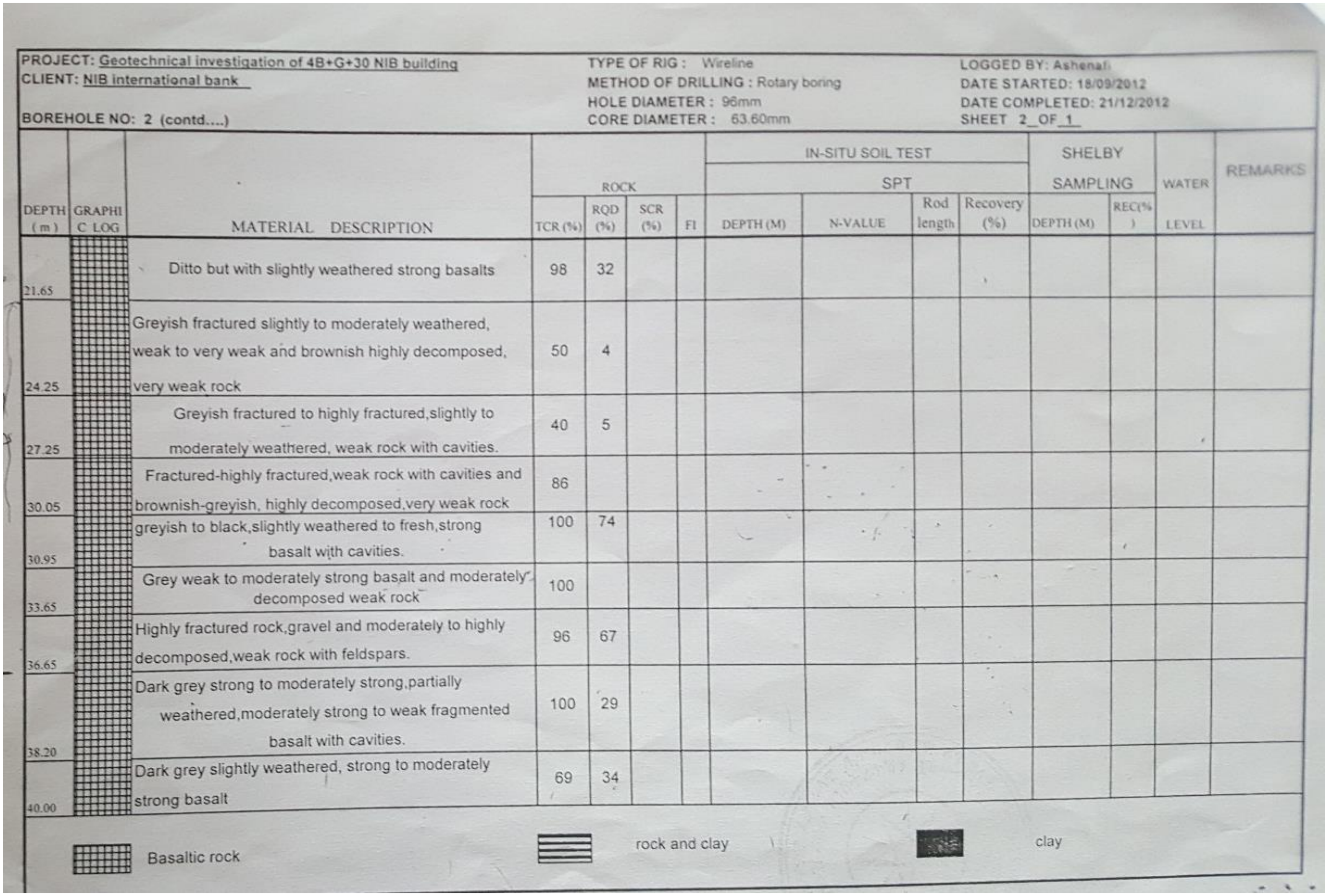


PROJECT: Geotechnical investigation of 4B+G+30 NIB building
CLIENT: NIB international bank
BOREHOLE NO: 2

TYPE OF RIG : Wireline
METHOD OF DRILLING : Rotary boring
HOLE DIAMETER : 96mm
CORE DIAMETER : 63.60mm

LOGGED BY: Ashenafi
DATE STARTED: 18/09/2012
DATE COMPLETED: 21/12/2012
SHEET 1 OF 1

			ROCK				IN-SITU SOIL TEST SPT				SHELBY SAMPLING		WATER	REMARKS
DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	TCR (%)	RQD (%)	SCR (%)	FI	DEPTH (M)	N-VALUE	Rod length	Recovery (%)	DEPTH (M)	REC(%)	LEVEL	
3.40	■	Greyish,fresh to slightly weathered,strong to very strong basalt and brownish highly plastic clay	26											
6.65	■	Brownish highly plastic clay	20											
9.65	▦	Greyish and brownish,highly decomposed,highly weathered, very weak rock	93											
12.00	▦	Ditto	43				12.00-12.45	25 12 refusal						
13.50	▦	Greyish,highly fractured rock with some changed to gravel	87											
15.65	▦	Ditto	44											
17.25	▦	Ditto	25											
18.65	▦	Ditto	40											
19.60	▦	Ditto	100											



PROJECT: Geotechnical investigation of 4B+G+30 NIB building
CLIENT: NIB international bank
BOREHOLE NO: 2 (contd....)

TYPE OF RIG : Wireline
METHOD OF DRILLING : Rotary boring
HOLE DIAMETER : 96mm
CORE DIAMETER : 63.60mm

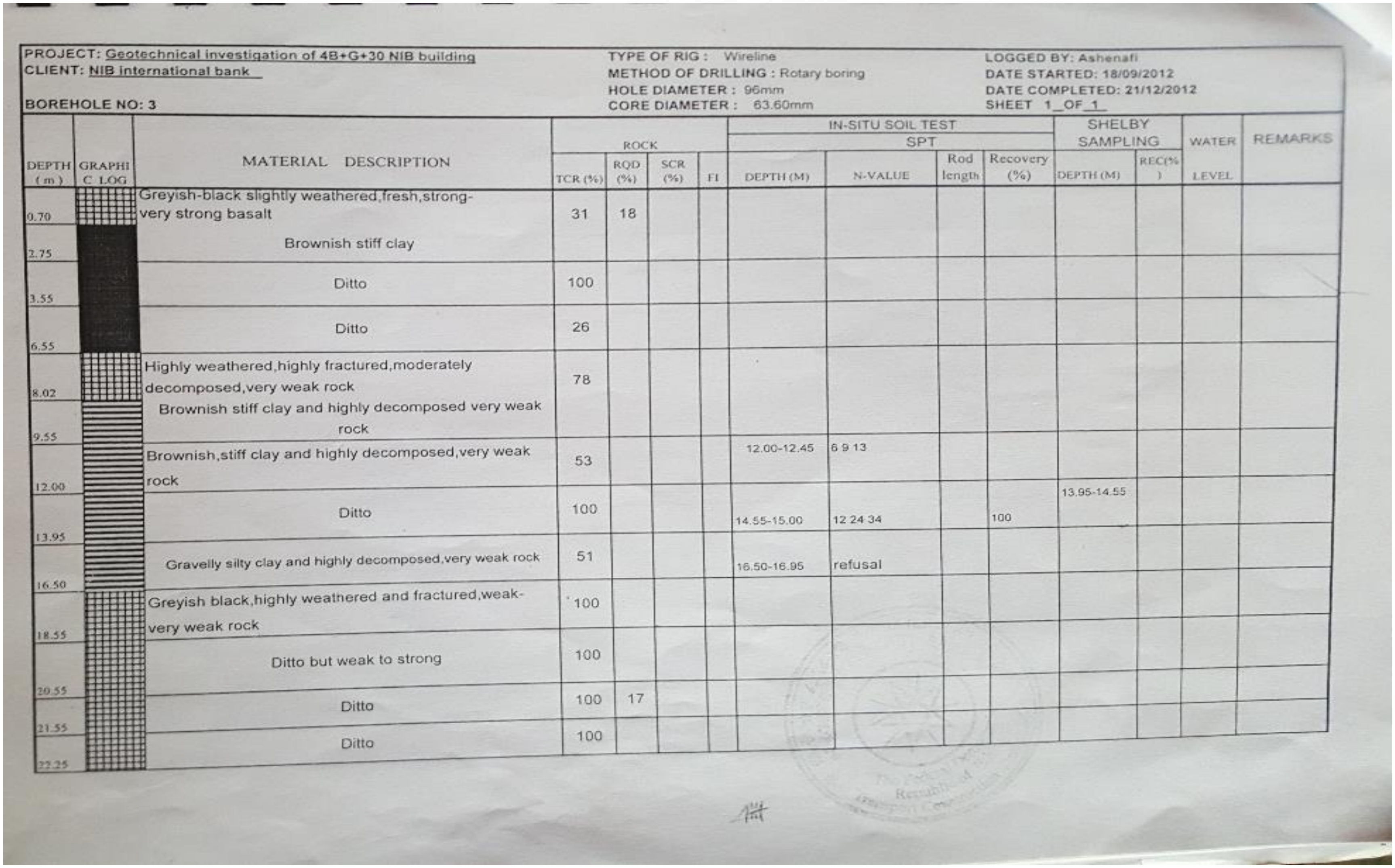
LOGGED BY: Ashenafi
DATE STARTED: 18/09/2012
DATE COMPLETED: 21/12/2012
SHEET 2_OF_1_

DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	ROCK				IN-SITU SOIL TEST SPT				SHELBY SAMPLING		WATER LEVEL	REMARKS
			TCR (%)	RQD (%)	SCR (%)	FI	DEPTH (M)	N-VALUE	Rod length	Recovery (%)	DEPTH (M)	REC(%)		
21.65	▦	Ditto but with slightly weathered strong basalts	98	32										
24.25	▦	Greyish fractured slightly to moderately weathered, weak to very weak and brownish highly decomposed, very weak rock	50	4										
27.25	▦	Greyish fractured to highly fractured,slightly to moderately weathered, weak rock with cavities.	40	5										
30.05	▦	Fractured-highly fractured,weak rock with cavities and brownish-greyish, highly decomposed,very weak rock	86											
30.95	▦	greyish to black,slightly weathered to fresh,strong basalt with cavities.	100	74										
33.65	▦	Grey weak to moderately strong basalt and moderately decomposed weak rock	100											
36.65	▦	Highly fractured rock,gravel and moderately to highly decomposed,weak rock with feldspars.	96	67										
38.20	▦	Dark grey strong to moderately strong,partially weathered,moderately strong to weak fragmented basalt with cavities.	100	29										
40.00	▦	Dark grey slightly weathered, strong to moderately strong basalt	69	34										

▦ Basaltic rock

☰ rock and clay

■ clay



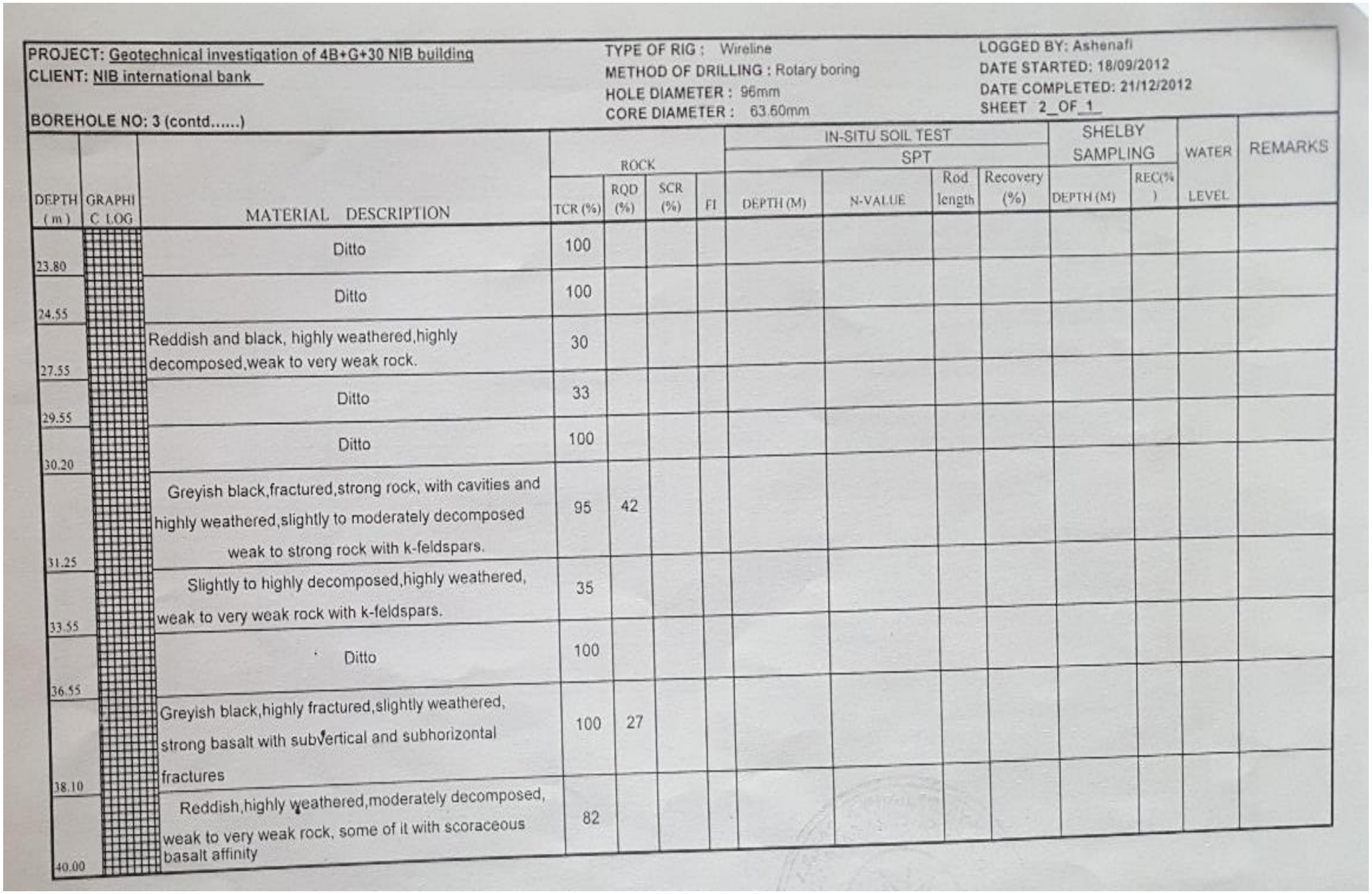
PROJECT: Geotechnical investigation of 4B+G+30 NIB building
CLIENT: NIB international bank

BOREHOLE NO: 3

TYPE OF RIG : Wireline
METHOD OF DRILLING : Rotary boring
HOLE DIAMETER : 96mm
CORE DIAMETER : 63.60mm

LOGGED BY: Ashenafi
DATE STARTED: 18/09/2012
DATE COMPLETED: 21/12/2012
SHEET 1 OF 1

DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	ROCK				IN-SITU SOIL TEST SPT				SHELBY SAMPLING		WATER LEVEL	REMARKS
			TCR (%)	RQD (%)	SCR (%)	FI	DEPTH (M)	N-VALUE	Rod length	Recovery (%)	DEPTH (M)	REC(%)		
0.70	▦	Greyish-black slightly weathered,fresh,strong-very strong basalt	31	18										
2.75	■	Brownish stiff clay												
3.55	■	Ditto	100											
6.55	■	Ditto	26											
8.02	▦	Highly weathered,highly fractured,moderately decomposed,very weak rock	78											
9.55	▤	Brownish stiff clay and highly decomposed very weak rock												
12.00	▤	Brownish,stiff clay and highly decomposed,very weak rock	53				12.00-12.45	6 9 13						
13.95	▤	Ditto	100				14.55-15.00	12 24 34		100	13.95-14.55			
16.50	▤	Gravelly silty clay and highly decomposed,very weak rock	51				16.50-16.95	refusal						
18.55	▦	Greyish black,highly weathered and fractured,weak-very weak rock	100											
20.55	▦	Ditto but weak to strong	100											
21.55	▦	Ditto	100	17										
22.25	▦	Ditto	100											

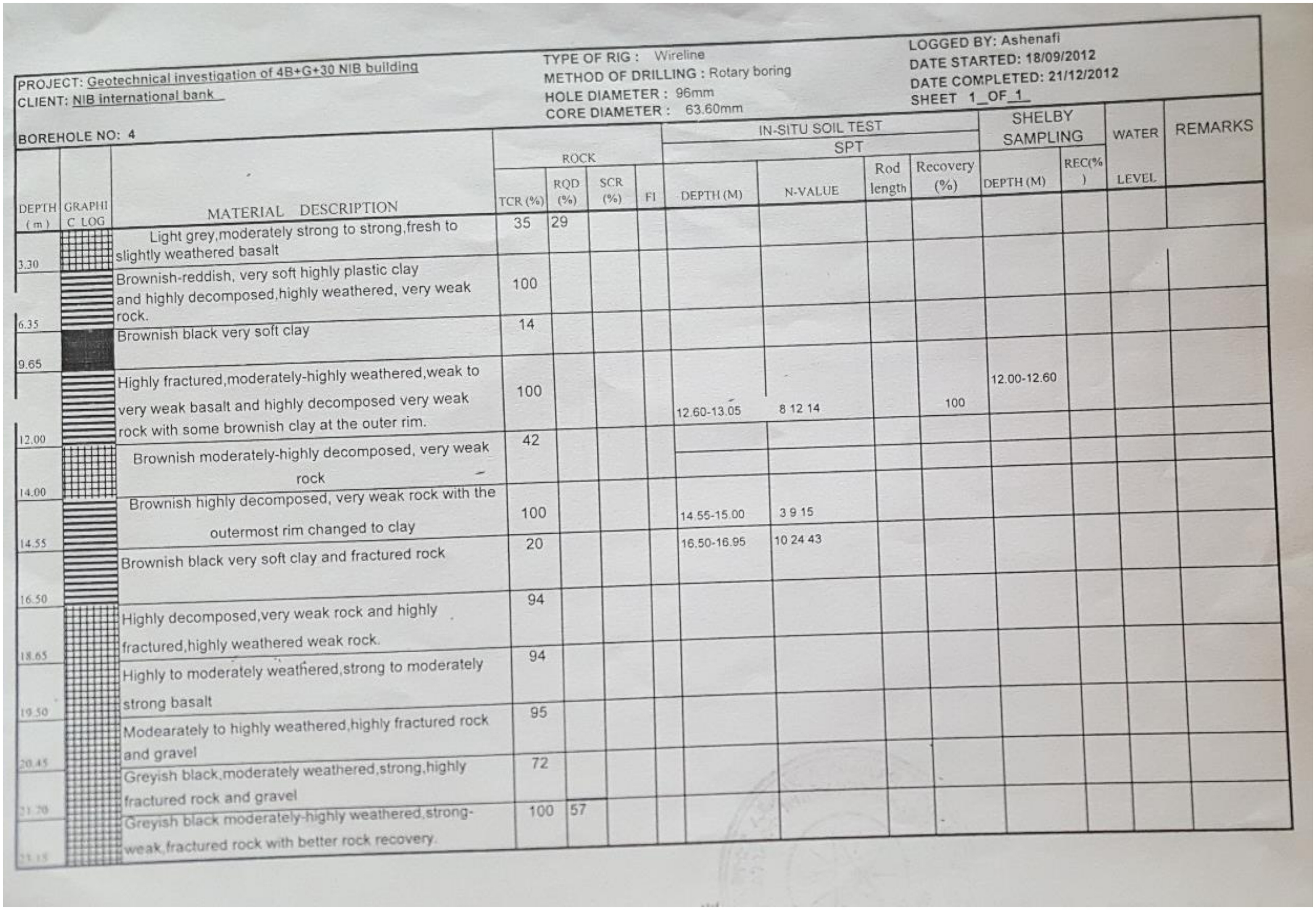


PROJECT: Geotechnical investigation of 4B+G+30 NIB building
CLIENT: NIB international bank
BOREHOLE NO: 3 (contd......)

TYPE OF RIG : Wireline
METHOD OF DRILLING : Rotary boring
HOLE DIAMETER : 96mm
CORE DIAMETER : 63.60mm

LOGGED BY: Ashenafi
DATE STARTED: 18/09/2012
DATE COMPLETED: 21/12/2012
SHEET 2 OF 1

DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	ROCK				IN-SITU SOIL TEST SPT				SHELBY SAMPLING		WATER LEVEL	REMARKS
			TCR (%)	RQD (%)	SCR (%)	FI	DEPTH (M)	N-VALUE	Rod length	Recovery (%)	DEPTH (M)	REC(%)		
23.80		Ditto	100											
24.55		Ditto	100											
27.55		Reddish and black, highly weathered,highly decomposed,weak to very weak rock.	30											
29.55		Ditto	33											
30.20		Ditto	100											
31.25		Greyish black,fractured,strong rock, with cavities and highly weathered,slightly to moderately decomposed weak to strong rock with k-feldspars.	95	42										
33.55		Slightly to highly decomposed,highly weathered, weak to very weak rock with k-feldspars.	35											
36.55		Ditto	100											
38.10		Greyish black,highly fractured,slightly weathered, strong basalt with subvertical and subhorizontal fractures	100	27										
40.00		Reddish,highly weathered,moderately decomposed, weak to very weak rock, some of it with scoraceous basalt affinity	82											



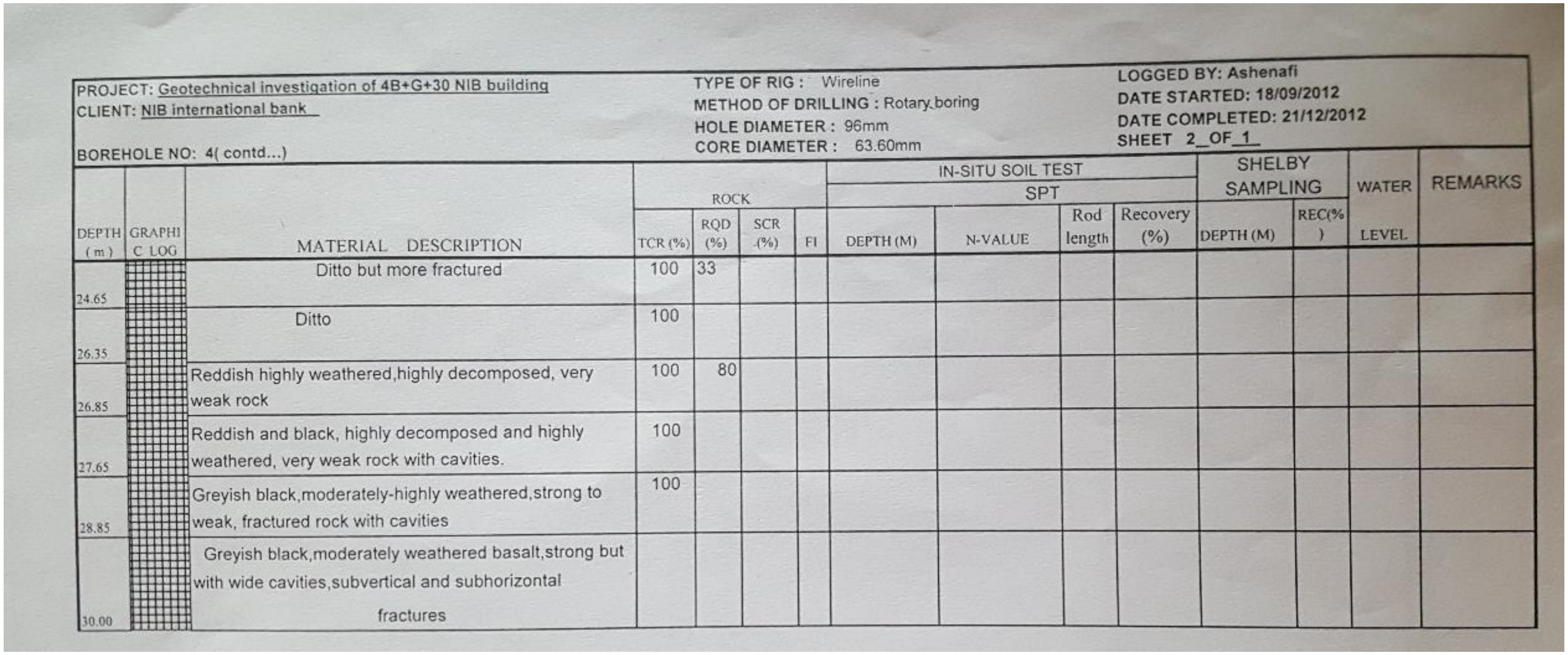
PROJECT: Geotechnical investigation of 4B+G+30 NIB building
CLIENT: NIB international bank

TYPE OF RIG : Wireline
METHOD OF DRILLING : Rotary boring
HOLE DIAMETER : 96mm
CORE DIAMETER : 63.60mm

LOGGED BY: Ashenafi
DATE STARTED: 18/09/2012
DATE COMPLETED: 21/12/2012
SHEET 1 OF 1

BOREHOLE NO: 4

			ROCK				IN-SITU SOIL TEST SPT				SHELBY SAMPLING		WATER	REMARKS
DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	TCR (%)	RQD (%)	SCR (%)	FI	DEPTH (M)	N-VALUE	Rod length	Recovery (%)	DEPTH (M)	REC(%)	LEVEL	
3.30	▦	Light grey,moderately strong to strong,fresh to slightly weathered basalt	35	29										
6.35	▤	Brownish-reddish, very soft highly plastic clay and highly decomposed,highly weathered, very weak rock.	100											
9.65	■	Brownish black very soft clay	14											
12.00	▤	Highly fractured,moderately-highly weathered,weak to very weak basalt and highly decomposed very weak rock with some brownish clay at the outer rim.	100				12.60-13.05	8 12 14		100	12.00-12.60			
14.00	▦	Brownish moderately-highly decomposed, very weak rock	42											
14.55	▤	Brownish highly decomposed, very weak rock with the outermost rim changed to clay	100				14.55-15.00	3 9 15						
16.50	▤	Brownish black very soft clay and fractured rock	20				16.50-16.95	10 24 43						
18.65	▦	Highly decomposed,very weak rock and highly fractured,highly weathered weak rock.	94											
19.50	▦	Highly to moderately weathered,strong to moderately strong basalt	94											
20.45	▦	Modearately to highly weathered,highly fractured rock and gravel	95											
21.70	▦	Greyish black,moderately weathered,strong,highly fractured rock and gravel	72											
23.15	▦	Greyish black moderately-highly weathered,strong-weak,fractured rock with better rock recovery.	100	57										



PROJECT: Geotechnical investigation of 4B+G+30 NIB building
CLIENT: NIB international bank
BOREHOLE NO: 4(contd...)

TYPE OF RIG : Wireline
METHOD OF DRILLING : Rotary boring
HOLE DIAMETER : 96mm
CORE DIAMETER : 63.60mm

LOGGED BY: Ashenafi
DATE STARTED: 18/09/2012
DATE COMPLETED: 21/12/2012
SHEET 2 OF 1

			ROCK				IN-SITU SOIL TEST SPT				SHELBY SAMPLING		WATER	REMARKS
DEPTH (m)	GRAPHIC LOG	MATERIAL DESCRIPTION	TCR (%)	RQD (%)	SCR (%)	FI	DEPTH (M)	N-VALUE	Rod length	Recovery (%)	DEPTH (M)	REC(%)	LEVEL	
24.65	▦	Ditto but more fractured	100	33										
26.35	▦	Ditto	100											
26.85	▦	Reddish highly weathered,highly decomposed, very weak rock	100	80										
27.65	▦	Reddish and black, highly decomposed and highly weathered, very weak rock with cavities.	100											
28.85	▦	Greyish black,moderately-highly weathered,strong to weak, fractured rock with cavities	100											
30.00	▦	Greyish black,moderately weathered basalt,strong but with wide cavities,subvertical and subhorizontal fractures												

Appendix B – Pile load test data

United bank project pile no. 1 pile load test data

Project UNITED BANK

Pile Load Test No. _01_ Page: _01-04_

Pile No. PC1-72 Date: 27-28 APRIL 2016

Load	work load	Pressure	Load Cell	time		temp.	reading dial gauges				settlement		remarks
				actual waiting			d.g. 1	d.g.2	d.g.3	d.g.4	total	w. time	
KN	%	bar		h	min	oc	mm	mm	mm	mm	mm	mm	
0	0%	0		11	00		7000	7000	7000	7000	0	0	
813	25%	55.3		11	00		6898	6790	6844	6854	1.535	1.535	
813	25%	55.3		11	05		6896	6788	6843	6853	1.55	0.015	
813	25%	55.3		11	10		6895	6787	6840	6853	1.5625	0.0125	
813	25%	55.3		11	20		6889	6777	6831	6845	1.645	0.0825	
813	25%	55.3		11	30		6886	6775	6828	6844	1.6675	0.0225	
813	25%	55.3		11	60		6881	6772	6825	6842	1.7	0.0325	
1625	50%	110.5		12	00		6849	6639	6739	6811	2.405	0.705	
1625	50%	110.5		12	05		6846	6638	6738	6810	2.42	0.015	
1625	50%	110.5		12	10		6844	6637	6736	6810	2.4325	0.0125	
1625	50%	110.5		12	20		6838	6625	6726	6802	2.5225	0.09	
1625	50%	110.5		12	30		6835	6620	6724	6801	2.55	0.0275	
1625	50%	110.5		12	60		6815	6601	6705	6784	2.7375	0.1875	
2438	75%	165.7		13	00		6782	6522	6661	6752	3.2075	0.47	
2438	75%	165.7		13	05		6780	6519	6658	6750	3.2325	0.025	
2438	75%	165.7		13	10		6779	6520	6660	6751	3.225	-0.0075	
2438	75%	165.7		13	20		6780	6519	6658	6752	3.2275	0.0025	
2438	75%	165.7		13	30		6781	6518	6659	6751	3.2275	0	
2438	75%	165.7		13	60		6779	6517	6659	6751	3.235	0.0075	
3250	100%	220.9		14	00		6739	6474	6619	6718	3.625	0.39	
3250	100%	220.9		14	05		6739	6474	6619	6718	3.625	0	
3250	100%	220.9		14	10		6739	6466	6618	6719	3.645	0.02	
3250	100%	220.9		14	20		6739	6466	6618	6719	3.645	0	
3250	100%	220.9		14	30		6739	6466	6618	6719	3.645	0	
3250	100%	220.9		14	60		6739	6466	6618	6719	**3.645**	0	
1625	50%	110.5		15	00		6784	6520	6768	6761	2.9175	-0.7275	
1625	50%	110.5		15	05		6784	6520	6767	6761	2.92	0.0025	
1625	50%	110.5		15	10		6784	6520	6767	6760	2.9225	0.0025	
1625	50%	110.5		15	20		6786	6523	6771	6763	2.8925	-0.03	
1625	50%	110.5		15	30		6785	6521	6769	6761	2.91	0.0175	
1625	50%	110.5		15	60		6801	6532	6780	6772	2.7875	-0.1225	
0	0%	0		16	00		6885	6734	6772	6860	1.8725	-0.915	
0	0%	0		16	05		6884	6773	6776	6862	1.7625	-0.11	
0	0%	0		16	10		6884	6773	6776	6862	1.7625	0	
0	0%	0		16	20		6884	6773	6776	6862	1.7625	0	
0	0%	0		16	30		6884	6773	6776	6862	1.7625	0	
0	0%	0		16	60		6884	6773	6776	6862	1.7625	0	
1625	50%	110.5		17	00		6823	6567	6709	6798	2.7575	0.995	
1625	50%	110.5		17	05		6823	6566	6710	6799	2.755	-0.0025	
1625	50%	110.5		17	10		6824	6567	6710	6799	2.75	-0.005	
1625	50%	110.5		17	20		6825	6567	6710	6799	2.7475	-0.0025	
1625	50%	110.5		17	30		6826	6568	6712	6800	2.735	-0.0125	
1625	50%	110.5		17	60		6829	6572	6714	6804	2.7025	-0.0325	

2438	75%	165.7		18	00		6812	6545	6696	6789	2.895	0.1925	
2438	75%	165.7		18	05		6812	6545	6695	6788	2.9	0.005	
2438	75%	165.7		18	10		6812	6546	6695	6788	2.8975	-0.0025	
2438	75%	165.7		18	20		6813	6546	6696	6789	2.89	-0.0075	
2438	75%	165.7		18	30		6814	6546	6697	6789	2.885	-0.005	
2438	75%	165.7		18	60		6814	6546	6697	6790	2.8825	-0.0025	
3250	100%	220.9		19	00		6782	6511	6662	6763	3.205	0.3225	
3250	100%	220.9		19	05		6780	6510	6661	6761	3.22	0.015	
3250	100%	220.9		19	10		6780	6511	6661	6761	3.2175	-0.0025	
3250	100%	220.9		19	20		6781	6511	6662	6762	3.21	-0.0075	
3250	100%	220.9		19	30		6782	6512	6663	6763	3.2	-0.01	
3250	100%	220.9		19	60		6783	6515	6666	6765	3.1775	-0.0225	
4063	125%	276.1		20	00		6745	6481	6631	6738	3.5125	0.335	
4063	125%	276.1		20	05		6742	6479	6631	6736	3.53	0.0175	
4063	125%	276.1		20	10		6741	6478	6630	6735	3.54	0.01	
4063	125%	276.1		20	20		6740	6477	6630	6735	3.545	0.005	
4063	125%	276.1		20	30		6738	6476	6629	6734	3.5575	0.0125	
4063	125%	276.1		20	60		6736	6475	6629	6735	3.5625	0.005	
4875	150%	331.3		21	00		6704	6447	6602	6713	3.835	0.2725	
4875	150%	331.3		21	05		6701	6445	6601	6711	3.855	0.02	
4875	150%	331.3		21	10		6700	6445	6601	6711	3.8575	0.0025	
4875	150%	331.3		21	20		6700	6445	6601	6712	3.855	-0.0025	
4875	150%	331.3		21	30		6700	6445	6602	6712	3.8525	-0.0025	
4875	150%	331.3		21	60		6695	6444	6602	6712	3.8675	0.015	
5688	175%	386.5		22	00		6637	6407	6566	6681	4.2725	0.405	
5688	175%	386.5		22	05		6631	6405	6565	6679	4.3	0.0275	
5688	175%	386.5		22	10		6629	6404	6564	6679	4.31	0.0100	
5688	175%	386.5		22	20		6627	6403	6563	6678	4.3225	0.0125	
5688	175%	386.5		22	30		6625	6402	6563	6678	4.33	0.0075	
5688	175%	386.5		22	60		6623	6401	6563	6678	4.3375	0.0075	
6500	200%	441.7		23	00		6558	6361	6526	6643	**4.78**	0.4425	
6500	200%	441.7		23	05		6550	6356	6524	6639	4.8275	0.0475	
6500	200%	441.7		23	10		6546	6356	6523	6639	4.84	0.0125	
6500	200%	441.7		23	20		6543	6355	6523	6638	4.8525	0.0125	
6500	200%	441.7		23	30		6541	6355	6522	6638	4.86	0.0075	
6500	200%	441.7		0	00		6533	6350	6519	6634	4.91	0.0500	
6500	200%	441.7		1	00		6530	6349	6518	6633	4.925	0.0150	
6500	200%	441.7		2	00		6523	6342	6512	6629	4.985	0.0600	
6500	200%	441.7		3	00		6519	6339	6509	6625	5.02	0.0350	
6500	200%	441.7		4	00		6521	6342	6512	6627	4.995	-0.0250	
6500	200%	441.7		5	00		6525	6346	6515	6631	4.9575	-0.0375	
6500	200%	441.7		6	00		6525	6344	6515	6630	4.965	0.0075	
6500	200%	441.7		7	00		6522	6343	6514	6630	4.9775	0.0125	
6500	200%	441.7		8	00		6510	6332	6504	6619	5.0875	0.1100	
6500	200%	441.7		9	00		6501	6323	6495	6611	5.175	0.0875	
6500	200%	441.7		10	00		6468	6288	6464	6581	5.4975	0.3225	
6500	200%	441.7		11	00		6468	6286	6462	6577	**5.5175**	0.0200	
4875	150%	331.3		11	00		6496	6309	6488	6599	5.27	-0.2475	
4875	150%	331.3		11	05		6483	6306	6486	6598	5.3175	0.0475	
4875	150%	331.3		11	10		6483	6305	6484	6596	5.33	0.0125	

4875	150%	331.3		11	20		6479	6300	6480	6592	5.3725	0.0425	
4875	150%	331.3		11	30		6485	6306	6485	6597	5.3175	-0.0550	
4875	150%	331.3		11	60		6478	6301	6481	6591	5.3725	0.0550	
3250	100%	220.9		12	00		6550	6363	6545	6648	4.735	-0.6375	
3250	100%	220.9		12	05		6547	6361	6543	6646	4.7575	0.0225	
3250	100%	220.9		12	10		6544	6358	6541	6643	4.785	0.0275	
3250	100%	220.9		12	20		6539	6353	6536	6639	4.8325	0.0475	
3250	100%	220.9		12	30		6547	6354	6547	6647	4.7625	-0.0700	
3250	100%	220.9		12	60		6541	6353	6537	6639	4.825	0.0625	
1625	50%	110.5		13	00		6619	6437	6608	6698	4.095	-0.7300	
1625	50%	110.5		13	05		6619	6436	6608	6698	4.0975	0.0025	
1625	50%	110.5		13	10		6620	6437	6609	6699	4.0875	-0.0100	
1625	50%	110.5		13	20		6630	6449	6620	6709	3.98	-0.1075	
1625	50%	110.5		13	30		6632	6449	6620	6710	3.9725	-0.0075	
1625	50%	110.5		13	60		6636	6447	6624	6714	3.9475	-0.0250	
0	0%	0		14	00		6651	6777	6640	6725	**3.0175**	-0.9300	
0	0%	0		14	05		6655	6782	6645	6730	2.97	-0.0475	
0	0%	0		14	10		6660	6785	6648	6733	2.935	-0.0350	
0	0%	0		14	20		6656	6780	6643	6729	2.98	0.0450	
0	0%	0		14	30		6653	6777	6640	6727	3.0075	0.0275	
0	0%	0		15	00		6664	6789	6651	6736	2.9	-0.1075	
0	0%	0		16	00		6831	6932	6969	6878	0.975	-1.9250	
0	0%	0		17	00		6849	6954	6990	6896	0.7775	-0.1975	
0	0%	0		18	00		6860	6968	7002	6906	0.66	-0.1175	
0	0%	0		19	00		6868	6976	7009	6912	0.5875	-0.0725	
0	0%	0		20	00		6872	6981	7013	6915	0.5475	-0.0400	
0	0%	0		21	00		6874	6985	7015	6917	0.5225	-0.0250	
0	0%	0		22	00		6877	6988	7019	6920	0.49	-0.0325	

United bank project pile no. 2 pile load test data

Project UNITED BANK
Pile Load Test No. __02_ Page: __01-04_
Pile No. PC1-182 Date: 9-May-16 - 10-May-16

Load	work load	Pressure	Load Cell	time		temp.	reading dial gauges				settlement		remarks
				actual waiting			d.g. 1	d.g.2	d.g.3	d.g.4	total	w. time	
KN	%	bar		h	min	oc	mm	mm	mm	mm	mm	mm	
0	0%	0		12	00		7000	7000	7000	7000	0	0	
813	25%	55.3		12	00		6702	6784	6804	6781	2.3225	2.3225	
813	25%	55.3		12	05		6707	6790	6810	6788	2.2625	-0.06	
813	25%	55.3		12	10		6708	6790	6806	6785	2.2775	0.015	
813	25%	55.3		12	20		6691	6777	6802	6774	2.39	0.1125	
813	25%	55.3		12	30		6688	6775	6797	6772	2.42	0.03	
813	25%	55.3		12	60		6681	6768	6793	6768	2.475	0.055	
1625	50%	110.5		13	00		6548	6740	6745	6714	3.1325	0.6575	
1625	50%	110.5		13	05		6540	6734	6739	6707	3.2	0.0675	
1625	50%	110.5		13	10		6540	6734	6739	6707	3.2	0	
1625	50%	110.5		13	20		6540	6734	6739	6707	3.2	0	
1625	50%	110.5		13	30		6540	6734	6739	6707	3.2	0	
1625	50%	110.5		13	60		6556	6752	6728	6658	3.265	0.065	
2438	75%	165.7		14	00		6425	6681	6718	6673	3.7575	0.4925	
2438	75%	165.7		14	05		6426	6682	6717	6672	3.7575	0	
2438	75%	165.7		14	10		6424	6681	6715	6671	3.7725	0.015	
2438	75%	165.7		14	20		6419	6674	6708	6663	3.84	0.0675	
2438	75%	165.7		14	30		6419	6674	6708	6663	3.84	0	
2438	75%	165.7		14	60		6419	6674	6708	6663	3.84	0	
3250	100%	220.9		15	00		6357	6652	6702	6639	4.125	0.285	
3250	100%	220.9		15	05		6357	6652	6702	6639	4.125	0	
3250	100%	220.9		15	10		6354	6652	6704	6639	4.1275	0.0025	
3250	100%	220.9		15	20		6352	6651	6703	6637	4.1425	0.015	
3250	100%	220.9		15	30		6350	6649	6700	6636	4.1625	0.02	
3250	100%	220.9		15	60		6346	6645	6696	6632	**4.2025**	0.04	
1625	50%	110.5		16	00		6460	6705	6733	6695	3.5175	-0.685	
1625	50%	110.5		16	05		6463	6705	6733	6696	3.5075	-0.01	
1625	50%	110.5		16	10		6463	6706	6734	6696	3.5025	-0.005	
1625	50%	110.5		16	20		6464	6706	6734	6696	3.5	-0.0025	
1625	50%	110.5		16	30		6465	6706	6734	6696	3.4975	-0.0025	
1625	50%	110.5		16	60		6465	6707	6734	6698	3.49	-0.0075	
0	0%	0		17	00		6756	6783	6815	6787	2.1475	-1.3425	
0	0%	0		17	05		6761	6787	6816	6789	2.1175	-0.03	
0	0%	0		17	10		6761	6789	6817	6789	2.11	-0.0075	
0	0%	0		17	20		6764	6790	6818	6789	2.0975	-0.0125	
0	0%	0		17	30		6765	6792	6818	6789	2.09	-0.0075	
0	0%	0		17	60		6770	6797	6820	6790	2.0575	-0.0325	
1625	50%	110.5		18	00		6520	6721	6742	6692	3.3125	#REF!	
1625	50%	110.5		18	05		6517	6719	6741	6692	3.3275	0.015	
1625	50%	110.5		18	10		6517	6719	6741	6692	3.3275	0	
1625	50%	110.5		18	20		6516	6719	6741	6691	3.3325	0.005	
1625	50%	110.5		18	30		6517	6720	6742	6691	3.325	-0.0075	
1625	50%	110.5		18	60		6514	6722	6745	6692	3.3175	-0.0075	
2438	75%	165.7		19	00		6446	6701	6736	6675	3.605	0.2875	
2438	75%	165.7		19	05		6443	6700	6736	6675	3.615	0.01	
2438	75%	165.7		19	10		6443	6700	6736	6675	3.615	0	
2438	75%	165.7		19	20		6443	6700	6736	6675	3.615	0	
2438	75%	165.7		19	30		6443	6700	6737	6675	3.6125	-0.0025	
2438	75%	165.7		19	60		6442	6699	6737	6675	3.6175	0.005	
3250	100%	220.9		20	00		6366	6662	6722	6645	4.0125	0.395	
3250	100%	220.9		20	05		6366	6662	6722	6644	4.015	0.0025	
3250	100%	220.9		20	10		6366	6662	6722	6644	4.015	0	
3250	100%	220.9		20	20		6365	6661	6722	6644	4.02	0.005	

3250	100%	220.9		20	30		6363	6661	6722	6643	4.0275	0.0075	
3250	100%	220.9		20	60		6363	6661	6722	6643	4.0275	0	
4063	125%	276.1		21	00		6289	6617	6710	6616	4.42	0.3925	
4063	125%	276.1		21	05		6286	6615	6710	6615	4.435	0.015	
4063	125%	276.1		21	10		6284	6615	6710	6614	4.4425	0.0075	
4063	125%	276.1		21	20		6281	6613	6713	6613	4.45	0.0075	
4063	125%	276.1		21	30		6288	6614	6712	6615	4.4275	-0.0225	
4063	125%	276.1		21	60		6288	6614	6716	6617	4.4125	-0.015	
4875	150%	331.3		22	00		6196	6560	6701	6588	4.8875	0.475	
4875	150%	331.3		22	05		6190	6558	6700	6587	4.9125	0.025	
4875	150%	331.3		22	10		6189	6557	6701	6586	4.9175	0.005	
4875	150%	331.3		22	20		6186	6556	6701	6585	4.93	0.0125	
4875	150%	331.3		22	30		6167	6546	6699	6580	5.02	0.09	
4875	150%	331.3		22	60		6162	6545	6700	6578	5.0375	0.0175	
5688	175%	386.5		23	00		6059	6479	6682	6548	5.58	0.5425	
5688	175%	386.5		23	05		6059	6479	6682	6548	5.58	0.0000	
5688	175%	386.5		23	10		6055	6478	6682	6547	5.595	0.0150	
5688	175%	386.5		23	20		6053	6476	6682	6545	5.61	0.0150	
5688	175%	386.5		23	30		6050	6474	6682	6543	5.6275	0.0175	
5688	175%	386.5		23	60		6027	6458	6676	6533	5.765	0.1375	
6500	200%	441.7		0	00		5955	6409	6663	6514	**6.1475**	0.3825	
6500	200%	441.7		0	05		5941	6400	6660	6508	6.2275	0.0800	
6500	200%	441.7		0	10		5936	6398	6660	6502	6.26	0.0325	
6500	200%	441.7		0	20		5931	6393	6660	6505	6.2775	0.0175	
6500	200%	441.7		0	30		5928	6392	6660	6504	6.29	0.0125	
6500	200%	441.7		1	00		5908	6377	6659	6497	6.3975	0.1075	
6500	200%	441.7		2	00		5897	6369	6656	6491	6.4675	0.0700	
6500	200%	441.7		3	00		5892	6364	6653	6487	6.51	0.0425	
6500	200%	441.7		4	00		5883	6359	6651	6482	6.5625	0.0525	
6500	200%	441.7		5	00		5877	6354	6649	6479	6.6025	0.0400	
6500	200%	441.7		6	00		5876	6354	6649	6479	6.605	0.0025	
6500	200%	441.7		7	00		5863	6346	6648	6474	6.6725	0.0675	
6500	200%	441.7		8	00		5860	6345	6647	6474	6.685	0.0125	
6500	200%	441.7		9	00		5856	6341	6646	6471	6.715	0.0300	
6500	200%	441.7		10	00		5852	6336	6642	6466	6.76	0.0450	
6500	200%	441.7		11	00		5846	6330	6636	6460	6.82	0.0600	
6500	200%	441.7		12	00		5822	6307	6614	6438	**7.0475**	0.2275	
4875	150%	331.3		12	00		5870	6336	6626	6458	6.775	-0.2725	
4875	150%	331.3		12	05		5870	6336	6626	6458	6.775	0.0000	
4875	150%	331.3		12	10		5871	6336	6626	6458	6.7725	-0.0025	
4875	150%	331.3		12	20		5872	6336	6626	6458	6.77	-0.0025	
4875	150%	331.3		12	30		5868	6334	6622	6454	6.805	0.0350	
4875	150%	331.3		12	60		5874	6340	6627	6459	6.75	-0.0550	
3250	100%	220.9		13	00		5983	6418	6652	6498	6.1225	-0.6275	
3250	100%	220.9		13	05		5984	6419	6652	6498	6.1175	-0.0050	
3250	100%	220.9		13	10		5984	6419	6652	6498	6.1175	0.0000	
3250	100%	220.9		13	20		5980	6418	6647	6492	6.1575	0.0400	
3250	100%	220.9		13	30		5978	6415	6645	6490	6.18	0.0225	
3250	100%	220.9		13	60		5981	6418	6647	6493	6.1525	-0.0275	
1625	50%	110.5		14	00		6198	6573	6736	6618	4.6875	-1.4650	
1625	50%	110.5		14	05		6200	6573	6736	6618	4.6825	-0.0050	
1625	50%	110.5		14	10		6201	6574	6736	6618	4.6775	-0.0050	
1625	50%	110.5		14	20		6202	6574	6735	6618	4.6775	0.0000	
1625	50%	110.5		14	30		6202	6575	6734	6717	4.43	-0.2475	
1625	50%	110.5		14	60		6202	6575	6734	6717	4.43	0.0000	
0	0%	0		15	00		6630	6746	6920	6812	**2.23**	-2.2000	
0	0%	0		15	05		6640	6752	6922	6818	2.17	-0.0600	
0	0%	0		15	10		6645	6756	6923	6820	2.14	-0.0300	
0	0%	0		15	20		6652	6760	6925	6825	2.095	-0.0450	
0	0%	0		15	30		6659	6763	6926	6829	2.0575	-0.0375	
0	0%	0		16	00		6667	6767	6928	6835	2.0075	-0.0500	
0	0%	0		17	00		6747	6839	6998	6911	1.2625	-0.7450	
0	0%	0		18	00		6779	6807	6980	6990	1.11	-0.1525	
0	0%	0		19	00		6786	6809	6982	6965	1.145	0.0350	
0	0%	0		20	00		6788	6810	6983	6966	1.1325	-0.0125	
0	0%	0		21	00		6791	6813	6984	6970	1.105	-0.0275	

Nib bank project pile no. 1 pile load test data

Project NIB BANK

Pile Load Test No. __01__ Page: ___01-04___

Pile No. WP-226 Date: March 21, 2016 - March 22, 2016

Load	work load	Pressure	Load Cell	time		temp.	reading dial gauges				settlement		remarks
			0	actual waiting			d.g. 1	d.g.2	d.g.3	d.g.4	total	w. time	
KN	%	bar	_	h	min	oc	mm	mm	mm	mm	mm	mm	
0	0%	0	_	11	00		7000	7000	7000	7000	0	0	
725	25%	60	_	11	00		6976	6951	7001	6965	0.2675	0.2675	
725	25%	60	_	11	05		6973	6950	7000	6963	0.285	0.0175	
725	25%	60	_	11	10		6972	6950	6999	6963	0.29	0.005	
725	25%	60	_	11	20		6969	6947	6996	6959	0.3225	0.0325	
725	25%	60	_	11	30		6968	6948	6994	6960	0.325	0.0025	
725	25%	60	_	11	60		6967	6945	6993	6956	0.3475	0.0225	
1450	50%	120	_	12	00		6947	6948	6987	6966	0.38	0.0325	
1450	50%	120	_	12	05		6945	6947	6985	6964	0.3975	0.0175	
1450	50%	120	_	12	10		6943	6945	6982	6962	0.42	0.0225	
1450	50%	120	_	12	20		6943	6945	6981	6961	0.425	0.005	
1450	50%	120	_	12	30		6943	6944	6979	6960	0.435	0.01	
1450	50%	120	_	12	60		6940	6942	6975	6957	0.465	0.03	
2175	75%	179	_	13	00		6925	6939	6969	6952	0.5375	0.0725	
2175	75%	179	_	13	05		6924	6939	6969	6952	0.54	0.0025	
2175	75%	179	_	13	10		6920	6939	6969	6952	0.55	0.01	
2175	75%	179	_	13	20		6918	6939	6965	6954	0.56	0.01	
2175	75%	179	_	13	30		6917	6937	6964	6955	0.5675	0.0075	
2175	75%	179	_	13	60		6917	6937	6964	6955	0.5675	0	
2900	100%	239	_	14	00		6899	6931	6955	6947	0.67	0.1025	
2900	100%	239	_	14	05		6898	6931	6954	6948	0.6725	0.0025	
2900	100%	239	_	14	10		6898	6929	6952	6949	0.68	0.0075	
2900	100%	239	_	14	20		6895	6927	6952	6949	0.6925	0.0125	
2900	100%	239	_	14	30		6895	6927	6952	6949	0.6925	0	
2900	100%	239	_	14	60		6892	6924	6952	6949	**0.7075**	0.015	
1450	50%	120	_	15	00		6920	6944	6963	6972	0.5025	-0.205	
1450	50%	120	_	15	05		6921	6945	6964	6974	0.49	-0.0125	
1450	50%	120	_	15	10		6921	6945	6964	6974	0.49	0	
1450	50%	120	_	15	20		6921	6945	6964	6974	0.49	0	
1450	50%	120	_	15	30		6921	6945	6964	6974	0.49	0	
1450	50%	120	_	15	60		6921	6945	6964	6974	0.49	0	
0	0%	0	_	16	00		6975	6966	6971	6963	0.3125	-0.1775	
0	0%	0	_	16	05		6975	6967	6972	6964	0.305	-0.0075	
0	0%	0	_	16	10		6976	6967	6973	6965	0.2975	-0.0075	
0	0%	0	_	16	20		6976	6968	6974	6965	0.2925	-0.005	
0	0%	0	_	16	30		6977	6969	6975	6966	0.2825	-0.01	
0	0%	0	_	16	60		6979	6971	6977	6969	0.26	-0.0225	
1450	50%	120	_	17	00		6940	6952	6978	6989	0.3525	0.0925	
1450	50%	120	_	17	05		6939	6952	6978	6989	0.355	0.0025	
1450	50%	120	_	17	10		6939	6952	6978	6989	0.355	0	
1450	50%	120	_	17	20		6939	6952	6978	6989	0.355	0	
1450	50%	120	_	17	30		6939	6952	6978	6989	0.355	0	
1450	50%	120	_	17	60		6939	6952	6978	6989	0.355	0	
2175	75%	179	_	18	00		6925	6946	6974	6979	0.44	0.085	
2175	75%	179	_	18	05		6925	6945	6973	6979	0.445	0.005	
2175	75%	179	_	18	10		6924	6945	6973	6979	0.4475	0.0025	
2175	75%	179	_	18	20		6924	6945	6973	6979	0.4475	0	
2175	75%	179	_	18	30		6924	6946	6973	6979	0.445	-0.0025	

2175	75%	179	_	18	60		6922	6945	6972	6976	0.4625	0.0175	
2900	100%	239	_	19	00		6903	6932	6960	6960	0.6125	0.15	
2900	100%	239	_	19	05		6902	6933	6960	6961	0.61	-0.0025	
2900	100%	239	_	19	10		6901	6932	6959	6960	0.62	0.01	
2900	100%	239	_	19	20		6901	6933	6959	6960	0.6175	-0.0025	
2900	100%	239	_	19	30		6901	6933	6958	6960	0.62	0.0025	
2900	100%	239	_	19	60		6900	6933	6958	6960	0.6225	0.0025	
3625	125%	298	_	20	00		6880	6924	6949	6949	0.745	0.1225	
3625	125%	298	_	20	05		6881	6925	6949	6950	0.7375	-0.0075	
3625	125%	298	_	20	10		6882	6926	6950	6952	0.725	-0.0125	
3625	125%	298	_	20	20		6882	6927	6949	6951	0.7275	0.0025	
3625	125%	298	_	20	30		6882	6928	6949	6952	0.7225	-0.005	
3625	125%	298	_	20	60		6882	6929	6949	6953	0.7175	-0.005	
4350	150%	358	_	21	00		6865	6918	6940	6940	0.8425	0.125	
4350	150%	358	_	21	05		6868	6921	6940	6941	0.825	-0.0175	
4350	150%	358	_	21	10		6868	6921	6940	6941	0.825	0	
4350	150%	358	_	21	20		6868	6921	6940	6941	0.825	0	
4350	150%	358	_	21	30		6868	6921	6940	6941	0.825	0	
4350	150%	358	_	21	60		6868	6921	6940	6941	0.825	0	
5220	180%	429	_	22	00		6863	6922	6931	6937	**0.8675**	0.0425	
5220	180%	429	_	22	05		6863	6922	6931	6937	0.8675	0	
5220	180%	429	_	22	10		6863	6922	6931	6937	0.8675	0	
5220	180%	429	_	22	20		6861	6920	6929	6937	0.8825	0.015	
5220	180%	429	_	22	30		6860	6920	6929	6937	0.885	0.0025	
5220	180%	429	_	23	00		6859	6920	6929	6937	0.8875	0.0025	
5220	180%	429	_	0	00		6858	6918	6929	6937	0.895	0.0075	
5220	180%	429	_	1	00		6857	6918	6926	6936	0.9075	0.0125	
5220	180%	429	_	2	00		6855	6918	6923	6934	0.925	0.0175	
5220	180%	429	_	3	00		6853	6916	6920	6932	0.9475	0.0225	
5220	180%	429	_	4	00		6851	6914	6917	6930	0.97	0.0225	
5220	180%	429	_	5	00		6848	6914	6914	6928	0.99	0.02	
5220	180%	429	_	6	00		6848	6914	6911	6926	1.0025	0.0125	
5220	180%	429	_	7	00		6848	6912	6908	6924	1.02	0.0175	
5220	180%	429	_	8	00		6848	6912	6905	6922	1.0325	0.0125	
5220	180%	429	_	9	00		6848	6912	6902	6920	1.045	0.0125	
5220	180%	429	_	10	00		6848	6912	6899	6918	1.0575	0.0125	
5220	180%	429	_	11	00		6848	6910	6896	6916	1.075	0.0175	
5220	180%	429	_	12	00		6838	6900	6873	6907	**1.205**	0.13	
4350	150%	358	_	12	00		6858	6911	6875	6931	1.0625	-0.1425	
4350	150%	358	_	12	05		6858	6911	6876	6931	1.06	-0.0025	
4350	150%	358	_	12	10		6856	6911	6883	6933	1.0425	-0.1625	
4350	150%	358	_	12	20		6856	6911	6883	6933	1.0425	0	
4350	150%	358	_	12	30		6856	6911	6883	6933	1.0425	0	
4350	150%	358	_	12	60		6856	6911	6883	6933	1.0425	0	
2900	100%	239	_	13	00		6883	6936	6908	6940	0.8325	-0.21	
2900	100%	239	_	13	05		6885	6936	6908	6940	0.8275	-0.005	
2900	100%	239	_	13	10		6886	6938	6909	6942	0.8125	-0.015	
2900	100%	239	_	13	20		6887	6939	6911	6942	0.8025	-0.01	
2900	100%	239	_	13	30		6887	6939	6911	6942	0.8025	0	
2900	100%	239	_	13	60		6887	6939	6910	6944	0.8	-0.0025	

1450	50%	120	_	14	00		6905	6964	6912	6950	0.6725	-0.1275	
1450	50%	120	_	14	05		6905	6965	6912	6950	0.67	-0.0025	
1450	50%	120	_	14	10		6907	6965	6915	6951	0.655	-0.015	
1450	50%	120	_	14	20		6910	6965	6915	6951	0.6475	-0.0075	
1450	50%	120	_	14	30		6912	6967	6915	6951	0.6375	-0.01	
1450	50%	120	_	14	60		6912	6967	6919	6951	0.6275	-0.01	
0	0%	0	_	15	00		6947	6940	6927	6962	0.56	-0.0675	
0	0%	0	_	15	05		6948	6940	6929	6962	0.5525	-0.0075	
0	0%	0	_	15	10		6949	6942	6930	6962	0.5425	-0.01	
0	0%	0	_	15	20		6955	6949	6946	6962	0.47	-0.0725	
0	0%	0	_	15	30		6958	6951	6942	6962	0.4675	-0.0025	
0	0%	0	_	16	00		6969	6962	6954	6965	0.375	-0.0925	
0	0%	0	_	17	00		6976	6970	6956	6967	0.3275	-0.0475	
0	0%	0	_	18	00		6981	6977	6959	6968	0.2875	-0.04	
0	0%	0	_	19	00		6982	6979	6963	6970	0.265	-0.0225	
0	0%	0	_	20	00		6982	6979	6963	6971	0.2625	-0.0025	
0	0%	0	_	21	00		6986	6985	6967	6974	0.22	-0.0425	
0	0%	0	_	22	00		6985	6984	6969	6974	0.22	0	
0	0%	0	_	23	00		6987	6985	6971	6976	0.2025	-0.0175	

Nib bank project pile no. 2 pile load test data

Project				NIB BANK									
Pile Load Test No.				___02__________		Page:	___01___________						
Pile No.				WP-31		Date:	March 24, 2016		-	March 25, 2016			
Load	**work load**	**Pressure**	**Load Cell**	**time**		**temp.**	**reading dial gauges**				**settlement**		**remarks**
				actual waiting			**d.g. 1**	**d.g.2**	**d.g.3**	**d.g.4**	**total**	**w. time**	
KN	**%**	**bar**		**h**	**min**	**oc**	**mm**	**mm**	**mm**	**mm**	**mm**	**mm**	
0	0%	0		12	00		7000	7000	7000	7000	0	0	
725	25%	60		12	00		6980	6846	6897	7035	0.605	0.605	
725	25%	60		12	05		6976	6841	6892	7034	0.6425	0.0375	
725	25%	60		12	10		6978	6842	6892	7035	0.6325	-0.01	
725	25%	60		12	20		6979	6842	6892	7036	0.6275	-0.005	
725	25%	60		12	30		6979	6842	6892	7036	0.6275	0	
725	25%	60		12	60		6979	6840	6889	7035	0.6425	0.015	
1450	50%	120		13	00		6953	6767	6835	7026	1.0475	0.405	
1450	50%	120		13	05		6953	6766	6833	7025	1.0575	0.01	
1450	50%	120		13	10		6952	6764	6830	7024	1.075	0.0175	
1450	50%	120		13	20		6949	6762	6826	7022	1.1025	0.0275	
1450	50%	120		13	30		6941	6752	6825	7011	1.1775	0.075	
1450	50%	120		13	60		6941	6752	6825	7011	1.1775	0	
2175	75%	179		14	00		6925	6729	6786	7006	1.385	0.2075	
2175	75%	179		14	05		6925	6729	6782	7002	1.405	0.02	
2175	75%	179		14	10		6925	6729	6782	7002	1.405	0	
2175	75%	179		14	20		6925	6729	6782	7002	1.405	0	
2175	75%	179		14	30		6925	6729	6782	7002	1.405	0	
2175	75%	179		14	60		6925	6729	6782	7002	1.405	0	
2900	100%	239		15	00		6900	6705	6745	6980	1.675	0.27	
2900	100%	239		15	05		6899	6702	6738	6975	1.715	0.04	
2900	100%	239		15	10		6894	6700	6733	6977	1.74	0.025	
2900	100%	239		15	20		6894	6700	6733	6977	1.74	0	
2900	100%	239		15	30		6889	6694	6723	6965	1.8225	0.0825	
2900	100%	239		15	60		6884	6689	6718	6960	**1.8725**	0.05	
1450	50%	120		16	00		6922	6724	6753	6990	1.5275	-0.345	
1450	50%	120		16	05		6922	6724	6753	6990	1.5275	0	
1450	50%	120		16	10		6921	6728	6754	6989	1.52	-0.0075	
1450	50%	120		16	20		6924	6730	6756	6991	1.4975	-0.0225	
1450	50%	120		16	30		6926	6732	6758	6993	1.4775	-0.02	
1450	50%	120		16	60		6927	6733	6760	6994	1.465	-0.0125	
0	0%	0		17	00		6957	6753	6809	6990	1.2275	-0.2375	
0	0%	0		17	05		6959	6755	6811	6992	1.2075	-0.02	
0	0%	0		17	10		6961	6757	6813	6994	1.1875	-0.02	
0	0%	0		17	20		6963	6759	6815	6996	1.1675	-0.02	
0	0%	0		17	30		6965	6761	6817	6998	1.1475	-0.02	
0	0%	0		17	60		6950	6765	6828	7004	1.1325	-0.015	
1450	50%	120		18	00		6946	6741	6774	7008	1.3275	0.195	
1450	50%	120		18	05		6935	6740	6773	7006	1.365	0.0375	
1450	50%	120		18	10		6934	6740	6772	7006	1.37	0.005	
1450	50%	120		18	20		6935	6740	6772	7007	1.365	-0.005	
1450	50%	120		18	30		6935	6740	6772	7006	1.3675	0.0025	
1450	50%	120		18	60		6933	6740	6772	7004	1.3775	0.01	
2175	75%	179		19	00		6916	6724	6755	6993	1.53	0.1525	
2175	75%	179		19	05		6915	6724	6755	6994	1.53	0	
2175	75%	179		19	10		6916	6724	6755	6993	1.53	0	
2175	75%	179		19	20		6914	6724	6755	6994	1.5325	0.0025	
2175	75%	179		19	30		6912	6722	6753	6992	1.5525	0.02	

2175	75%	179		19	60		6910	6720	6751	6990	1.5725	0.02	
2900	100%	239		20	00		6902	6711	6742	6986	1.6475	0.075	
2900	100%	239		20	05		6902	6710	6742	6986	1.65	0.0025	
2900	100%	239		20	10		6902	6710	6740	6986	1.655	0.005	
2900	100%	239		20	20		6902	6710	6740	6986	1.655	0	
2900	100%	239		20	30		6903	6710	6740	6986	1.6525	-0.0025	
2900	100%	239		20	60		6903	6710	6740	6986	1.6525	0	
3625	125%	298		21	00		6889	6694	6724	6976	1.7925	0.14	
3625	125%	298		21	05		6889	6693	6723	6976	1.7975	0.005	
3625	125%	298		21	10		6889	6693	6721	6976	1.8025	0.005	
3625	125%	298		21	20		6887	6690	6718	6974	1.8275	0.025	
3625	125%	298		21	30		6888	6691	6717	6974	1.825	-0.0025	
3625	125%	298		21	60		6881	6683	6707	6967	1.905	0.08	
4350	150%	358		22	00		6864	6664	6689	6957	2.065	0.16	
4350	150%	358		22	05		6863	6663	6687	6956	2.0775	0.0125	
4350	150%	358		22	10		6863	6662	6684	6955	2.09	0.0125	
4350	150%	358		22	20		6863	6662	6682	6954	2.0975	0.0075	
4350	150%	358		22	30		6863	6662	6681	6954	2.1	0.0025	
4350	150%	358		22	60		6862	6660	6676	6953	2.1225	0.0225	
5220	180%	429		23	00		6834	6635	6655	6940	**2.34**	0.2175	
5220	180%	429		23	05		6832	6634	6652	6938	2.36	0.02	
5220	180%	429		23	10		6831	6634	6651	6937	2.3675	0.0075	
5220	180%	429		23	20		6829	6632	6648	6935	2.39	0.0225	
5220	180%	429		23	30		6828	6632	6645	6934	2.4025	0.0125	
5220	180%	429		0	00		6828	6632	6644	6934	2.405	0.0025	
5220	180%	429		1	00		6824	6629	6638	6930	2.4475	0.0425	
5220	180%	429		2	00		6823	6628	6636	6929	2.46	0.0125	
5220	180%	429		3	00		6823	6627	6635	6929	2.465	0.005	
5220	180%	429		4	00		6823	6627	6634	6929	2.4675	0.0025	
5220	180%	429		5	00		6823	6627	6634	6929	2.4675	0	
5220	180%	429		6	00		6821	6625	6632	6927	2.4875	0.02	
5220	180%	429		7	00		6819	6623	6630	6925	2.5075	0.02	
5220	180%	429		8	00		6819	6621	6628	6924	2.52	0.0125	
5220	180%	429		9	00		6817	6618	6625	6924	2.54	0.02	
5220	180%	429		10	00		6812	6612	6619	6918	2.5975	0.0575	
5220	180%	429		11	00		6810	6611	6617	6917	**2.6125**	0.015	
4350	150%	358		11	00		6822	6623	6625	6923	2.5175	-0.095	
4350	150%	358		11	05		6823	6625	6626	6923	2.5075	-0.01	
4350	150%	358		11	10		6819	6621	6621	6919	2.55	0.0425	
4350	150%	358		11	20		6819	6621	6621	6919	2.55	0	
4350	150%	358		11	30		6815	6617	6618	6915	2.5875	0.0375	
4350	150%	358		11	60		6815	6617	6618	6915	2.5875	0	
2900	100%	239		12	00		6844	6648	6638	6930	2.35	-0.2375	
2900	100%	239		12	05		6843	6648	6638	6929	2.355	0.005	
2900	100%	239		12	10		6841	6647	6636	6927	2.3725	0.0175	
2900	100%	239		12	20		6840	6645	6634	6924	2.3925	0.02	
2900	100%	239		12	30		6835	6641	6630	6921	2.4325	0.04	
2900	100%	239		12	60		6831	6637	6625	6915	2.48	0.0475	
1450	50%	120		13	00		6861	6671	6648	6930	2.225	-0.255	
1450	50%	120		13	05		6861	6670	6647	6927	2.2375	0.0125	
1450	50%	120		13	10		6861	6670	6647	6929	2.2325	-0.005	
1450	50%	120		13	20		6863	6672	6649	6931	2.2125	-0.02	
1450	50%	120		13	30		6865	6674	6651	6933	2.1925	-0.02	
1450	50%	120		13	60		6867	6676	6653	6935	2.1725	-0.02	
0	0%	0		14	00		6867	6676	6653	6935	2.1725	0	
0	0%	0		14	05		6867	6676	6653	6935	2.1725	0	
0	0%	0		14	10		6909	6681	6660	6888	2.155	-0.0175	
0	0%	0		14	20		6910	6682	6663	6889	2.14	-0.015	
0	0%	0		14	30		6911	6684	6666	6892	2.1175	-0.0225	
0	0%	0		15	00		6912	6686	6670	6895	2.0925	-0.025	
0	0%	0		16	00		6914	6689	6675	6899	2.0575	-0.035	
0	0%	0		17	00		6920	6696	6685	6907	1.98	-0.0775	
0	0%	0		18	00		6930	6704	6694	6916	1.89	-0.09	
0	0%	0		19	00		6929	6705	6696	6915	1.8875	-0.0025	
0	0%	0		20	00		6929	6705	6696	6916	1.885	-0.0025	
0	0%	0		21	00		6931	6707	6700	6919	1.8575	-0.0275	
0	0%	0		22	00		6932	6708	6701	6921	**1.845**	-0.0125	

Zemen bank project pile no. 1 pile load test data

Project___ zemen bank

Pile Load Test No. ____________ Page: ___________

Pile No. _____________________ Date: ___________

Load	Work Load	Pressure	Load cell	Time		Temp.	Reading dial gauges				Settlement		Remarks
				Actual	Waiting		dial gage_1	dial gage_2	dial gage_3	dial gage_4	Total	W.time	
KN	%	bar		h	min.	^{0}C	mm	mm	mm	mm	mm	mm	
0	0	0		7	00		8000	8000	8000	8000	0		
813	25	191		7	00		7820	7835	7921	7964	1.150	-	
813					05		7811	7830	7920	7966	1.183	0.033	
813					10		7810	7828	7920	7966	1.190	0.007	
813					20		7810	7827	7920	7966	1.193	0.002	
813					30		7809	7826	7920	7965	1.200	0.008	
813					60		7808	7825	7919	7964	1.210	0.010	
1625	50	254		8	00		7748	7780	7912	7995	1.413	-	
1625					05		7755	7778	7910	7996	1.403	-0.010	
1625					10		7744	7777	7911	7996	1.430	0.027	
1625					20		7743	7776	7910	7996	1.438	0.008	
1625					30		7741	7774	7909	7995	1.453	0.015	
1625					60		7738	7771	7906	7994	1.478	0.025	
2438	75	127		9	00		7707	7730	7873	7945	1.863	-	
2438					05		7706	7727	7871	7944	1.880	0.017	
2438					10		7705	7726	7870	7943	1.890	0.010	
2438					20		7700	7721	7866	7940	1.933	0.043	
2438					30		7699	7719	7863	7941	1.945	0.013	
2438					60		7699	7719	7862	7940	1.950	0.005	
3250	100	0		10	00		7671	7678	7833	7885	2.333	-	
3250					05		7668	7672	7828	7878	2.385	0.052	
3250					10		7667	7671	7827	7878	2.393	0.008	
3250					20		7664	7667	7824	7875	2.425	0.032	
3250					30		7664	7665	7826	7874	2.428	0.003	
3250					60		7663	7664	7824	7873	2.440	0.012	
1625	50	64		11	00		7695	7708	7853	7914	2.075	-	
1625					05		7697	7710	7854	7915	2.060	-0.015	
1625					10		7697	7710	7853	7915	2.063	0.002	
1625					20		7693	7709	7852	7914	2.080	0.018	
1625					30		7692	7709	7846	7914	2.098	0.018	
1625					60		7695	7709	7852	7919	2.063	-0.035	
0	0	0		12	00		7822	7784	7980	7970	1.110	-	
0					05		7823	7801	7984	7971	1.053	-0.058	
0					10		7824	7802	7885	7971	1.295	0.243	
0					20		7827	7804	7887	7972	1.275	-0.020	
0					30		7828	7805	7888	7972	1.268	-0.007	
0					60		7829	7805	7889	7973	1.260	-0.008	
813	25	64		13	00		7759	7777	7888	7947	1.573	-	
813					05		7762	7776	7889	7946	1.568	-0.005	
813					10		7762	7776	7889	7947	1.565	-0.002	
813					20		7754	7776	7882	7946	1.605	0.040	
813					30		7751	7774	7878	7946	1.628	0.023	
813					60		7751	7774	7877	7946	1.630	0.002	
1625	50	0		14	00		7716	7738	7856	7935	1.888	-	
1625					05		7717	7737	7862	7936	1.870	-0.017	

1625					10		7719	7737	7864	7838	2.105	0.235	
1625					20		7719	7737	7864	7937	1.858	-0.248	
1625					30		7718	7737	7864	7938	1.858	0.000	
1625					60		7718	7736	7865	7939	1.855	-0.002	
2438	75	0		15	00		7691	7700	7842	7904	2.158	-	
2438					05		7691	7700	7844	7905	2.150	-0.008	
2438					10		7691	7699	7846	7905	2.148	-0.002	
2438					20		7693	7699	7847	7905	2.140	-0.007	
2438					30		7691	7699	7844	7905	2.153	0.012	
2438					60		7692	7699	7847	7905	2.143	-0.010	
3250	100	0		16	00		7668	7664	7823	7874	2.428	-	
3250					05		7666	7661	7823	7871	2.448	0.020	
3250					10		7666	7660	7823	7870	2.453	0.005	
3250					20		7666	7659	7824	7869	2.455	0.002	
3250					30		7666	7658	7824	7869	2.458	0.002	
3250					60		7666	7658	7826	7869	2.453	-0.005	
3900	120	0		17	00		7645	7626	7803	7833	2.733	-	
3900					05		7640	7621	7798	7826	2.788	0.055	
3900					10		7640	7620	7798	7825	2.793	0.005	
3900					20		7641	7619	7798	7823	2.798	0.005	
3900					30		7641	7618	7799	7823	2.798	0.000	
3900					60		7640	7615	7798	7820	2.818	0.020	
4875	150			18	00		7604	7559	7756	7754	3.318	0.500	
4875					05		7600	7551	7752	7745	3.380	0.063	
4875					10		7698	7547	7749	7740	3.165	-0.215	
4875					20		7597	7544	7747	7736	3.440	0.275	
4875					30		7594	7539	7744	7731	3.480	0.040	
4875					60		7593	7536	7742	7728	3.503	0.023	
4875				19	30		7591	7532	7740	7725	3.530	0.027	
4875				20	00		7591	7531	7740	7724	3.535	0.005	
4875				21	00		7590	7530	7739	7722	3.548	0.012	
4875				22	00		7589	7528	7739	7721	3.558	0.010	
4875				23	00		7590	7527	7739	7720	3.560	0.002	
4875				24	00		7590	7526	7739	7720	3.563	-	
4875				1	00		7589	7525	7738	7719	3.573	0.010	
4875				2	00		7588	7524	7738	7717	3.583	0.010	
4875				3	00		7588	7524	7738	7716	3.585	0.002	
4875				4	00		7588	7523	7738	7716	3.588	0.002	
4875				5	00		7587	7522	7737	7716	3.595	0.008	
4875				6	00		7587	7522	7737	7716	3.595	-	
4875				7	00		7587	7522	7737	7716	3.595	0.000	
3250	100	254		7	00		7618	7572	7773	7770	3.168	-0.428	
3250					05		7618	7572	7773	7770	3.168	0.000	
3250					10		7618	7573	7773	7770	3.165	-0.002	
3250					20		7620	7575	7774	7772	3.148	-0.018	
3250					30		7620	7575	7774	7772	3.148	-	
3250					60		7620	7576	7778	7773	3.133	-0.015	
1625	50	0		8	00		7671	7655	7825	7850	2.498	-0.635	
1625					05		7674	7660	7828	7855	2.458	-0.040	
1625					10		7670	7661	7828	7856	2.463	0.005	
1625					20		7674	7662	7828	7857	2.448	-0.015	
1625					30		7674	7662	7827	7858	2.448	0.000	
1625					60		7675	7664	7827	7859	2.438	-0.010	
0	0	0		9	00		7824	7776	7872	7930	1.495	-0.943	
0					05		7825	7777	7871	7930	1.493	-0.003	
0					10		7828	7778	7874	7931	1.473	-0.020	
0					20		7827	7778	7872	7931	1.480	0.008	
0					30		7827	7779	7870	7931	1.483	0.002	
0				10	00		7829	7780	7869	7932	1.475	-0.007	
0				11	00		7829	7783	7869	7932	1.468	-0.008	
0				0	00		7832	7783	7870	7932	1.458	-0.010	
0				1	00						80.000	78.543	

Appendix-C- Data File of the Proposed Numerical Model for Nib Bank Project

```
new
set fish autocreate off
set fish safe on
;layer 1
gen zone radcyl p0 (0,0,0) p1 (30,0,0) p2 (0,0,-4.3) p3 (0,30,0) p4 (30,0,-4.3) &
p5 (0,30,-4.3) p6 (30,30,0) p7 (30,30,-4.3) p8 (0.4,0,0) p9 (0,0.4,0) &
p10 (0.4,0,-4.3) p11 (0,0.4,-4.3)&
size 3 4 11 30 ratio 1 1 1 1.15
group layer1

;layer 2
gen zone radcyl p0 (0,0,-4.3) p1 (30,0,-4.3) p2 (0,0,-9) p3 (0,30,-4.3) p4 (30,0,-9) &
p5 (0,30,-9) p6 (30,30,-4.3) p7 (30,30,-9) p8 (0.4,0,-4.3) p9 (0,0.4,-4.3) &
p10 (0.4,0,-9) p11 (0,0.4,-9)&
size 3 5 11 30 ratio 1 1 1 1.15
group layer2 range group layer1 not

;layer 3
gen zone radcyl p0 (0,0,-9) p1 (30,0,-9) p2 (0,0,-13.7) p3 (0,30,-9) p4 (30,0,-13.7) &
p5 (0,30,-13.7) p6 (30,30,-9) p7 (30,30,-13.7) p8 (0.4,0,-9) p9 (0,0.4,-9) &
p10 (0.4,0,-13.7) p11 (0,0.4,-13.7)&
size 3 3 11 30 ratio 1 1 1 1.15
group layer3 range = group layer1 not, group layer2 not

;layer 4
gen zone radcyl p0 (0,0,-13.7) p1 (30,0,-13.7) p2 (0,0,-20.25) p3 (0,30,-13.7) p4 (30,0,-20.25) &
p5 (0,30,-20.25) p6 (30,30,-13.7) p7 (30,30,-20.25) p8 (0.4,0,-13.7) p9 (0,0.4,-13.7) &
p10 (0.4,0,-20.25) p11 (0,0.4,-20.25)&
size 3 7 11 30 ratio 1 1 1 1.15
group layer4 range = group layer1 not, group layer2 not, group layer3 not

;layer 5
gen zone radcyl p0 (0,0,-20.25) p1 (30,0,-20.25) p2 (0,0,-21) p3 (0,30,-20.25) p4 (30,0,-21) &
p5 (0,30,-21) p6 (30,30,-20.25) p7 (30,30,-21) p8 (0.4,0,-20.25) p9 (0,0.4,-20.25) &
p10 (0.4,0,-21) p11 (0,0.4,-21)&
size 3 1 11 30 ratio 1 1 1 1.15
;layer
gen zone radcyl p0 (0,0,-21) p1 (30,0,-21) p2 (0,0,-42) p3 (0,30,-21) p4 (30,0,-42) &
p5 (0,30,-42) p6 (30,30,-21) p7 (30,30,-42) p8 (0.4,0,-21) p9 (0,0.4,-21) &
p10 (0.4,0,-42) p11 (0,0.4,-42)&
size 3 21 11 30 ratio 1 1 1 1.15 fill
group layer5 range = group layer1 not, group layer2 not, group layer3 not, group layer4 not
;
```

```
gen zone reflect dip 270 dd 90
range name=layer1 group layer1
range name=layer2 group layer2
range name=layer3 group layer3
range name=layer4 group layer4
range name=layer5 group layer5
interface 1 face range cylinder end1 (0,0,0) end2 (0,0,-4.3) radius .41 &
cylinder end1 (0,0,0) end2 (0,0,-4.3) radius .39 not
interface 2 face range cylinder end1 (0,0,-4.3) end2 (0,0,-9) radius .41 &
cylinder end1 (0,0,-4.3) end2 (0,0,-9) radius .39 not
interface 3 face range cylinder end1 (0,0,-9) end2 (0,0,-13.7) radius .41 &
cylinder end1 (0,0,-9) end2 (0,0,-13.7) radius .39 not
interface 4 face range cylinder end1 (0,0,-13.7) end2 (0,0,-20.25) radius .41 &
cylinder end1 (0,0,-13.7) end2 (0,0,-20.25) radius .39 not
interface 5 face range cylinder end1 (0,0,-20.25) end2 (0,0,-21.1) radius .41 &
cylinder end1 (0,0,-20.25) end2 (0,0,-21.1) radius .39 not
;
interface 6 face range cylinder end1 (0,0,-20.9) end2 (0,0,-21.1) radius .41
;
gen zone cyl p0 (0,0,22) p1 (.4,0,22) p2 (0,0,1) p3 (0,.4,22) &
p4 (.4,0,1) p5 (0,.4,1) &
size 3 41 11
gen zone cyl p0 (0,0,22.1) p1 (.4,0,22.1) p2 (0,0,22) p3 (0,.4,22.1) &
p4 (.4,0,22) p5 (0,.4,22) &
size 3 1 11
gen zone reflect dd 90 dip 270 range z 1 22.1
group pile range z 1 22.1
ini z add -22.0 range group pile
save pile_geomnib210.8.sav
pause
;
model mech mohr range group layer1
prop poiss 0.3 coh 0. fric 41.4 range group layer1
prop young 30.6e6 grad 0,0,-3516703.933 range z -4.3 0.
pause
;
model mech mohr range group layer2
prop poiss 0.3 coh 100000. fric 43 range group layer2
prop young 27913244.32 grad 0,0,-4141530.834 range z -9 -4.3
;
model mech mohr range group layer3
prop poiss 0.3 coh 0. fric 42 range group layer3
prop young 46066045.69 grad 0,0,-2124552.905 range z -13.7 -9
;
model mech mohr range group layer4
prop poiss 0.3 coh 50000. fric 42.5 range group layer4
prop young 47470728.66 grad 0,0,-2022021.3 range z -20.25 -13.7
```

```
model mech mohr range group layer5
prop poiss 0.3 coh 100000. fric 45 range group layer5
prop young 58536017.27 grad 0,0,-1475587.295 range z -20.25 -42
;
;
model mech elas range group pile
prop young 75172420.48 poiss 0.3 range group pile
interface 1 prop kn 2.15e10 ks 2.15e10 fric 30.44 coh 0.
interface 2 prop kn 2.15e10 ks 2.15e10 fric 31.87 coh 66666.67
interface 3 prop kn 2.15e10 ks 2.15e10 fric 30.98 coh 0.
interface 4 prop kn 2.15e10 ks 2.15e10 fric 31.42 coh 33333.33
interface 5 prop kn 2.15e10 ks 2.15e10 fric 33.69 coh 66666.67
interface 6 prop kn 5.4232e11 ks 5.4232e11 fric 45 coh 100000.;
;
ini dens 700 range group layer1
ini dens 1400 range group layer2
ini dens 800 range group layer3
ini dens 1200 range group layer4
ini dens 1400 range group layer5
;
;
ini dens 1000 range group pile
model mech null range z -0.1 0.15
;
fix z range z -42.1 -41.9
fix x range x -30.1 -29.9
fix x range x 29.9 30.1
fix y range y -.1 .1
fix y range y 29.9 30.1
set grav 0 0 -10
ini szz 0. grad 0 0 17000. range z -4.3 0.
ini szz 30100. grad 0 0 24000. range z -9 -4.3
ini szz -23900. grad 0 0 18000. range z -13.7 -9
ini szz 30900. grad 0 0 22000. range z -20.25 -13.7
ini szz 71400. grad 0 0 24000. range z -20.25 -42
;
ini sxx 0. grad 0 0 5440. range z -4.3 0.
ini sxx 12943. grad 0 0 10320. range z -9 -4.3
ini sxx -10277. grad 0 0 7740. range z -13.7 -9
ini sxx 13287. grad 0 0 9460. range z -20.25 -13.7
ini sxx 30702. grad 0 0 10320. range z -20.25 -42
;
ini syy 0. grad 0 0 5440. range z -4.3 0.
ini syy 12943. grad 0 0 10320. range z -9 -4.3
ini syy -10277. grad 0 0 7740. range z -13.7 -9
ini syy 13287. grad 0 0 9460. range z -20.25 -13.7
ini syy 30702. grad 0 0 10320. range z -20.25 -42
```

```
water density 1000
water table origin 0,0,0 normal 0 0 -1
ini dens 1700 range group layer1
ini dens 2400 range group layer2
ini dens 1800 range group layer3
ini dens 2200 range group layer4
ini dens 2400 range group layer5
hist add unbal
;
solve rat 1.e-6
save pilen21
;
model mech elas range group pile
prop young 25e9 poiss 0.15 range group pile
ini dens 2500 range group pile
;
def find_add
global top_head = null
loop foreach local gp_pnt gp_list
if gp_zpos(gp_pnt) > 0.05 then
local mem_head = get_mem(2)
mem(mem_head) = top_head
mem(mem_head+1) = gp_pnt
top_head = mem_head
endif
endloop
end
@find_add
solve rat 1.e-7
save pilen22
;
ini state 0
ini xdis 0 ydis 0 zdis 0
;
; monitor vertical loading at pile cap
def zs_top
local ad = top_head
local zftot = 0.0
loop while ad # null
local gp_pnt = mem(ad+1)
local zf = gp_zfunbal(gp_pnt)
zftot = zftot + zf
ad = mem(ad)
endloop
zs_top = zftot / 0.2513
end
fix z range z 0.05 .15 group pile
```

```
;
def init_ramp
global udapp = 0
global udmax = -1e-8
global ncut = 30000
end
@init_ramp
def ramp
while_stepping
if step < ncut then
udapp = float(step) * udmax / float(ncut)
else
udapp = udmax
endif
local ad = top_head
loop while ad # null
local gp_pnt = mem(ad+1)
gp_zvel(gp_pnt) = udapp
ad = mem(ad)
endloop
end
;
hist add gp zdis 0,0,0
hist add gp zvel 0,0,0
hist add fish @zs_top
hist add zone szz 0,0,-.1
;
set mech damp comb
step 2200000
save pilenibfinal
ret
;;;;;;;;;;;;;
```