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DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

SIMULATION OF PILE LOAD TEST USING FINITE ELEMENT METHOD.

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Declaration

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

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Dedicated to My Father

Zeleke Demissie

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LIST OF SYMBOLS

E	Young's modulus
G	Shear modulus
γ:	Unit weight of the soil
k_0:	Coefficient of earth pressure at rest
p	Mean stress
q	Deviator stress
φ':	effective friction angle of the soil
c:	cohesion of the soil
ν:	Poisson’s ratio of the soil
σ:	normal stress
σ_z:	vertical stress
τ:	shear stress
d:	Intersection of yield surface Fs with the t-axis
β:	Slope of the yield surface Fs in the p-t plane
C_c:	Compression index
C_s:	Swelling index
λ:	Slop of normal consolidation line in e-lnp plane (reloading)
κ:	Slop of normal consolidation line in e-lnp plane (un-loading)
ψ	Angle of dilatancy of a soil
μ:	Stiffness coefficient
w:	Stiffness exponent
p_a:	Atmospheric pressure
$\varepsilon^p{}_v$	Volumetric plastic strain
$\varepsilon^{pl}{}_{vo}$	Initial cap position
R:	Cap eccentricity
K:	Flow stress ratio
P_b:	Mean effective yield stress
LL:	Liquid limit

W: Natural moisture content

L: length of the pile

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ABSTRACT

Pile foundations are widely used in different construction site, to support loads from superstructures. Study on predicting bearing capacity and other parameters of pile foundations is still attracting interests of geotechnical researchers. Besides the laboratory or field tests, finite element method is used increasingly to deal with such problems.

In this thesis, simulation of pile loading test by using finite element method is dealt with in detail for different loading and soil conditions. The soil- pile interaction is best simulated by different constitutive models and equations that are based on experimental findings and embodied in the numerical analysis such as the finite element method.

An axisymmetric finite element model using the program ABAQUS is used in order to establish numerical models and analysis procedures that help to simulate actual pile load tests.

In the finite element analysis, the pile is assumed to be linear elastic and for the different soil layers two constitutive models, namely, Mohr Coulomb and Cap Plasticity Model are used. Influence of mesh and interface elements is discussed and the result of the FEM analysis are compared with results of actual pile load tests.

A parametric study is done based on soil finding that affect the load settlement behavior of the pile.

KEYWORDS: constitutive model, finite element method (FEM), soil pile interaction, cap hardening, pile foundation.

1 INTRODUCTION

1.1 Background of the study

Pile foundations are often used in weaker soils to transfer the loads of superstructures to an underlying ground, aiming at increasing the bearing capacity of the soil or lessen the settlement of infrastructures. Hence, determining the capacity of piles is one of the crucial step in system of pile foundation.
The bearing capacity of isolated piles may be determined from one of the following methods:

- loading tests
- prevailing Building Codes
- sounding tests
- dynamic pile-driving formulas
- analytical methods.

Recently, with the rapid development of computational technology, numerical analysis involving finite element method (FEM) is widely used to understand the behavior of pile soil interactions. The advantage of numerical analysis methods lies in their ability to address complex soil formations and the interaction between soil and structures.

In this thesis, simulation of pile loading test, by using the software ABAQUS is examined in detail. First, the available constitutive models representing the soil-pile interaction are discussed; then based on the different constitutive models, the load settlement curves of the FEM simulation are compared with actual load test results. Finally a parametric study on one of the models is carried out by varying stiffness modulus, angle of internal friction and angle of dilatancy of the different soil layers.

1.2 Statement of the Problem

Pile foundation is one of the common deep foundation techniques that are used in soft soil strata to support the super structural loads without any detrimental settlement and bearing failures. To know the ultimate capacities of the pile, loading tests are usually performed in different projects. However the cost of running this tests and the time it takes is one of the difficulties that engineers faced in current geotechnical practices. So in this study the possibility of applying a finite element model to simulate the pile load test is examined in order to compare results with load tests and to use the model as one alternative for determining pile capacities, at least for preliminary design purposes.

1.2.1 Objective

The objectives of this study are:

- simulating a pile load test using finite element method and to compare output with actual pile load test results.
- showing the possibility of application of finite element method to develop load settlement curve
- studying the effects of different soil parameters on load-settlement behavior

1.2.2 Methodology

Interaction of piles with soil layers is the mechanism that enables the transfer of loads from superstructure to the adjacent soil formation. Studying this interaction and trying to simulate it using the finite element model is the prime goal of this study.

Collection of soil parameters from investigation reports done in the lab and from field tests, and gathering actual pile load test data are the first and foremost steps required to start this study. Different correlation techniques are also used to determine soil parameters that are not included in the soil investigation report.

Once we get all the required soil parameters, by using elasticity theories and plasticity constitutive model, the failure mechanism of the soil is developed and its plastic deformation behavior is studied.

Simulation of the pile - soil interaction based on the different constitutive models are formulated using the software '**ABAQUS 2013**' , which bases its analysis on finite element method. Using these simulation models the output in terms of load settlement curves are compared with actual pile load test data.

1.2.3 Scope of the study

Vertically loaded bore piles are considered in this study for the simulation of pile load test using finite element method. Only loading part of the unloading – reloading procedures in actual load tests are simulated in this study.

With the attempts to achieve the above mentioned research objectives, required data were collected from projects in Addis Ababa, Dire Dawa and Gidabo Dam sites. The data were collected from different soil investigation and design reports of the respective projects and from correlated soil parameters.
The major results and findings of this study are thus applicable for the particular project sites and other comparable soil conditions.

2 LITERATURE REVIEW

2.1 General

The bearing capacity of a single pile is most usually determined by pile load test or by empirical methods. With the current rapid development of numerical analysis, the use of finite element method is attracting engineer's attention towards using this method as one option. Different constitutive models are used to simulate the soil pile interaction following the development of more and more comprehensive constitutive models to describe the complex behavior of geo-material under different loading conditions (Hashiguchi, 1989; Collins and Kelly, 2002).

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 sufficiently accuracy (Chen & Saleeb 1983). The failure envelope, being dependent on the major and minor principle stresses is defined by cohesion, c and internal frictional angle, φ. In the MC model a constant stiffness is used. Whereas the modified Drucker-Prager/Cap plasticity model exhibits pressure- dependent yield. The yield surface includes two main segments: a shear failure surface, providing dominantly shearing flow, and a "cap," which intersects the equivalent pressure stress axis
During the last three decades, pile–soil interaction has been investigated by physical and numerical modeling. Many experimental and analytical studies have been conducted to determine the sources of nonlinearities. Beside these, numerical methods were extensively used to analyze the system using boundary element and finite element methods due to their high accuracy and capability to handle complex problems. However both of these powerful numerical methods require a considerable amount of engineering judgment and computation cost to completely solve the physical problem. Because of these reasons, researchers in this area have focused on simplified and economical models to characterize the problem.

Numerical analyses for simulation of soil-pile interaction can be done in two ways: The first is a continuum-based method like boundary element method or finite element method (FEM), in which continuity of the soil domain is inherent in formulations; while the second method is a load transfer method which models the soil through a set of independent springs attached to the piles. In the first method, the use of Mindlin's solution was used in a linear boundary element formulation (Poulos 1968; Banerjee 1978). Subsequently, Banerjee and Davies (1978) developed linear solutions for piles and pile groups in non-homogenous soils.

FEM approaches were attempted by Desai (1974), Randolph (1981), and Faruque and Desai (1982). Besides them, Pressley and Poulos (1986) used an elastic perfectly plastic soil model in an axisymmetric FEM to approximately analyze pile groups. Three-dimensional analysis of vertically loaded pile groups was studied by Ottaviani (1975). Also, Muqtadir and Desai (1986) analyzed three-dimensional pile groups under vertical loads. Another study was done for a single pile by using the commercial code ABAQUS (Trochanis et al. 1991a,b). They simplified the analysis into a one-dimensional model via the use of spring supported piles.

2.2 PILE LOADING TEST

2.2.1 General

According to their purposes, load tests on foundation pile can be divided into design load tests and proof load tests.

The design load test is usually kept to failure or at least to a maximum load not less than three times the intended service load. It is a destructive test, and has to be carried out on a purposely installed test pile, which doesn't belong to the foundation. The aim of a design load test is to determine, at the design stage, the bearing capacity of the pile and its load settlement relationship. Carlo Viggiani et. al.,2012, has tried to show that, if the pile shaft is properly instrumented, it allows the determination of fraction of the bearing capacity taken by the base and the shaft of the pile.

The proof load test, on the contrary is carried out on piles selected among the piles of the foundation, after they have been all installed. This test cannot be destructive, and hence the maximum test load is usually limited to 2 times the intended service load the total test load shall be applied in increments amounting to 25, 50, 75, 100, 125, 150, 175, and 200 per cent of the anticipated working load. (ASTM D1143 D1143-57 T.)

Proof load test is aimed at verifying the correct installation of piles and also load settlement behavior and determination of bearing capacity may also be obtained.

The piles to be proof tested are selected only after all piles have been installed in order to prevent a particular carful installation of the intended test pile and to obtain an equal care for all the piles.

Usually a bore hole or a CPT is carried out in the vicinity of the test pile, in order to know the exact sub surface profile at the test site. This can be helpful to obtain soil parameters as inputs for the finite element analysis.

In cohesive soils, settlement is a function of the length of time of load application. Since the time in a pile load test is generally too short relative to the time required to approach full settlement. The load test tells nothing about the settlement behavior of a single pile.

In cohesion less soils, the load test will show the settlement behavior of a single pile but tells little of the settlement behavior of the group.

Basically, a pile load test can determine only the ultimate bearing capacity and not the settlement characteristics of the pile group. Settlement computation are a separate matter and the subject of soil mechanics calculation. (Robbert D. Chellis, 1961)

2.2.2 Method used to carry out pile loading test

There are a number of different arrangements of apparatus developed for carrying out loading tests on piles. Different methods are available to apply test loads:

i) by direct loads from a platform on which heavy weights are placed
ii) by direct loads from a platform on which water tanks are placed to be filled as desired
iii) by jacking against a loaded platform
iv) by jacking against existing structure
v) by jacking against previously driven pile
vi) by application of loads by means of a cantilever arm

2.2.3 Plastic and Elastic Deformations of Pile

Movement of the pile head is caused by elastic deformation of pile and soil and plastic deformation of soil. The later causes undue settlement of structure. The plastic deformation is the significant one to be determined from load tests and not primarily the total downward movement of the head of the pile under the test load. The curve of the plastic deformation is the most significant and this is the one from which the working load and factor of safety should be determined. (Robbert D. Chellis, 1961).

By removing the load from the pile several times during the process of adding loads by increment and plotting the rebound, the plastic deformation curve can be obtained.

For further review on the method of pile load test for individual piles under vertical axial load one can refer "ASTM method of test" with serial designation D1143-57 T.

2.2.4 Determination of Limiting Pile Capacity.

In most soil mechanics literatures, the ultimate pile capacity is defined as the load beyond which the pile will begin to settle excessively or expressed mathematically, when $\Delta s/ \Delta p$ approaches infinity, that is when the tangent to the load-settlement curve becomes vertical with the settlement plotted as ordinate. As listed in Robert D. Chellis, 1996, to select the failure point different methods are available as mentioned here under:

In a very few cases, the load settlement curve may show a marked limiting value. But in most cases the load settlement curves do not distinctly show the limiting load. As a result there are numerous suggestions for determining the limiting load.

The limiting load is read from load-total settlement or from load-plastic settlement curves. For this study load –total settlement curve is considered.

From Load Total Settlement Curves

Different researchers and codes suggest various methods of determining the limiting load capacity of a pile, among which some are listed below. (Refer fig 2.1)

(i) The point at which the end tangents of the load-settlement curve meet (Mansur/Kaufmann 1956)
(ii) The point at which the curve begins to show a steeper slope (DIN 1054)
(iii) The point at which the curve manifests the steepest slope i.e., ds/dQ= max (Vesic 1963)

(iv) For a total settlement of 0.1d(Terzaghi/Peck 1961).
For a total settlement of 50mm(Terzaghi/Peck 1948)

For a total settlement of 25mm(New York City Building Law)

For a total settlement of 20mm(Muhs 1959, 1963)

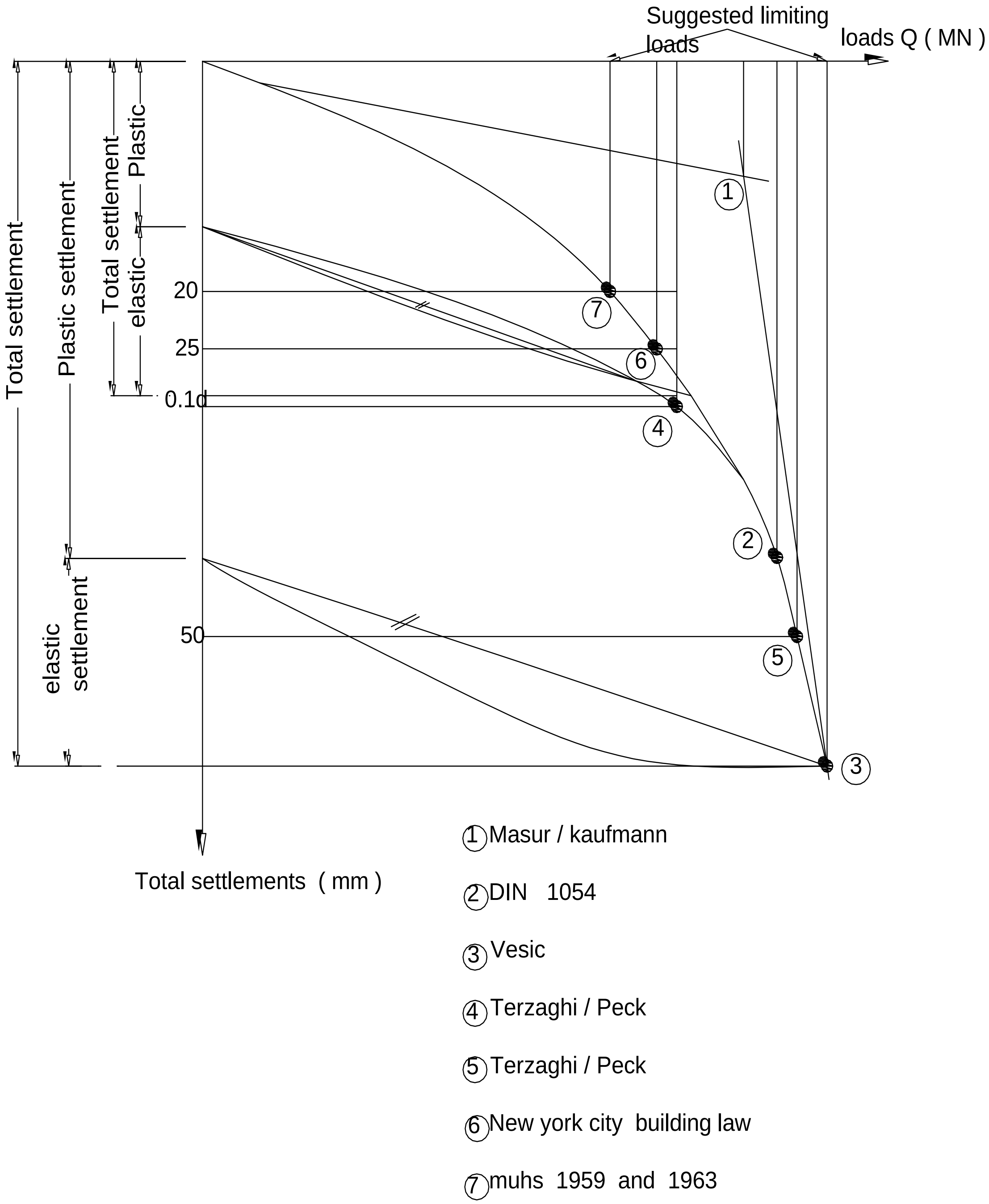

Figure 2.1 Determination of limiting load from pile loading test

2.3 Related Studies

Different researchers are developing a finite element model to simulate and analyze the behavior of interaction of various structures with soils. Some of related studies are outlined below:

Yun-gang et. al, 2012, had studied modeling of vertical bearing capacity of pile foundation by using the software ABAQUS. They have examined the bearing capacity taking a 4-node bilinear axisymmetric solid element and reduced integration techniques to overcome shear locking. For the load transfer mechanism a hard contact with no relative sliding between soil and pile were assumed as provided by the ABAQUS software. `However some researchers have shown that, the use of interface elements in the FEM model will reduce capacity of the pile and increase computational cost and instability in the simulation model. (Gang Wang and Nicholas Sitar, 2004). Mohr –Coulomb constitutive model were used in their study

Study of pile driving by finite element method was carried out by Mabsout and Tassoulas (1994), after that different researchers have tried to study similar areas using FEM. Lymon C. Reese et. al, 1995 are among these researchers. They have used axisymmetric discretization and take into account the non-linear behavior of the soil by the bounding surface plasticity model for clay. They have simulated the driving by tracing the penetration of the pile in to the soil based on the slide line contact algorithm. The impact of the hammer on the pile is represented by a periodic forcing function applied on the top of the pile. Since their study mainly focused on offering a qualitative interpretation of the result of the driving, their analysis make use of suitable assumptions for parameters describing the hammer, pile and the soil.

Drucker-Prager type model (Drucker and Prager, 1952) has been successfully adopted in analysis of geo-materials due to its relative simplicity. For example, a comprehensive nonlinear finite element analysis of vertically loaded pile was carried out using ABAQUS TM (Trochanis et al. 1991). In this study, the surrounding soil was modeled using extended Drucker-Prager plasticity while the piles were modeled as linearly elastic material. Yang and Jeremi_ (2002) used non-associative Drucker-Prager for cohesion less soil and von Mises criterion for cohesive soil, and developed p-y curves for laterally loaded piles in multi-layered soil profiles. Although these previous analyses dealt with pile-soil interaction with various degrees of success, detailed information on model assumptions and uncertainties associated with model selection are not available.

Gang Wang et. at. 2004 had tried to show the effect of soil dilatancy by examining the load deflection response for varying the angle of dilatancy, ψ, where all other parameters are kept similar. Their result shows that the system response is very sensitive to the choice of ψ.

Gang Wang et. at. 2004 had also examined the effect of interface element between the pile and the soil. They used node to node zero length frictional contact element in openness. The contact elements were placed along the shaft and the problem was analyzed. They choose the friction angle of contact element to be the same as soil friction angle, $\emptyset$, for a clear comparison. The significant difference that discussed above for full dilatant and non-dilatant soil is greatly suppressed by the presence of the interface contact elements. Instead of yielding through the Drucker – Prager type soil elements, the contact elements essentially enforce Mohr – Coulomb type failure mode along the shaft. They finally found out that with interface elements, lower capacity of pile resistance is resulted. They also noted that computational costs and numerical instability increases considerably in simulation with interface elements.

Mesh dependency of numerical analysis was seen by M.Wehnert et. al. (2004), from Institute of geotechnical engineering, University of Stuttgart. To study this they performed six analysis, i.e. three different calculations with interface elements and three without interface elements. One with a very fine, one with a medium coarse and one with a very coarse mesh. They using MC model for this study. Their result showed that the difference between calculations with a very fine and a very coarse mesh is essentially for the shaft resistance negligible. Only for the base resistance one can observe a small difference, because for the coarse mesh there is only one element underneath the pile base. For calculation without interface element this difference is bigger.

They have also tried to see the effect of three constitutive models, Mohr Coulomb (MC), and a cap like yield surface introduced in the Soft Soil (SS), and Hardening Soil (HS) models. Comparing the curves of the base resistance, all curves give the same shape. The SS model shows that stiffer than the other two. This is due to the different formulation of the odometer stiffness in the different model. For base resistance different model gave similar result and the choice of a constitutive model in not as important as the right choice of the stiffness. Comparing the shaft resistance, they found out that, the shaft resistance for MC and SS model is increasing more or less linear up to peak. Whereas the pick value of shaft resistance is higher for SS model. In HS model there is no linear increase, the shape of the curve is hyperbolic and the pick value is even higher than the SS model. After the pick the shaft resistance is decreasing for all models.

3 SITE GEOLOGY AND SOIL PARAMETERS

3.1 General.

Soil investigation and detailed material report data are the basic input, for the FEM analysis of different geotechnical problems. Most of geotechnical reports that are usually available and/or used in current design and construction practices are commonly done with the intention of determining only the basic soil parameters like cohesion and angle of internal friction. However other important parameters should be determined to get sufficient input parameters for the FEM model.

Availability of such soil data is the main factor governing the selection of the study area of this thesis. Three different sites are selected with sufficient data for this study in various areas of the country, namely:

- Addis Ababa, K.K. building project, B+G+18 stories
- Dire Dawa, Tiwan bridge project
- S.N.N.P. Regional State, Gidabo Dam project

Two pile loading tests were done for each site, and the required geotechnical report have been acquired. The detailed site characterization and soil parameters are presented as follow.

3.2 K.K. Mixed Use Building Project

K.K. mixed use building project is located in central Addis Ababa, Kirkos sub city, around the national theater on the right hand side of the main road leading to Mexico square.

Construction Design Share Company has carried out the soil investigation task to determine the engineering properties of the soil strata and the nature of the subsurface geological material.

Core drilling of seven boreholes (with a maximum drilling depth of 50m) along with in situ and laboratory tests have been conducted. Accordingly the following engineering geological layers have been identified.

1. Top soil
2. Successive flow basalt
3. Red silty clay
4. Decomposed tuff and scoria

In all bore holes, the ground water level has been found between 6m to 7.5m below the natural ground floor level.

Hence in the analysis submerged unit weight of the soil is taken for layers located below the ground water level.

From geotechnical point of view, the site is subdivided in to eight major layers. The top 8m depth is expected to be excavated for the construction of the two basement floors. Only layers below 8m are considered, and the summarized soil properties of the different layers are shown in Table 3.1

Two pile loading test were performed by ANCOR FOUNDATION on working piles cast as part of the foundation system. 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 09/10/2007 with a diameter of 800mm to a depth of 16.3m from working platform level. The reinforcement of the pile consists of 12 numbers of vertical main bars of diameter 24mm. The pile was covered by helix diameter 10 mm with a spacing of 100 mm over the length of the reinforcement and a 50 mm concrete cover was maintained by concrete spacers attached to the helical stirrups at 3.0 m intervals over the length of the reinforcement. The concrete quality is not specified in the geotechnical report. Details of the drilling are recorded in the pile drilling and concreting record. See Annex B

Working pile 2:

The test pile has been drilled and cast with concrete on 10.11.07 with a diameter of 1000mm to a depth of 15.4m from working platform level. The reinforcement of the pile consists of 20 nos. of vertical main bars diameter of 25mm. The pile was covered by helix diameter 10 mm with a spacing of 100 mm over the length of the reinforcement and a 50 mm concrete cover was maintained by concrete spacers attached to the helical stirrups at 3.0 m intervals over the length of the reinforcement. Details of the drilling are recorded in the pile drilling and concreting record.

The load vs settlement curve of the piles are shown in Fig. 3.1 and Fig. 3.2

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

Table 3.1 Description of soil strata (K.K. project), (obtained from the soil investigation report by CDSC)

Layer No.	**Description**	**Depth (m)**	**Bulk unit weight (γ)**	**submerged unit weight (γ')**	**Friction angle ($\emptyset$)**	**Cohesion (c)**	**S.P.T. (N- values)**	**Poisons ratio (ν)**
1	Gray to brownish, highly weathered decomposed weak basalt	4.4-8.7	18.0 kN/m^3	8.0kN/m^3	30^{O}	-	49	0.3
2	Gray, slightly weathered , fractured strong basalt	8.7-15.0	20.0kN/m^3	10.0kN/m^3	35^{O}	-	>50	0.3
3	stiff reddish brown to yellowish silty clay	15.0-19.2	17.0kN/m^3	7.0kN/m^3	27^{O}	19kPa	>50	0.3
4	Gray, slightly weathered, moderately strong basalt	19.2-26.0	24.26kN/m^3	14.26kN/m^3	38^{O}	-	>50	0.3
5	Medium dense, yellowish decomposed tuff	26.0-28.2	16.3kN/m^3	6.3kN/m^3	27^{o}	16KPa	>50	0.3
6	Dense to very dense reddish brown scoria	30.5-35.0	15.2 kN/m^3	5.2 kN/m^3	27^{o}	19KPa	>50	0.3

The above table shows general description of different soil layer formation and parametric values which are results of soil investigation report. Shear strength values shown in the above table are not directly used in numerical analysis of this study, as it seems to underestimate the shear strength of the described soil layer. Detail method of calculation of shear strength parameters are shown in the next sub topics.

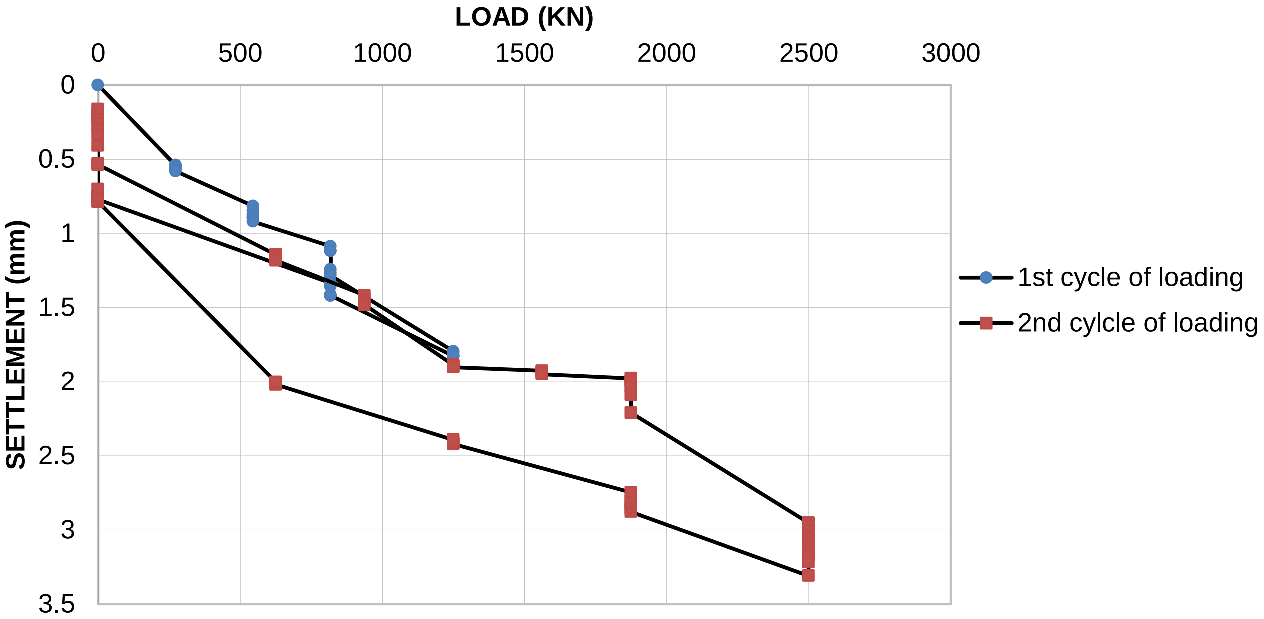

Figure 3.1 Load –settlement graph, K.K. test pile 1(from test reports by Anchor Foundation)

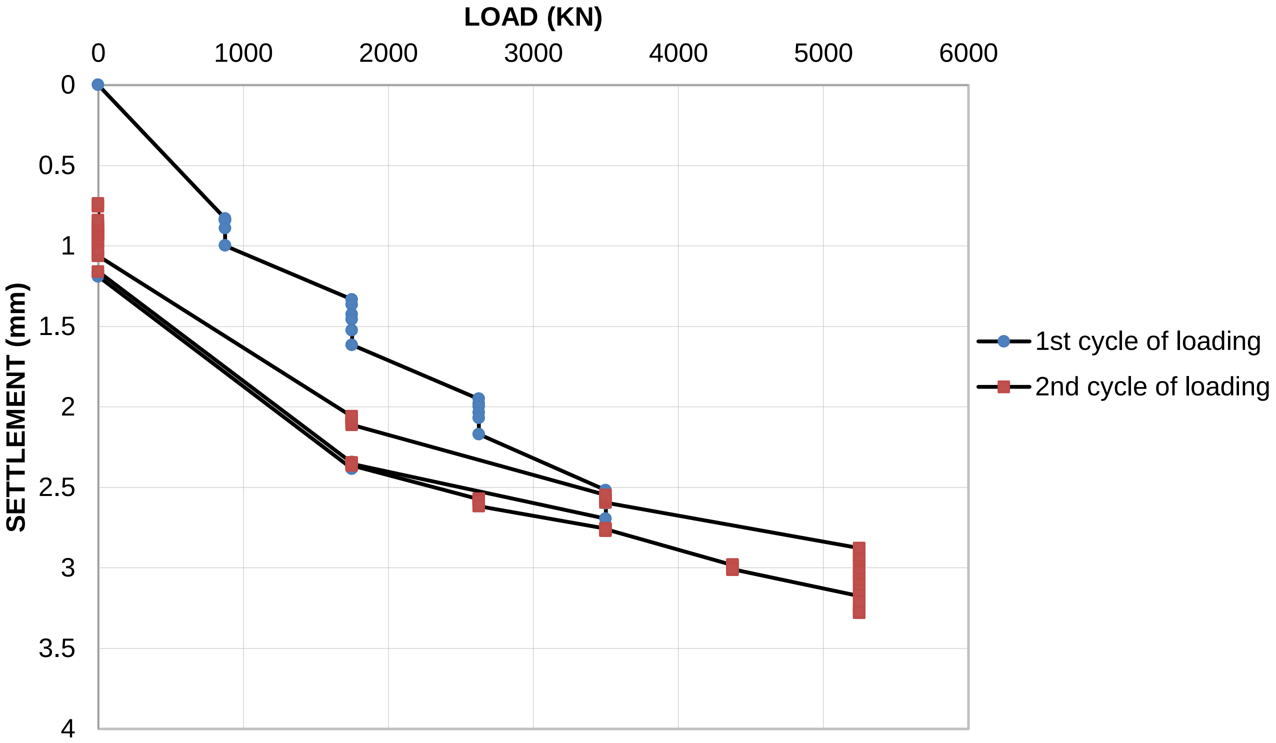

Figure 3.2 Load –settlement graph, K.K. test pile 2 (from test reports by Anchor Foundation)

3.3 Dire Dawa Taiwan Bridge Project

Dire Dawa, Taywan Bridge project is located in Dire dawa town around local area called Taywan. The project includes construction of a bridge crossing the sandy flood plain.

Addis Geosystem co.ltd. had carried out the soil investigation task to determine the engineering properties of soil and nature of the subsurface geological material.

Core drilling of three boreholes up to a depth of 20m, and along with in situ and laboratory tests has been conducted. Accordingly, the different geological layers have been identified and tabulated in Table 3.2

In all bore holes, ground water level is not encountered till the maximum boring depth.

Two pile loading test were performed by ANCOR FOUNDATION on working piles casted as part of the foundation system. Both piles are loaded up to 200% of the working load.

Pile No 1

The test pile has been drilled and cast with concrete on the 22/12/2013 with a diameter of 600mm to a depth of 16.49m from working platform level. The reinforcement of the pile consists of 14 vertical main bars of diameter 20mm. 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. Details of the drilling are recorded in the pile drilling and concreting record.

Pile no.2

The test pile has been drilled and cast with concrete on the 23/01/2014 with a diameter of 600mm to a depth of 21.53m from working platform level. The reinforcement of the pile consists of 14 nos. of vertical main bars diameter of 20mm. 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. Details of the drilling are recorded in the pile drilling and concreting record sheet.

The load vs settlement curve for the pile load test of Dire Dawa bridge project are shown in Fig. 3.3 and Fig. 3.4.

For further reference on loading test data and borehole logo sheets, refer appendix A and B respectively.

Table 3.2 Description of soil strata, Dire Dawa project (obtained from the soil investigation report by Addis Geosystems Co. Ltd.)

Layer	**Description**	**Layer Thickness**	**Bulk unit weight* (γ)**	**Friction angle ($\emptyset$)**	**Cohesion (c)**	**S.P.T. (N-values)**
1	loose, brown, non plastic silty sand	3.0 m	17.0 KN/M^3	35^O	5.0KPa	16
2	Medium dense brown gravely silty sand	2.0 m	17.0KN/M^3	30^O	6.0KPa	36
3	stiff reddish brown to yellowish silty clay	5.0m	17.0KN/M^3	34^O	8.0KPa	47
4	Dense, light brown, sandy silty clay	2.5 m	17.0KN/M^3	34^O	8.0KPa	50
5	Dense, light brown, fine to medium grained silty sand	3.5 m	17.0KN/M^3	33^o	6.0KPa	44
6	Very dense, fine to medium grained, gravely clayey sand	4.0 m	17.0 KN/M^3	35^o	5KPa	52

N.B.: Bulk unit weight of the soil is not given in the test result, hence the provided values are assumed.

Shear strength values shown in the above table are not directly taken for the numerical analysis, as the SPT values are not in good accordance with the respective shear strength values, correlation techniques based on SPT values are used for their determination. Detail method of calculation of shear strength parameters are shown in the next sub topics.

A generalized layer of three representing the overall stratification of the soil continuum has been considered for the analysis of this study,

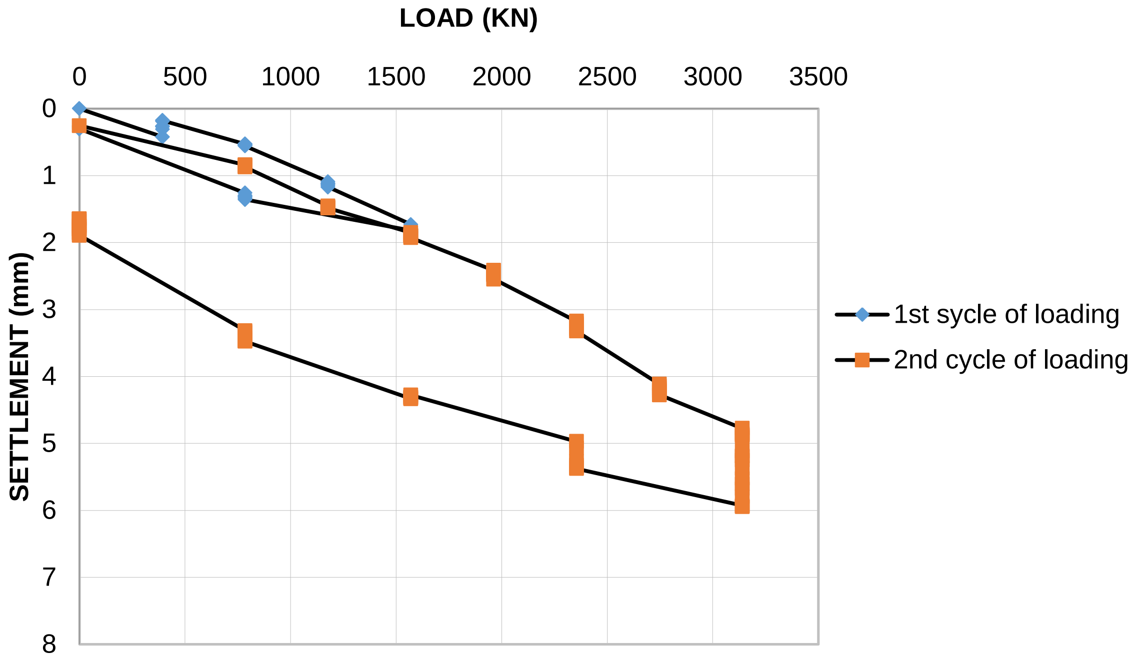

Figure 3.3 Load –settlement graph, D.D. test pile 1(from test reports by Anchor Foundation)

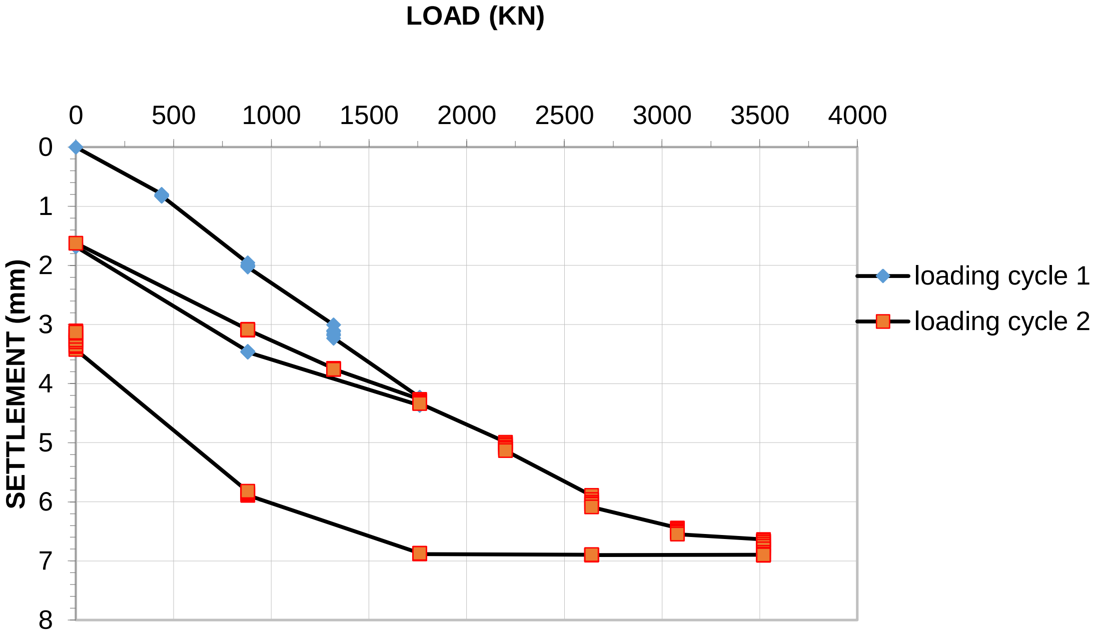

Figure 3.4 Load –settlement graph, D.D. test pile 2(from test reports by Anchor Foundation)

3.4 Gidabo Irrigation Dam Project

The Gidabo dam project site is located 377km away from Addis Ababa, in the Abaya-Chamo sub-basin of the Rift Valley, where, the lake basin is found in the southern part of the country within Oromiya and SNNPR States.

Water Works Design and Supervision Enterprise (WWDSE) had carried out the soil investigation task to determine the engineering properties of soil and nature of the subsurface geological material.

Two test types are performed in this project namely; design load tests and proof load tests on test piles and working piles respectively.

Core drilling of 41 bore holes were done up to 30m depth for the working and test piles. Out of these three pile load tests were performed, one on test pile and the other two on working piles. In this study one test pile and one working pile are considered.

Pile No. 1

The preliminary pile load test had been done on the date November 16, 2013, on pile with a diameter of 600mm to a depth of 28.8m from the working plat form level, at bore hole no. GITPBH-2. Details of concrete quality and reinforcement schedule of the test pile is not specified in geotechnical report.

Pile No. 2

A working pile load test had been done on two piles on bore hole GITPBH-1 & GITPBH-38. In this study only working pile on GITPBH-1 is considered.

The working pile load test was done on December 2013 G.C., on a pile with diameter 600mm with a length of 30m from working plat form level, at bore hole GITPBH-1. Here also details of concrete quality and reinforcement schedule of the working pile is not specified in geotechnical report.

For all the above piles, data of the different soil layers, an idealized soil profile has been generated to simplify the complex stratification of the soil layers with due consideration of the existing conditions. This helps reduce the complexity of the numerical model and the associated computational efforts.

The load vs settlement curves for the pile load test of Gidabo Dam Project are shown in Fig. 3.5 and Fig. 3.6.

For further reference on loading test data and borehole logo sheets, refer appendix A and B respectively.

Finally, summarized data of piles for all projects are shown in Table 3.3.

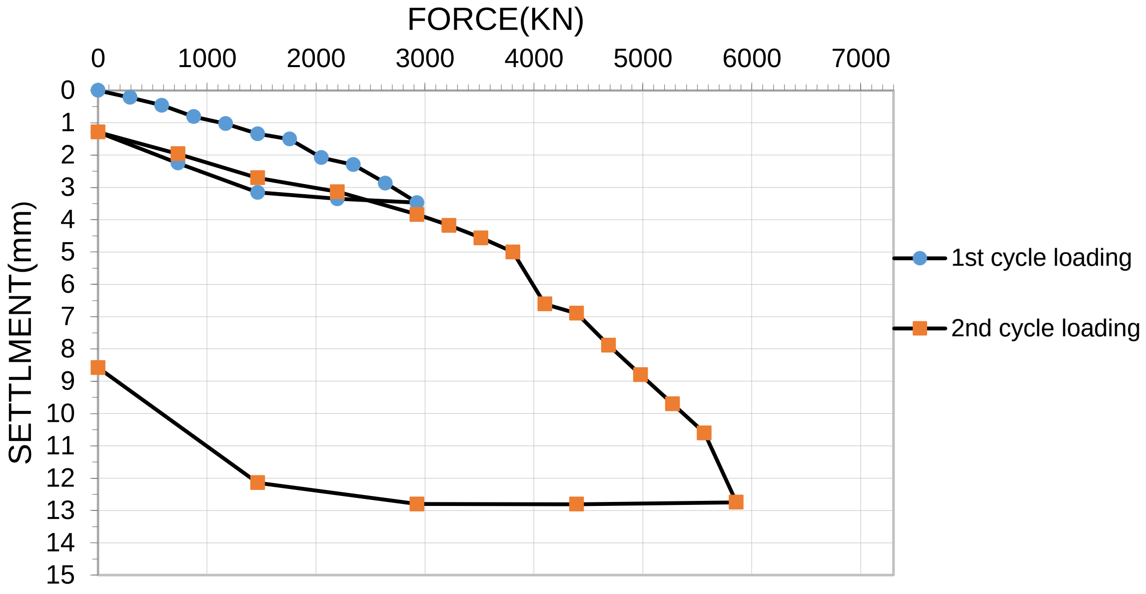

Figure 3.5 Load –settlement graph, Gidabo dam failure load test pile 1, on GITPBH 2(from test report by MIDROC Foundation)

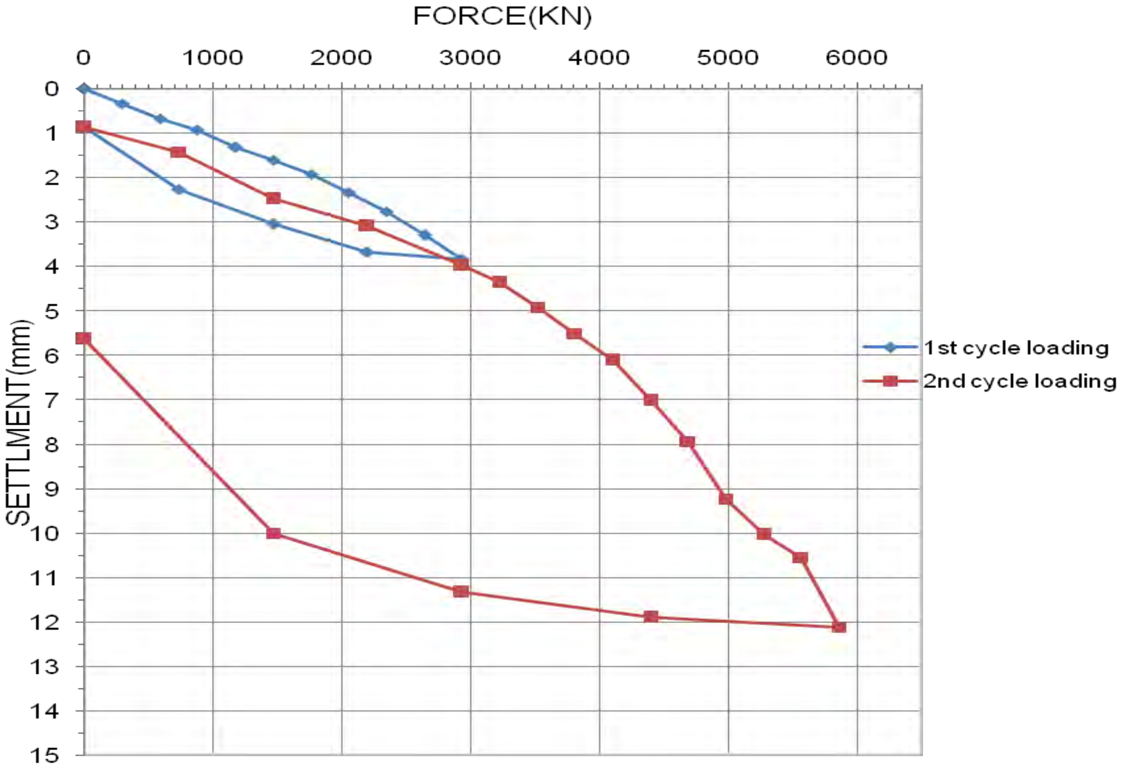

Figure 3.6 Load –settlement graph, Gidabo dam working pile test 2, on pile, @ GITPBH 1(from test report by MIDROC Foundation)

Table 3.3 Summary of pile data of all projects

Project Name	**Bore Hole No.**	**Pile ID**	**Pile Diameter**	**Pile length**	**Pile type**
Dire Dawa Bridge roject	BH-1	Pile 1	600 mm	16.49 m	Working pile
	BH-2	Pile 2	600 mm	21.53 m	Working pile
K.K. mixed use building project	BH-1	Pile 1	800 mm	16.30 m	Working pile
	BH-2	Pile 2	1000 mm	15.40 m	Working pile
Gidabo Dam Project	GITPBH 1	Pile 2	600 mm	28.80 m	Working pile
	GITPBH 2	Pile 1	600 mm	30.00 m	Test pile

3.5 Determination of Basic Soil Parameter for the Numerical Model

The soil investigation results, that have been obtained and are used in this study are tabulated and shown in the preceding sections. However, almost all the above data are not complete in terms of the requirement of soil parameters for the numerical analysis. Usually, only basic soil parameters are likely to be obtained from geotechnical reports.

Parameters necessary for the numerical analyses in this study have been determined in two ways; by taking directly from the field and laboratory test results and the remaining, perhaps most of the parameters which could not be obtained directly from the laboratory tests, have been indirectly derived by using empirical correlations based on the recommendations of different literature, international geotechnical codes and standards.

Atterberg Limits

i. K.K. Mixed use building project

Table 3.4 Atterberg limits and unit weight of silty clay and tuff layers

Item No.	BH No.	Depth (m)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Unit weight (kN/m^3)	Specific Gravity (Gs)
1	1	15.4	69.00	40.56	28.44	17	2.57
2	1	18.0	78.90	51.05	27.85	16.4	2.67
3	1	26.5	86.50	74.96	11.54	16.3	2.57

ii. Gidabo Dam Project

Aterberg limits tests are performed on some selected layers of GIBH-2 and the result is summarized in Table 3.5.

Table 3.5 Atterberg limits and unit weight

Description	BH No.	Depth (m)	Liquid limit (%)	Plasticity index(%)	Specific Gravity (Gs)	Bulk unit weight (KN/m^3)
Silty clay	GIBH-2	7.1-7.4	72.65	42.25	2.65	17.88

Stiffness Modulus (E_s)

The stiffness modulus E_s is a basic parameter that describes the load-settlement behavior of soils and governs the results of settlement related problems. The use of a practical and reasonable stiffness values representing the in situ conditions is of great importance in finite element analysis for better simulation of the actual condition of the soil.

Several methods are available for estimating the stiffness modulus of a soil as described by Bowles (1996). Unconfined compression tests, tri-axial compression tests and in situ tests are among the test methods. While unconfined compression tests tend to give conservative values, tri-axial tests tend to produce more usable values of E_s since any confining stress “stiffens” the soil so that a larger initial tangent modulus is obtained.

For K.K. project the geotechnical investigation report has included E values for the different layers based on experience data. However, no E_s values have been obtained from test results in all of the cases in this study. Therefore its determination totally lies based on the following methods:

i. Empirical formulas and range values provided by Bowles(1996)

For sand (nc)

$E_s = 500(N+15)$

$E_s = 7000(N)^{1/2}$

$E_s = 6000(N)$

For gravely Sand

$E_s = 600(N + 6), \text{ for } N < 15$

$= 600(N + 6) + 2000, \text{ for } N > 15$

$= 100 – 200$ Mpa

For clayey sand

$E_s = 320(N+15)$

For clayey silt

$E_s = 300(N+6)$

Where, $N= N_{55}$

In this study, only average range values provided for gravely sand soil is used for Gidabo sandy gravely soil.

ii. Using code provisions;
German code (DIN 4094-1 (2002) and DIN 4094-2 (2003)

$$\boldsymbol{E_s = \mu \times p_a \left(\frac{\sigma_z + 0.5\Delta\sigma_z}{p_a} \right)^w} \tag{3.1}$$

where:

μ = stiffness coefficient depending on values of N-SPT

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

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

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

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

Values of E_s obtained by DIN methods for sandy and clayey layers are tabulated as shown in tables below.

Once E_s is determined using the method of DIN, E from E_s has been found using the formula based on EVB (1996). It is used as input for the analysis using ABAQUS software.

$$\boldsymbol{E} = \boldsymbol{E}_s\left(\frac{1-\boldsymbol{\nu}-2\boldsymbol{\nu}}{1-\boldsymbol{\nu}}\right) \tag{3.2}$$

Where, ν = poison's ratio

Based on Eq. 3.2, values of stiffness modulus of different soil layers for all projects are computed and summarized as shown in Tables 3.6 to 3.9.

Table 3.6 Stiffness modulus value for K.K project

Depth (m)	Description	N valuees	μ	σ_z kN/m^2	$\Delta\sigma z$ kN/m^2	Es, (kN/m^2) (using DIN)	E (kN/m^2)
9	Highly weathered decomposed basalt	>50	450	76.5	83.00	-	50,000.0*
16	Slightly weathered strong basalt	>50	450	289.0	10.75	-	100,000.0*
19	Silty clay	49	450	427.8	5.49	101,094.20	75,098.5
26	Slightly weathered strong basalt	>50	450	529.2	3.32	-	100,000.0*
33	Decomposed tuff	50	450	713.7	1.93	154,882.6	115,055.7
40.3	scoria	50	450	892.2	1.26	132,321.3	98,295.8

* AS method of DIN is applicable for clayey and sandy soils, for the successive basaltic layers, provided value of E_s in the geotechnical report is taken for the analysis.

Table 3.7 Summary of stiffness modulus value for Dire Dawa Tayiwan Bridge project

Depth (m)	Description	N valuees	μ	σ_z kN/m^2	$\Delta\sigma z$ kN/m^2	E_s (kN/m^2) (using DIN)	E (kN/m^2)
3.0	silty sand	16	450	25.5	666.33	85817.45	63750.10
5.0	Medium dense silty sand	36	450	68.00	93.70	45562.31	36074.86
10.0	Stiff silty sand	47	450	127.50	26.65	53774.18	39946.53
12.5	Dense sandy silty clay	50	517	191.20	11.85	78078.64	58001.51
16.0	Dense silty sand	44	450	242.3	7.38	71063.84	52790.13
20.0	Very dense clayey sand	52	450	306.00	4.63	79566.06	59106.22

Table 3.8 Summary of stiffness modulus value for Gidabo Dam Project, TP-1 (GIBH-2)

Depth (m)	Description	N values	μ	σ_z kN/m^2	$\Delta\sigma z$ kN/m^2	Es (kN/m^2) (using DIN)	E (kN/m^2)
0-9	Silty clay	12	162	80.1	138.12	19923.35	14800.0
9-29	Sandy gravely material	14	450	320.2	7.75	81574.34	150,000.0*

Table 3.9 Summary of stiffness modulus value for Gidabo Dam Project, WP-2 (GIBH-1)

Depth (m)	Description	N valuees	μ	σ_z kN/m^2	$\Delta\sigma z$ kN/m^2	Es (kN/m^2) (using DIN)	E (kN/m^2)
9	Silty clay	14	178	80.10	138.12	22,752.50	16,901.8
30	Sandy gravely material	49	450	328.20	7.36	92,942.56	150,000.0*
33	sand	45	450	521.70	2.82	122,119.00	90,716.9
38	Silty clay ,sandy gravel	52	450	606.70	2.10	133,671.30	99,258.5

*For sandy gravel material typical average value provided on Bowls (1996) has been taken

Details of the calculation method and determination of the coefficients can be referred from DIN 4094-1 (2002) and DIN 4094-2 (2003). The stress dependent stiffness modulus E_s has been determined using Eq. 3.1, at the midpoint of each soil layer. The values of soil stiffness modulus computed using Eq. (3.1) designated as E_s, are tabulated in Tables 3.6-3.9.

Shear Strength Parameters

Shear strength parameters, namely, angle of internal friction (∅') and cohesion (c') are determined mostly from the direct shear test result that are taken from samples obtained from shallow depth. But for deeper layers, where laboratory test results are not available, correlation method provided by Bowles (1996), taken from Shioi and Fukui (1982) Japanese standard, has been used to determining angle of internal friction.

$$\boldsymbol{\phi} = \sqrt{18\boldsymbol{N}_{70}} + 15 \tag{3.3}$$

For sandy and granular soil layers value of internal friction angle are determined using Eq. 3.3 and results are summarized in Tables 3.10 to 3.12. For cohesive soil layers values of internal friction angle (*) and cohesion (*) are directly taken from the respective soil investigation reports.

Table 3.10 Summary of shear parameters of K.K. project

Depth (m)	Description	SPT 'N' value	Angle of internal friction (∅')	Cohesion (c), KPa
9	Highly weathered decomposed basalt	49	44.7	-
16	Slightly weathered strong basalt	50	45.0	-
19	Silty clay	-	27*	19*
26	Slightly weathered strong basalt	50	45.0	-
33	Decomposed tuff	-	27*	16*
40.3	scoria	50	45.0	-

Table 3.11 Summary of shear parameters for Dire Dawa project

Depth (m)	Description	SPT ‘N’ value	Angle of internal friction ($\varnothing$’)	Cohesion (c) KPa
3.0	Loose silty sand	16	32.0	-
5.0	Medium dense silty sand	36	40.5	-
10.0	Stiff silty sand	47	44.1	-
12.5	Dense sandy silty clay	-	34*	8*
16.0	Dense silty sand	44	43.1	-
20.0	Very dense clayey sand	52	45.6	-

Table 3.12 Summary of shear parameters for Gidabo Dam project

Depth (m)	Description	SPT ‘N’ value	Angle of internal friction ($\varnothing$’)	Cohesion (c), KPa
9	Silty clay	-	32*	23*
30	Sandy gravely soil	49	44.7	-
33	sand	45	43.5	-
38	Silty clay ,sandy gravel	52	45.6	-

Cohesion values for the deeper soil layers are not determined in the lab, hence the values is omitted in the analysis as most of the layers are sandy gravel soil.

Initial stress coefficient, k_0

The initial stress condition of the soil mass is the ratio of the horizontal stresses to the vertical stresses which is called coefficient of earth pressure at rest, k_0. Empirical correlations have been used to determine the value of $k_{0.}$

For normally consolidated clays, and for granular soils, Holtz and Kovacs (1981) and Jaky(1944) empirical correlation formulas have been taken respectively as follows:

$$K_o = 0.44 + 0.0042 \text{ x } I_p \qquad (3.4)$$

$$K_o = 1 – \text{Sin}(\varnothing) \qquad (3.5)$$

Where:

K_o = coefficient of earth pressure at rest

I_p = plasticity index

Based on Eq. 3.4 and 3.5, values of initial stress coefficient are summarized in Tables 3.13 to 3.15.

Table 3.13 Initial stress coefficient for K.K. project

Depth (m)	Description	Initial stress coefficient (K_o)
9	Highly weathered decomposed basalt	0.50
16	Slightly weathered strong basalt	0.43
19	Silty clay	0.56
26	Slightly weathered strong basalt	0.38
33	Decomposed tuff	0.49
40.3	scoria	0.54

Table 3.14 Initial stress coefficient for Dire Dawa project

Depth (m)	Description	Initial stress coefficient (K_o)
3.0	Loose silty sand	0.43
5.0	Medium dense silty sand	0.36
10.0	Stiff silty sand	0.44
12.5	Dense sandy silty clay	0.44
16.0	Dense silty sand	0.46
20.0	Very dense clayey sand	0.43

Table 3.15 Initial stress coefficient for Gidabo

Depth (m)	Description	Initial stress coefficient, K_o
9	Silty clay	0.62
30	Sandy gravel	0.29
33	Sand	0.32
38	Silty clay ,sandy gravel	0.50

Poisson's Ratio, ν

Poisson's ratio is a property that describes the volume change of a material in a direction perpendicular to the application of a load. It is defined as the ratio of the axial compression to the lateral expansion of soils. Bowels (1996) recommend a range of values of Poison's ratio between 0.4 and 0.5 for most clay and 0.2 to 0.4 for medium to dense cohesion less soil. Hence typical values of 0.4 and 0.3 have been taken for the numerical analysis of this study respectively.

Consolidation Settlement Parameters

Compression indices for K.K. project have been determined in lab for some compressible soil layers in the strata. However, in most cases no such values can be obtained from laboratory result, hence, its determination lies on relating compression indices to the simple classification properties of soils.

Terzaghi and Peck (1967) proposed the following equation:

$$\boldsymbol{C_c} = 0.009(\boldsymbol{LL} - 10) \tag{3.6}$$

Azzouz et. Al. (1976)

$$\boldsymbol{C_c} = 0.01(\boldsymbol{W} - 5) \tag{3.7}$$

$$\boldsymbol{C_s} = 0.003(\boldsymbol{W} + 7) \tag{3.8}$$

Nagaraj and Murthy (1985) presented correlation between Cs, Gs, LL

$$\boldsymbol{C_s} = 0.000463 \times \boldsymbol{LL} \times \boldsymbol{G_s} \tag{3.9}$$

Where:

LL = liquid limit in percent

W = natural moisture content in percent

Budhu (2011), highlighted typical ranges of values for Cc and Cs. Cc ranges from 0.1 to 0.8 and Cs ranges from 0.015 to 0.035. Values from these ranges are used in the analysis when there is no data available in the soil test result.

λ *and* κ which are slopes of the normal consolidation and unloading–reloading lines in the *e*–ln *p* plane are related to the compression index *Cc* and swelling index *Cs* obtained from the above correlation:

$$\lambda = \frac{C_C}{\ln 10} = \frac{C_C}{2.3}$$

$$\kappa = \frac{C_S}{\ln 10} = \frac{C_S}{2.3} \tag{3.10}$$

Based on the above techniques, values of C_c,C_s, λ and κ are tabulated in Tables 3.16 to 3.18 for all piles:

Table 3.16 Consolidation parameters for K.K project

Depth (m)	**Description**	**Compression index (C_c)**	**Swelling index (C_s)**	**λ**	**κ**
0 - 9	Highly weathered decomposed basalt	0.100	0.015	0.043	0.007
9 - 16	Slightly weathered strong basalt	0.100	0.015	0.043	0.007
16 - 19	Silty clay	0.310*	0.044*	0.135	0.019
19 - 26	Slightly weathered strong basalt	0.100	0.015	0.043	0.007
26 - 33	Decomposed tuff	0.689	0.035	0.300	0.015
33- 40.3	scoria	0.689	0.035	0.300	0.015

*Cc & *Cs values are taken from test result, the rest from correlations and typical range value specified above.

For slightly and highly weathered soil layers in K.K. project, lower range values provided by Budhu (2011) are taken, as consolidation in this layer is expected to be very small.

Table 3.17 Consolidation parameters for Dire Dawa Bridge Project

Depth (m)	Description	Compression index (C_c)	Swelling index (C_s)	λ	κ
0.0-3.0	Loose silty sand	0.100	0.041	0.043	0.018
3.0-5.0	Medium dense silty sand	0.100	0.041	0.043	0.018
5.0-10.0	Stiff silty sand	0.100	0.047	0.043	0.020
10.0-12.5	Dense sandy silty clay	0.100	0.047	0.043	0.020
12.5-16.0	Dense silty sand	0.100	0.059	0.043	0.025
16.0-20.0	Very dense clayey sand	0.110	0.070	0.049	0.030

N.B.: calculated Cc values are taken as the lower value of recommended ranges

Table 3.18 Consolidation parameters for Gidabo Dam Project

Depth (m)	Description	Compression index (C_c)	Swelling index (C_s)	λ	κ
9	Silty clay	0.564	0.035*	0.245	0.015
30	Sandy gravely material	0.399	0.035*	0.173	0.015
33	sand	0.325	0.035*	0.141	0.015
38	Silty clay ,sandy gravel	0.325	0.035*	0.141	0.015

*calculated Cs values based on correlations are out of the ranges recommended by Budhu(2011), hence higher value of the range is taken for this study.

4 THE PROPOSED CONSTITUTIVE MODELS AND THEORIES

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4.1 Stress Matrix

The stress state at a point *A* within a soil mass can be represented by an infinitesimal cube with three stress components on each of its six sides (one normal and two shear components), as shown in Figure 4.1. Since point *A* is under static equilibrium (assuming the absence of body forces such as the self-weight), only nine stress components from three planes are needed to describe the stress state at point *A*. These nine stress components can be organized into the stress matrix:

$$\begin{pmatrix} \sigma_{11} & \tau_{12} & \tau_{13} \\ \tau_{21} & \sigma_{22} & \tau_{23} \\ \tau_{31} & \tau_{32} & \sigma_{22} \end{pmatrix} \tag{4.1}$$

Where σ_{11}, σ_{22}, and σ_{33} are the normal stresses (located on the diagonal of the stress matrix) and τ_{12}, τ_{21}, τ_{13}, τ_{31}, τ_{23}, and τ_{32} are the shear stresses. The shear stresses across the diagonal are identical (i.e., $\tau_{12} = \tau_{21}$, $\tau_{13} = \tau_{31}$ and $\tau_{23} = \tau_{32}$) as a result of static equilibrium (to satisfy moment equilibrium). This arrangement of the nine stress components is also known as the stress tensor *(Sam Helwayn 2007)*.

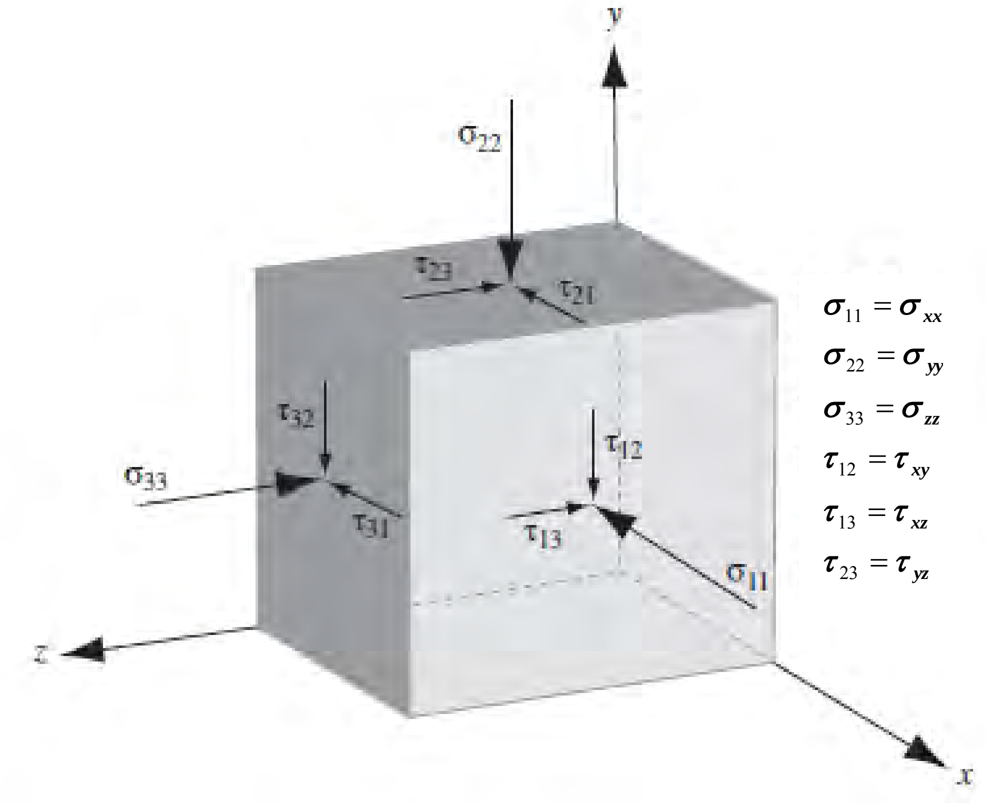
The subscripts 1, 2, and 3 are used here instead of the subscripts *x*, *y*, and *z*, respectively (see Figure 4.1).

The subscripts used for the nine stress components $\sigma_{\alpha\beta}$ and $\tau_{\alpha\beta}$ have the following meaning: α is the direction of the surface normal upon which the stress acts, and β is the direction of the stress component.

4.1.1 Elasticity

The generalized Hooke’s law will be applied to the uniaxial stress condition (one dimensional), and the plane stress condition (two-dimensional).

Sam Helwany 2007, had pointed out that Hooke’s law is not appropriate for soils because soils are neither linear elastic nor isotropic. Nevertheless, soil is idealized as being linear elastic and isotropic materials—only then one can use Hooke’s law to estimate the elastic strains associated with applied stresses within a soil mass.



$$\sigma_{11} = \sigma_{xx}$$
$$\sigma_{22} = \sigma_{yy}$$
$$\sigma_{33} = \sigma_{zz}$$
$$\tau_{12} = \tau_{xy}$$
$$\tau_{13} = \tau_{xz}$$
$$\tau_{23} = \tau_{yz}$$

Figure 4.1 Stresses in three-dimensional space (Sam Helwayn, 2007).

The simplest form of linear elasticity is the isotropic case, which is the elastic moduli, such as E and ν, are orientation independent. This means, for example, that E_{11}, E_{22}, and E_{33} are identical and they are all equal to E (Young's modulus). The stress–strain relationship of the linear elastic isotropic case is given by

$$\begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \tau_{12} \\ \tau_{13} \\ \tau_{23} \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ \nu & 1-\nu & \nu & 0 & 0 & 0 \\ \nu & \nu & 1-\nu & 0 & 0 & 0 \\ 0 & 0 & 0 & 1-2\nu & 0 & 0 \\ 0 & 0 & 0 & 0 & 1-2\nu & 0 \\ 0 & 0 & 0 & 0 & 0 & 1-2\nu \end{bmatrix} \times \begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{13} \\ \varepsilon_{23} \end{Bmatrix} \quad (4.2)$$

The elastic properties are defined completely by Young's modulus, E, and Poisson's ratio, ν. Equation (4.2) is also known as the *generalized Hooke's law*. Hooke's law for the one-dimensional (uniaxial) stress condition is $\sigma = E\varepsilon$. This equation has the same general form as (4.2). It will be shown below that (4.2) reduces to $\sigma = E\varepsilon$ for the uniaxial stress condition. Equation (4.2) can be inverted to yield:

$$\begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{13} \\ \varepsilon_{23} \end{Bmatrix} = \begin{bmatrix} 1/E & -\nu/E & -\nu/E & 0 & 0 & 0 \\ -\nu/E & 1/E & -\nu/E & 0 & 0 & 0 \\ -\nu/E & -\nu/E & 1/E & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/2G & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/2G & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/2G \end{bmatrix} \times \begin{Bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{33} \\ \tau_{12} \\ \tau_{13} \\ \tau_{23} \end{Bmatrix} \tag{4.3}$$

In this equation, the shear modulus, G, can be expressed in terms of E and ν as

$$G = \frac{E}{2(1+\nu)} \tag{4.4}$$

4.1.1.1 Uniaxial Stress Condition

In a uniaxial stress condition we have $\sigma_{22} = \sigma_{33} = \tau_{12} = \tau_{13} = \tau_{23} = 0$, and $\sigma_{11} \neq 0$. Substituting into (4.3), we get

$$\begin{Bmatrix} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{33} \\ \varepsilon_{12} \\ \varepsilon_{13} \\ \varepsilon_{23} \end{Bmatrix} = \begin{bmatrix} 1/E & -\nu/E & -\nu/E & 0 & 0 & 0 \\ -\nu/E & 1/E & -\nu/E & 0 & 0 & 0 \\ -\nu/E & -\nu/E & 1/E & 0 & 0 & 0 \\ 0 & 0 & 0 & 1/2G & 0 & 0 \\ 0 & 0 & 0 & 0 & 1/2G & 0 \\ 0 & 0 & 0 & 0 & 0 & 1/2G \end{bmatrix} \times \begin{Bmatrix} \sigma_{11} \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix} \tag{4.5}$$

4.1.2 Plasticity

When an elastic material is subjected to load, it sustains elastic strains. Elastic strains are reversible in the sense that the elastic material will spring back to its un-deformed condition if the load is removed. On the other hand, if a plastic material is subjected to a load, it sustains elastic and plastic strains. If the load is removed, the material will sustain permanent plastic (irreversible) strains, whereas the elastic strains are recovered. Hooke's law, which is based on elasticity theory, is sufficient (in most cases) to estimate the elastic strains. To estimate the plastic strains, one needs to use plasticity theory.

Plasticity theory was originally developed to predict the behavior of metals subjected to loads exceeding their elastic limits. Similar models were developed later to calculate the irreversible strains in concrete, soils, and polymers.(Sam Helwany 2007)

A plasticity model includes (1) a yield criterion that predicts whether the material should respond elastically or plastically due to a loading increment, (2) a strain hardening rule that controls the shape of the stress–strain response during plastic straining, and (3) a plastic flow rule that determines the direction of the plastic strain increment caused by a stress increment.

In this study only two plasticity models (Mohr-Coulomb model and Cap model) that are frequently used in geotechnical engineering applications will be reviewed and used in the simulation of the pile loading test with finite element method.

4.1.2.1 Mohr Coulomb Model

The Mohr-Coulomb failure or strength criterion has been widely used for geotechnical applications. Indeed, a large number of the routine design calculations in the geotechnical area are still performed using the Mohr- Coulomb criterion.

The Mohr-Coulomb criterion assumes that failure is controlled by the maximum shear stress and that this failure shear stress depends on the normal stress, (Abacus software manual). This can be represented by plotting Mohr's circle for states of stress at failure in terms of the maximum and minimum principal stresses. The Mohr-Coulomb failure line is the best straight line that touches these Mohr's circles (Figure 4.2). Thus the Mohr-Coulomb criterion can be written as:

$$\tau = c - \sigma \tan\phi \tag{4.6}$$

Where τ is the shear stress, σ is the normal stress (negative in compression), c is the cohesion of the material, and $\varnothing$ is the material angle of friction.

$$\begin{aligned} \tau &= s\cos\phi \\ \sigma &= \sigma_m + s\sin\phi \end{aligned} \tag{4.7}$$

Substituting for τ and σ , the Mohr-Coulomb criterion can be rewritten as

$$s + \sigma_m \sin\phi - c\cos\phi = 0 \tag{4.8}$$

Where

$$s = \frac{1}{2}(\sigma_1 - \sigma_3) \tag{4.9}$$

is half of the difference between the maximum and minimum principal stresses (and is, therefore, the maximum shear stress) and

$$\sigma_m = \frac{1}{2}(\sigma_1 + \sigma_3) \quad (4.10)$$

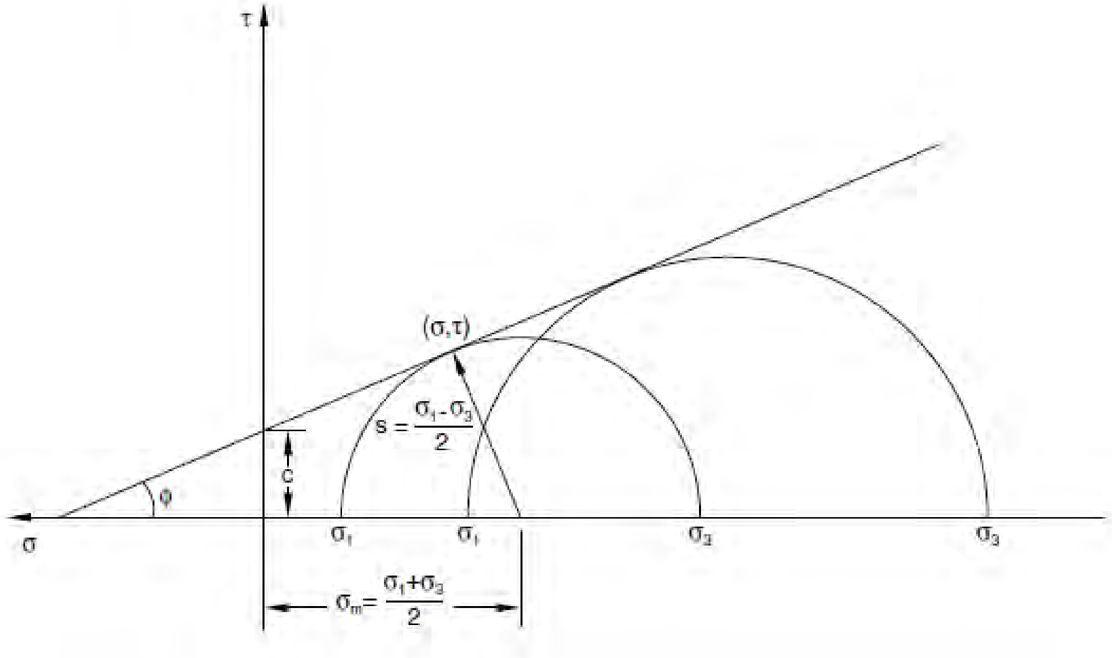


Figure 4.2 Mohr-Coulomb failure criterion.

is the average of the maximum and minimum principal stresses (the normal stress). Thus, the Mohr-Coulomb criterion assumes that failure is independent of the value of the intermediate principal stress. The failure of typical geotechnical materials generally includes some small dependence on the intermediate principal stress, but the Mohr-Coulomb model is generally considered to be sufficiently accurate for most applications. This failure model has vertices in the deviatoric stress plane.

The constitutive model described here is an extension of the classical Mohr-Coulomb failure criterion. It is an elasto-plastic model that uses a yield function of the Mohr- Coulomb form; this yield function includes isotropic cohesion hardening/softening. However, the model uses a flow potential that has a hyperbolic shape in the meridian stress plane and has no corner in the deviatoric stress space. This flow potential is then completely smooth and, therefore, provides a unique definition of the direction of plastic flow.

4.1.2.2 Modified Drucker-Prager/Cap Model

The Drucker–Prager/cap plasticity model has been widely used in finite element analysis programs for a variety of geotechnical engineering applications. The cap model is appropriate to soil behavior because it is capable of considering the effect of stress history, stress path, dilatancy, and the effect of the intermediate principal stress. The yield surface of the modified Drucker–Prager/cap plasticity model consists of three parts: a Drucker–Prager shear failure surface, an elliptical *cap*, which intersects the mean effective stress axis at right angle, and a smooth transition region between the shear failure surface and the cap, as shown in Figure 4.3

The cap serves two main purposes: it bounds the yield surface in hydrostatic compression, thus providing an inelastic hardening mechanism to represent plastic compaction, and it helps to control volume dilatancy when the material yields in shear by providing softening as a function of the inelastic volume increase created as the material yields on the Drucker-Prager shear failure and transition yield surfaces.(Abacus tutorial 2002)

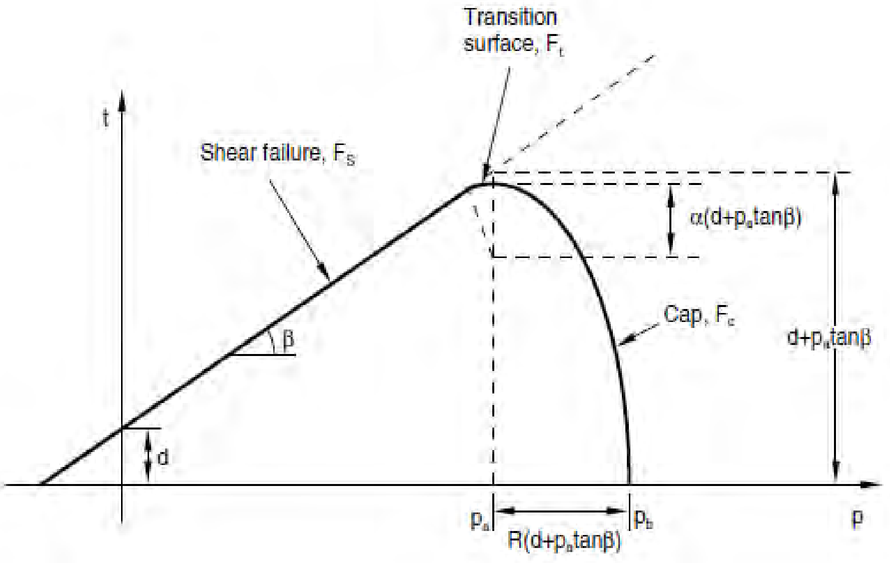


Figure 4.3 Modified Drucker-Prager/Cap model: yield surfaces in the p–t plane. (Adapted from ABAQUS 2002)

The model uses associated flow in the cap region and non-associated flow in the shear failure and transition regions.

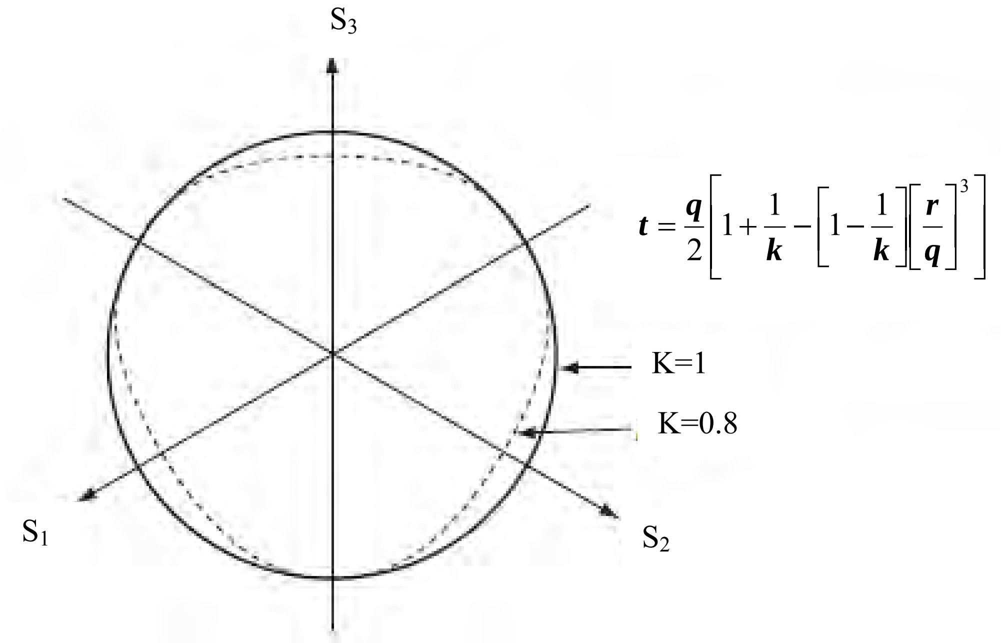


Figure 4.4 Projection of the modified cap yield surface on the π-plane`

Elastic behavior is modeled as linear elastic using the generalized Hooke's law. Alternatively, an elasticity model in which the bulk elastic stiffness increases as the material undergoes compression can be used to calculate the elastic strains. The onset of plastic behavior is determined by the Drucker–Prager failure surface and the cap yield surface. The Drucker–Prager failure surface is given by Eq. 4.11.

$$\boldsymbol{F_s}=\boldsymbol{t}-\boldsymbol{p}\tan\boldsymbol{\beta}-\boldsymbol{d}=0 \tag{4.11}$$

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

$$\boldsymbol{t}=\frac{\boldsymbol{q}}{2}\left[1+\frac{1}{\boldsymbol{K}}-\left[1-\frac{1}{\boldsymbol{K}}\right]\left[\frac{\boldsymbol{r}}{\boldsymbol{q}}\right]^3\right] \tag{4.12}$$

$$\boldsymbol{q}=\sqrt{\frac{1}{2}\left((\boldsymbol{\sigma}_1-\boldsymbol{\sigma}_2)^2+(\boldsymbol{\sigma}_2-\boldsymbol{\sigma}_3)^2+(\boldsymbol{\sigma}_3-\boldsymbol{\sigma}_1)^2\right)} \tag{4.13}$$

$$\boldsymbol{p}=\frac{1}{3}(\boldsymbol{\sigma}_1+\boldsymbol{\sigma}_2+\boldsymbol{\sigma}_3) \tag{4.14}$$

q = deviatoric stress (Misses equivalent stress)
d = cohesion in the p-t-plane
p = hydrostatic stress
β = slope of the yield surface FS in the p-t-plane
K = shape parameter of the yield surface FS determined from tri-axial compression and extension tests with usual values, 0.778 < K < 1

The cap yield surface is an ellipse with eccentricity = *R* in the *p–t* plane. The cap yield surface is dependent on the third stress invariant, *r*, in the deviatoric plane as shown in Figure 4.4. The cap surface hardens (expands) or softens (shrinks) as a function of the volumetric plastic strain. When the stress state causes yielding on the cap, volumetric plastic strain (compaction) results, causing the cap to expand (hardening). But when the stress state causes yielding on the Drucker–Prager shear failure surface, volumetric plastic dilation results, causing the cap to shrink (softening). The cap yield surface is given as shown on Eq. 4.15.

$$\boldsymbol{F_c} = \sqrt{(\boldsymbol{p}-\boldsymbol{p_a})^2+\left(\frac{\boldsymbol{Rt}}{1+\boldsymbol{\alpha}-\boldsymbol{\alpha}/\mathbf{cos}\boldsymbol{\beta}}\right)^2}-\boldsymbol{R}(\boldsymbol{d}+\boldsymbol{p_a}\,\mathbf{tan}\boldsymbol{\beta})=0 \qquad (4.15)$$

Where *R* is a material parameter that controls the shape of the cap and α is a small number (typically, 0.01 to 0.05) used to define a smooth transition surface between the Drucker–Prager shear failure surface and the cap defined by Eq. 4.16.

$$\boldsymbol{F_t} = \sqrt{(\boldsymbol{p}-\boldsymbol{p_a})^2+\left(\boldsymbol{t}-\left(1-\frac{\boldsymbol{\alpha}}{\mathbf{cos}\boldsymbol{\beta}}\right)(\boldsymbol{d}+\boldsymbol{p_a}\,\mathbf{tan}\boldsymbol{\beta})\right)^2}-\boldsymbol{\alpha}(\boldsymbol{d}+\boldsymbol{p_a}\,\mathbf{tan}\boldsymbol{\beta})=0 \qquad (4.16)$$

p_a is an evolution parameter that controls the hardening–softening behavior as a function of the volumetric plastic strain. The hardening–softening behavior is simply described by a piecewise linear function relating the mean effective (yield) stress *p_b* and the volumetric plastic strain as shown in Figure 4.5. This function can easily be obtained from the results of one isotropic consolidation test with several unloading–reloading cycles. However, in this study since no tri-axial tests are performed, methods specified by Sam Helwany 2007, are used for the computation, eq. 4.17

$$\varepsilon_v^p = \frac{\lambda - \kappa}{1 + e_o} \ln \frac{p'}{p'_o} = \frac{C_c - C_s}{2.3(1 + e_o)} \ln \frac{p'}{p'_o} \tag{4.17}$$

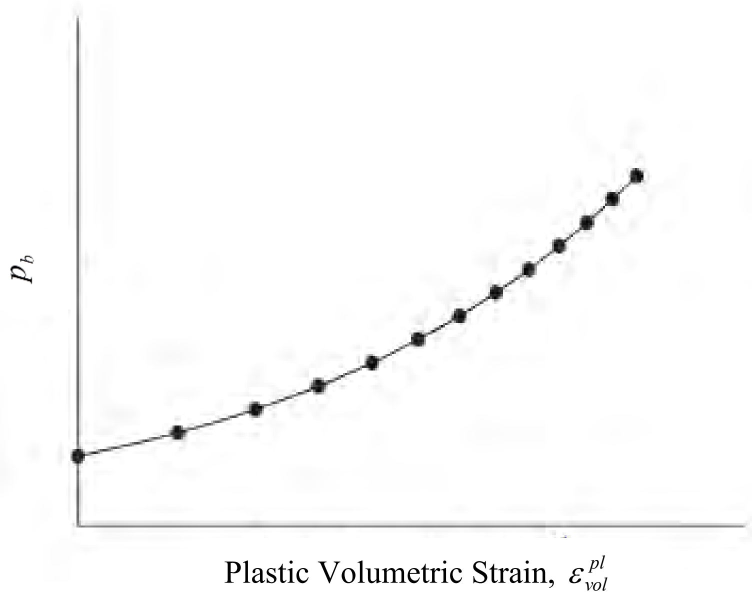


Figure 4.5 Typical cap hardening behavior (Adapted from ABACUS 2002)

The initial cap position is one of the input parameters for the cap model and can be obtained from the p_b vs $\varepsilon^{pl}{}_{vol}$ plots with the value of p used as input parameter.
The cap hardening behavior of the different soil layers in all cases of piles are summarized in Tables 4.1 to 4.3 based on Eq.4.17.

Table 4.1 Cap hardening behavior for KK soil layers

Layer 1 (highly weathered basalt)		Layer 2 (strong basalt)		Layer 3 (silty clay)		Layer 4 (strong basalt)		Layer 5 (tuff)		Layer 6 (scoria)	
ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)
0.000	76.5	0.000	289	0.000	427	0.000	529	0.000	713	0.000	893
0.010	140	0.003	350	0.009	500	0.002	600	0.015	800	0.008	950
0.019	230	0.010	540	0.032	770	0.007	800	0.046	1000	0.015	1000
0.026	350	0.016	770	0.049	1040	0.011	1040	0.071	1200	0.041	1200
0.033	540	0.022	1040	0.063	1350	0.016	1350		-	-	-
0.039	770	0.026	1350	0.076	1690	-	-	-	-	-	-
0.044	1040	0.030	1690	0.084	2000	-	-	-	-	-	-
0.056	2000	0.032	2000	0.12305	4000	-	-	-	-	-	-
0.062	3000	0.045	4000	-	-	-	-	-	-	-	-
0.067	4000	-	-	-	-	-	-	-	-	-	-

Table 4.2 Cap hardening behavior for Dire Dawa Taywan Bridge soil layers

Layer 1 (loose silty sand)		Layer 2 (medium dense silty sand)		Layer 3 (stiff silty sand)		Layer 4 (dense sandy silty clay)		Layer 5 (dense silty sand)		Layer 6 (dense clayey sand)	
ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)
0.000	25.5	0.000	68	0.000	127.5	0.000	191.2	0.000	242.2	0.000	306
0.016	70.0	0.026	350	0.015	350	0.009	350.0	0.014	800.0	0.014	950
0.027	140.0	0.033	540	0.026	750	0.018	700.0	0.016	1000.0	0.014	1000
0.041	350.0	0.038	770	0.030	1000	0.024	1000.0	0.018	1200.0	0.016	1200
0.052	700.0	0.042	1000	-	-	-	-	-	-	-	-
0.063	1400.0	-	-	-	-	-	-	-	-	-	-
0.068	2000.0	-	-	-	-	-	-	-	-	-	-

Table 4.3 Cap hardening behavior for Gidabo dam soil layers

Layer 1 (silty clay)		Layer 2 (sandy gravel)	
ε^{pl}_{vo}	P(kPa)	ε^{pl}_{vo}	P(kPa)
0.000	80.1	0.000	568.2
0.085	160.2	0.039	976.2
0.211	450.0	0.054	1200.0
0.272	750.0	0.070	1500.0
0.308	1000.0	0.090	2000.0
0.349	1400.0		
0.392	2000.0		
0.440	3000.0		

Cap hardening curves of two representative soil layers from each project are shown in Figures 4.6 to 4.11 as mean effective stress vs volumetric plastic strain graph.
Detail method of calculation and graph of all layers is attached in Appendix C.

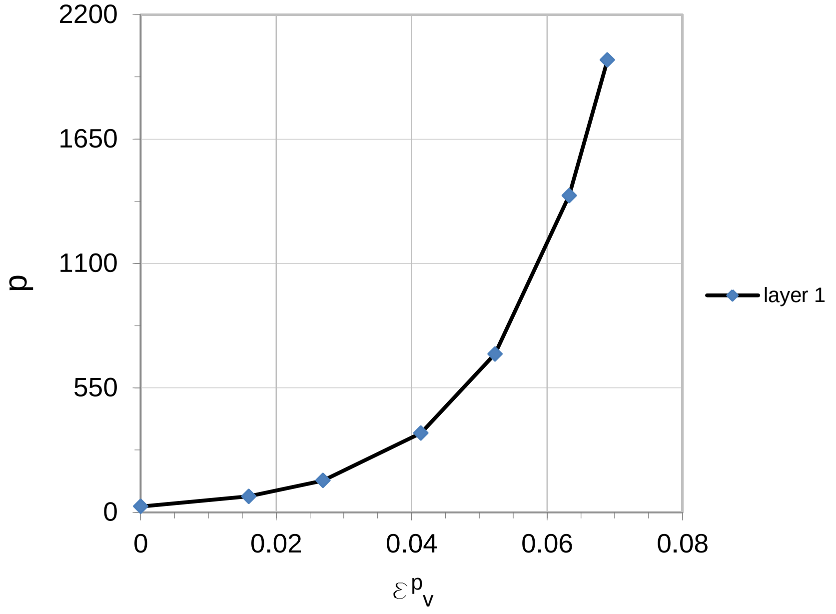


Figure 4.6 Cap hardening curve for top silty sand layer of dire dawa project soil,

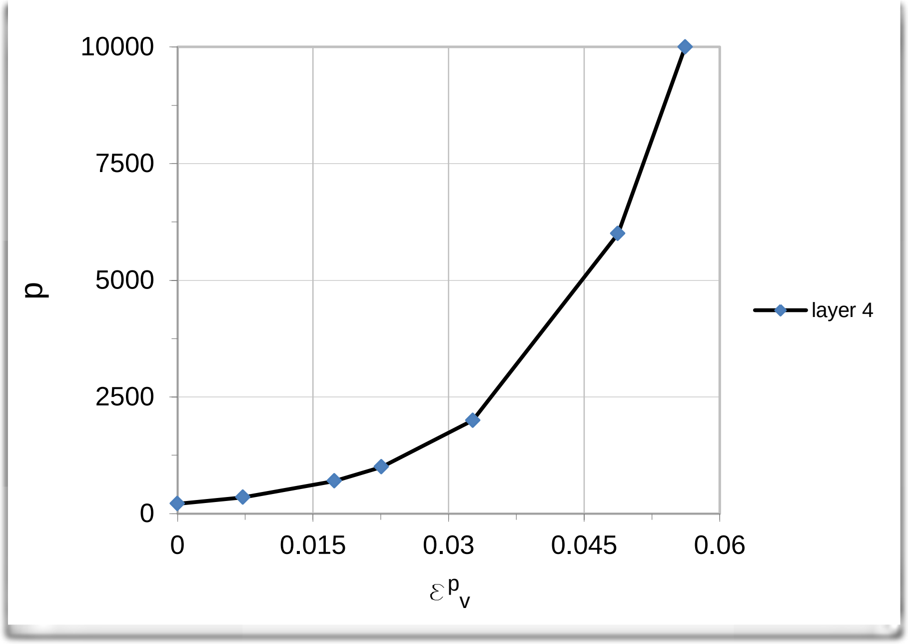


Figure 4.7 Cap hardening curve for sandy silty clay layer of dire dawa project soil

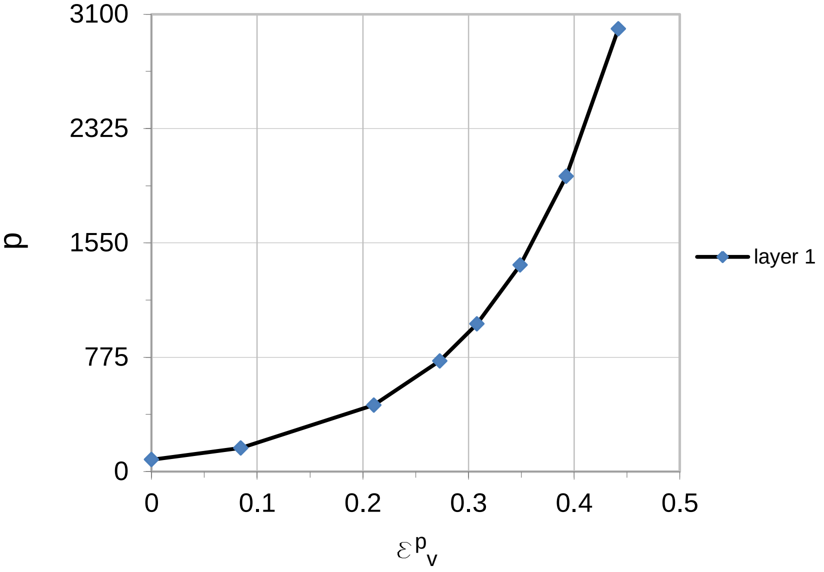


Figure 4.8 Cap hardening curve for top clay layer of Gidabo project soil

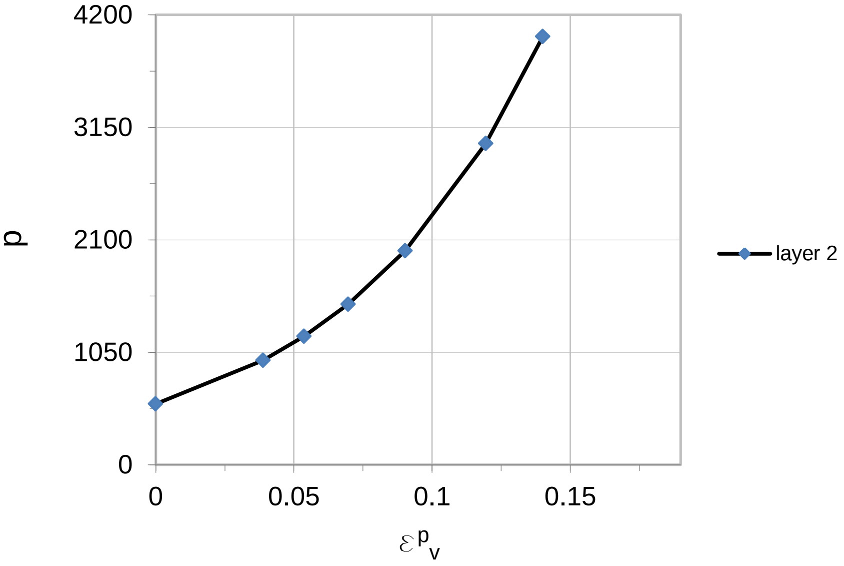


Figure 4.9 Cap hardening curve for sandy gravel layer of Gidabo project soil

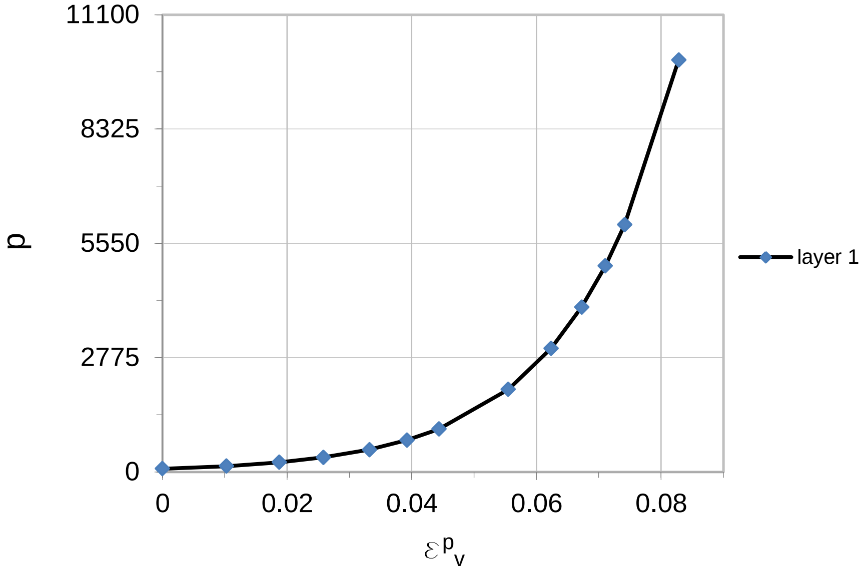


Figure 4.10 Cap hardening curve for top basaltic layer of K.K. project soil

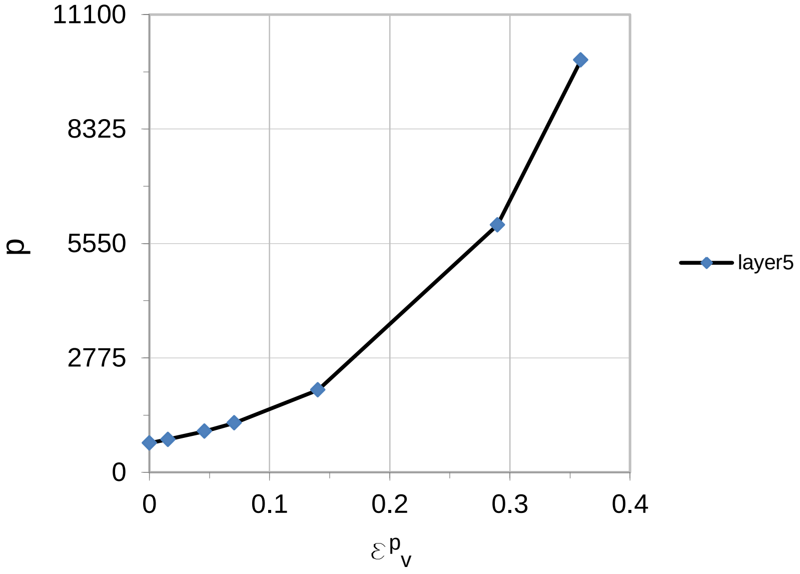


Figure 4.11 Cap hardening curve for tuff layer of K.K. project soil

4.1.2.3 Flow Rule

In this model the flow potential surface in the *p–t* plane consists of two parts, figure 4.12. In the cap region the plastic flow is defined by a flow potential that is identical to the yield surface (i.e., associated flow). For the Drucker–Prager failure surface and the transition yield surface, a non-associated flow is assumed: The shape of the flow potential in the *p–t* plane is different from the yield surface as shown in Figure 4.6. In the cap region the elliptical flow potential surface is given as

$$\boldsymbol{G_c} = \sqrt{(\boldsymbol{p} - \boldsymbol{p_a})^2 + \left(\frac{\boldsymbol{Rt}}{1 + \boldsymbol{\alpha} - \boldsymbol{\alpha} / \mathbf{cos}\boldsymbol{\beta}}\right)^2} \tag{4.19}$$

The elliptical flow potential surface portion in the Drucker–Prager failure and transition regions is given as

$$\boldsymbol{G_s} = \sqrt{[(\boldsymbol{p_a} - \boldsymbol{p})\mathbf{tan}\boldsymbol{\beta}]^2 + \left(\frac{\boldsymbol{t}}{1 + \boldsymbol{\alpha} - \boldsymbol{\alpha} / \mathbf{cos}\boldsymbol{\beta}}\right)^2} \tag{4.20}$$

As shown in Figure 4.12, the two elliptical portions, *Gc* and *Gs*, provide a continuous potential surface. Because of the non-associated flow used in this model, the material stiffness matrix is not symmetric. Thus, an un-symmetric solver should be used in association with the cap model. (Abacus tutorial 2007)

with:

$$\boldsymbol{p_a} = \frac{\boldsymbol{p_b} - \boldsymbol{Rd}}{1 + \boldsymbol{R}\tan\boldsymbol{\beta}}$$

p_b = compression yield stress
R = shape parameter of the yield surface FC
α = shape parameter for the transition surface Ft

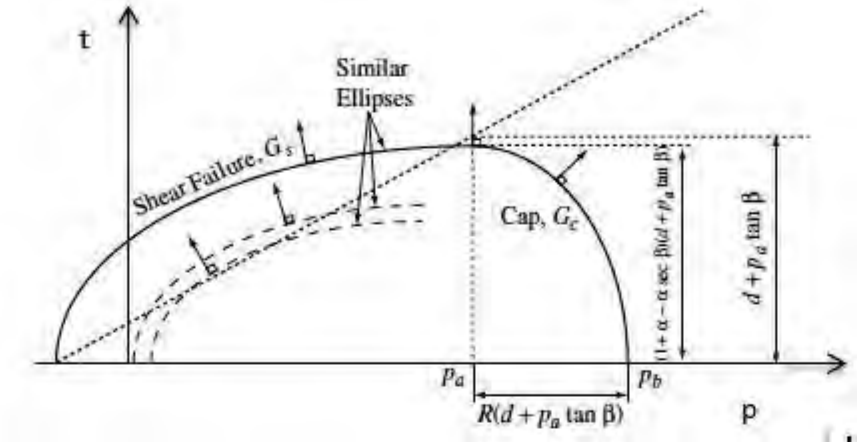


Figure 4.12 Flow potential of the modified cap model in the p-t plan (Adapted from ABACUS manual, 2002)

R and α are determined from tri-axial compression and extension tests with various stress paths. Since the experiments are extensive, their values are usually assumed based on experience when such experimental results are lacking, Sam Helwany (2007)

The values of d and β can be calculated from the values of cohesion and angle of friction according to:

$$\tan\boldsymbol{\beta} = \frac{6\sin\boldsymbol{\phi}'}{3-\sin\boldsymbol{\phi}'} \tag{4.21}$$

$$\boldsymbol{d} = \left(1-\frac{1}{3}\tan\boldsymbol{\beta}\right)\left(\frac{\cos\boldsymbol{\phi}'}{1-\sin\boldsymbol{\phi}'}\right)2\boldsymbol{c}' \tag{4.22}$$

$$\boldsymbol{K} = \frac{3-\sin\boldsymbol{\phi}'}{3+\sin\boldsymbol{\phi}'} \tag{4.23}$$

where $0.778 < K < 1$

The value of K in the linear Drucker-Prager model is restricted to K > 0.778 for the yield surface to remain convex. The result for K shows that, it implies $\emptyset < 22^0$. Many real materials have a larger of Mohr-Coulomb friction angle than this value. Hence the lower range is one approach in such circumstances to be chosen.

SUMMARY OF BASIC SOIL PARAMETERS FOR THE CAP MODEL

Basic soil parameters that are required as input in the cap constitutive models are summarized in Tables 4.4 to 4.6.

Table 4.4 summary of basic soil parameters for K.K. project

Soil parameter	Sym.	Unit	Highly weathered basalt	Strong basalt	Silty clay	Strong basalt	Tuff	scoria	Pile
Modulus of Elasticity	**E**	[MN/m^2]	50.00	100.0	75.1	100.0	115.1	98.3	29000.0
Poisson's ratio	**ν**	[-]	0.3	0.3	0.4	0.3	0.4	0.3	0.2
Unit weight	γ	kN/m^3	18.0	20.0	17.0	24.3	16.3	15.2	24
Angle of friction	**ø'**	[^{o}]	44.7	45	45	45	45	45	-
Slope of the yield surface Fs in the p-t plane	β	[^{o}]	61.4	61.6	61.6	61.6	61.6	61.6	-
Cohesion	**c**	[kN/m^2]	0.5	0.5	19	0.5	16.0	19.0	-
Intersection of yield surface Fs with the t-axis	**d**	[kN/m^2]	0.9	0.9	35.2	0.9	29.6	35.2	-
Initial cap position	ε^{pl}_{vo}	[-]	0.0	0.0	0.0	0.0	0.0	0.0	-
Cap Eccentricity	**R**	[-]	0.8	0.8	0.1	0.8	0.1	0.8	-
Flow stress ratio	**K**	[-]	0.788	0.788	0.788	0.788	0.788	0.788	-

Table 4.5 Summary of basic soil parameters for Dire Dawa Bridge project

Soil parameter	Sym.	Unit	Loose silty sand	Dense silty sand	Stiff silty sand	Sandy silty clay	Silty sand	Claye y sand	Pile
Modulus of Elasticity	**E**	[MN/m^2]	63.75	36.08	39.95	58.01	52.79	59.10	29000.0
Poisson's ratio	**ν**	[-]	0.3	0.3	0.3	0.4	0.3	0.3	0.2
Unit weight	**γ**	[kN/m^3]	17	17	17	17	17	17	24
Angle of friction	**ø'**	[°]	32	40.5	44.1	45	43.1	45.6	-
Slope of the yield surface Fs in the p-t plane	β	[°]	52.2	58.9	61.1	61.6	60.5	61.9	-
Cohesion	**c**	[kN/m^2]	5	6	8	8	6	5	-
Intersection of yield surface Fs with the t-axis	**d**	[kN/m^2]	10.3	11.6	15.0	14.8	11.3	9.2	-
Initial cap position	ε^{pl}_{vo}	[-]	0.0	0.0	0.0	0.0	0.0	0.0	-
Cap Eccentricity	**R**	[-]	0.8	0.8	0.8	0.1	0.8	0.8	-
Flow stress ratio	**K**	[-]	0.788	0.788	0.788	0.788	0.788	0.788	-

Table 4.6 Summary of basic soil parameters for Gidabo Dam project

Soil parameter	Sym.	Unit	Silty clay	Sandy gravel	sand	Silty clay sandy gravel	Pile
Modulus of Elasticity	**E**	[MN/m^2]	16.9	150.0	90.7	99.3	29000.0
Poisson's ratio	**ν**	[-]	0.4	0.3	0.3	0.3	0.2
Unit weight	γ	[kN/m^3]	17.8	16.4	16.4*	16.4*	24.0
Angle of friction	**ø'**	[^{o}]	30.9	44.7	43.5	45.6	-
Slope of the yield surface Fs in the p-t plane	β	[^{o}]	51.1	61.4	60.8	61.9	-
Cohesion	**c**	[kN/m^2]	23.0	0.0	0.0	0.0	-
Intersection of yield surface Fs with the t-axis	**d**	[kN/m^2]	47.6	0.0	0.0	0.0	-
Initial cap position	ε^{pl}_{vo}	[-]	0.0	0.0	0.0	0.0	-
Cap Eccentricity	**R**	[-]	0.1	0.8	0.8	0.8	-
Flow stress ratio	**K**	[-]	0.788	0.788	0.788	0.788	-

*at this layer level the unit weight is not determined in the laboratory; hence the preceding value is taken for successive layers below.

5 FEM MODELING AND PILE- SOIL INTERACTION

5.1 General

A theoretical solution must satisfy Equilibrium, Compatibility, the material Constitutive behavior and Boundary conditions (both force and displacement). Each of these conditions is considered separately below.

As described on Potts and Lidija Zdravkovic (1999), the finite element method involves the following steps.

Element discretization: this is the process of modeling the geometry of the problem under Investigation by assemblage of small regions, termed finite elements. These elements have nodes defined on the element boundaries, or within the element.

Primary variable approximation: a primary variable must be selected (e.g. displacements, stresses etc.) and rules as to how it should vary over a finite element established. This variation is expressed in terms of nodal values. In geotechnical engineering it is usual to adopt displacements as the primary variable.

Global equations: Combine element equations to form global equations

$$\{M_d\}= [K_G]\{\Delta d_G\} \qquad (5.1)$$

Where [KG] is the global stiffness matrix, $\{\Delta d_G\}$ is the vector of all incremental nodal displacements and $\{M_d\}$ is the vector of all incremental nodal forces.

Boundary conditions: Formulate boundary conditions and modify global equations. Loadings (e.g. line and point loads, pressures and body forces) affect {Md}, while the displacements affect $\{\Delta d_G\}$

Solve the global equations: The global Equations (5.1) are in the form of a large number of simultaneous equations. These are solved to obtain the displacements $\{\Delta d_G\}$ at all the nodes. From these nodal displacements secondary quantities, such as stresses and strains, are evaluated.

5.2 Interface Modeling for Pile Soil Interaction

Soil-structure interaction is an important topic in the study of behavior of interaction of soil with different structures .To be successful in using FEM methods in practical design, the soil model should sufficiently display the actual interaction of the soil with the structure. And also, the model should also be able to realistically capture the most important aspects of soil-structure nonlinearities.

Since it was first introduced, Drucker-Prager type model (Drucker and Prager, 1952) has been successfully adopted in analysis of geo-materials due to its relative simplicity. There are different methods of idealizing the contact between the soil and concrete foundation unit. Some of these idealizations were summarized by Reul (2000). According to Potts and Zdravkovic (2001), it is preferable not to use interface elements for the case of vertical loading. Researchers in the university of Delaware, Newark,DE have shown that, “with interface element lower capacity of pile is always predicted. They also noted that computational costs and numerical instability increase considerably in simulation with interface elements. Gang Wang et. Al.(2004)

Gebreziabher H.F. (2011) on his Ph.d dissertation had used an ideal contact without interface element and shown that an ideal contact can simulate the soil pile interaction in FEM model.

It is assumed that the pile is in perfect contact with the surrounding soil and hence the concept of ideal contact (as usually referred as “shear bands or shaft zone elements”, on studies made by Rolf Katzenbach et. al. 2000) is used throughout the analysis of this study. Fig 5.2 shows typical section of the shaft zone.

The shear band zones are continuum elements, with specified width normal the shaft surface and exhibit the same material behavior as the soil elements. Rolf Katzenbach et. Al. 2000, has further studied, effect of the width of the shaft zone elements on the pile resistance- settlement relationship.

Continuum elements of thickness 0.1D at the pile-soil interface have been applied throughout the numerical studies as recommended by Reul (2000) and de Sanctis and Mandolini (2003, 2006).

5.3 Finite Element Idealization

5.3.1 Discretization

In general, for this study, a circular concrete pile of different diameter is assumed. The pile is installed and is assumed to be perfectly bounded with the surrounding soil. The pile is vertically loaded on its top under un-drained condition. Then a finite element model was developed to simulate the pile soil interaction. Due to axisymmetric of this problem, only one half of the cross section is meshed using an 8 node, biquadratic, axisymmetric quadrilateral (CAX8) type element. As shown in Fig. 5.1, the mesh extends as equal length as the pile in depth below the pile and 1.5L in width in the horizontal direction in order to reduce the effect of boundary on the numerical results. The base of the mesh is fixed and only vertical movement is allowed along right hand side of the mesh and the axis of symmetry (the left hand side of the mesh). The pile is modeled with linearly elastic elements with a Young’s modulus E= 29GPa and Poisson’s ratio v=0.2.

A typical model with meshing techniques is shown in Fig. 5.1 below

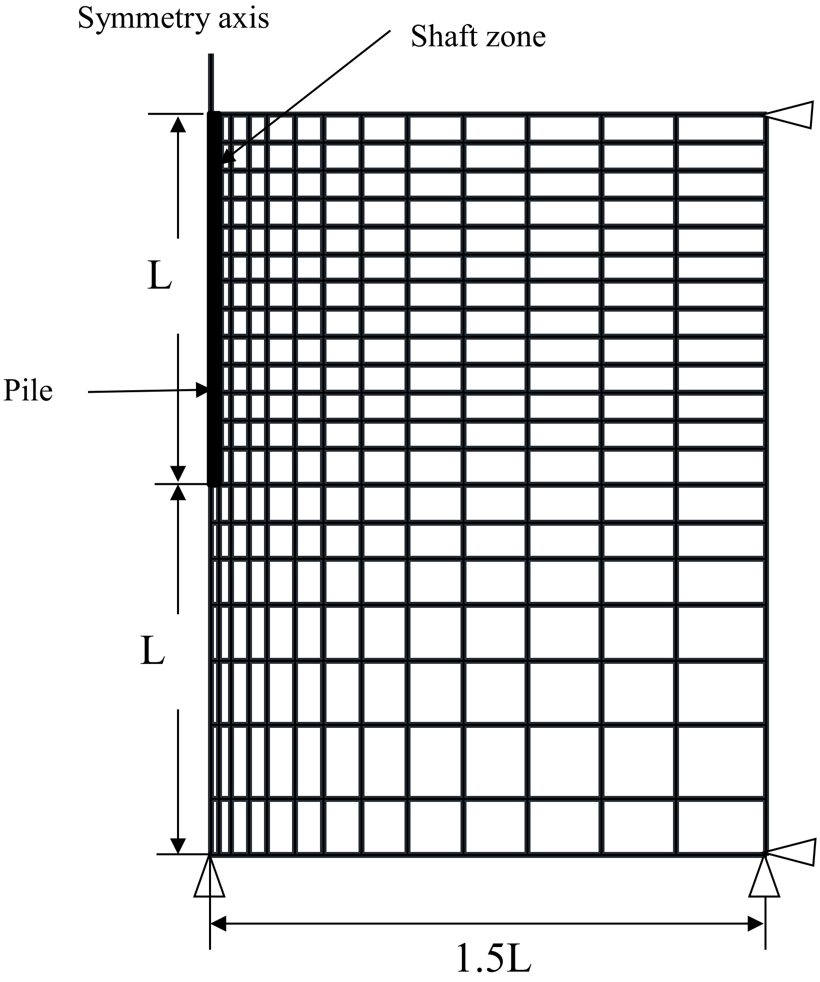


Figure 5.1 Typical discretization of the computational model

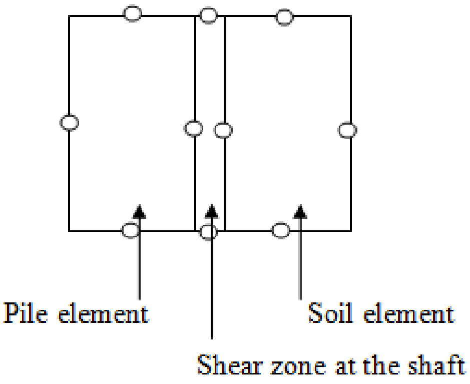


Figure 5.2 Enlarged mesh at the shear zone between the pile and soil

5.3.2 Mesh Dependency

The effect of mesh types applied just beneath the pile tip is seen in this study. Numerical model done for Pile1 of Dire Dawa project is taken to see the effect of applying different types of meshing techniques just below the bottom of the pile, as shown in fig. 5.3 and 5.4. Mesh on fig. 5.3 is coarser and the pile is not meshed vertically. Whereas mesh shown in fig 5.4 is finer and the pile is also meshed vertically aligned with the bottom soil mesh elements. Analysis of these two types of mesh is done using MC model on pile1 from Dire Dawa project.

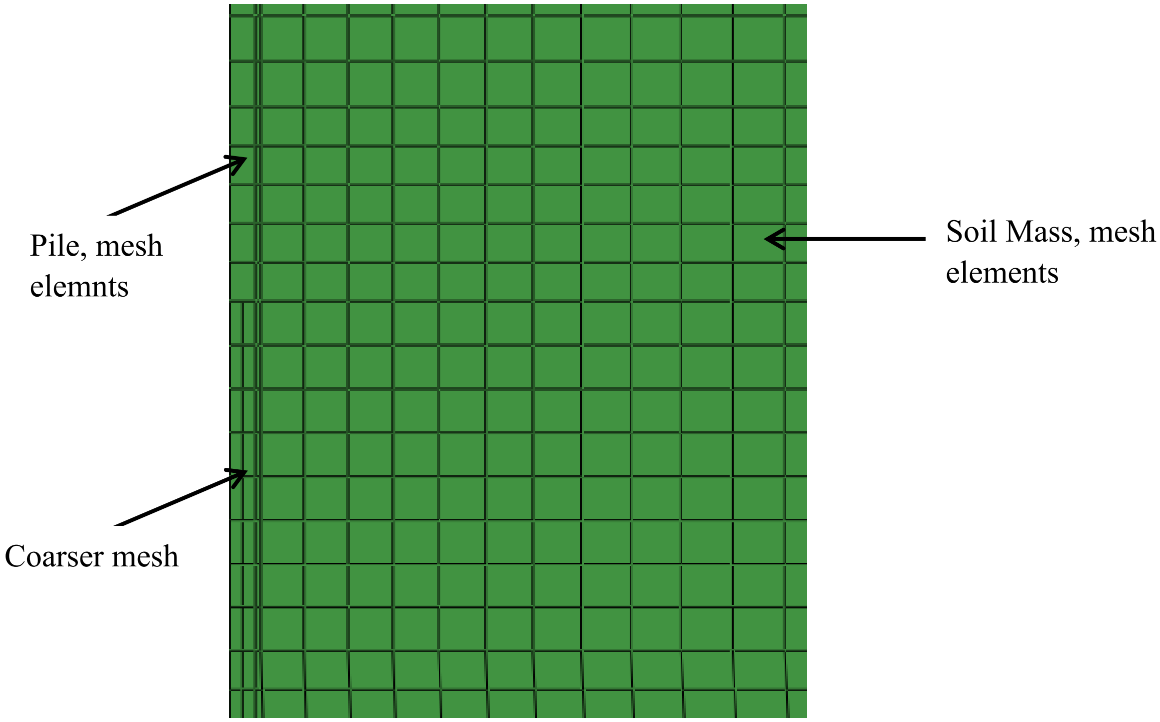


Figure 5.3 Coarser mesh below bottom of pile (mesh type 1)

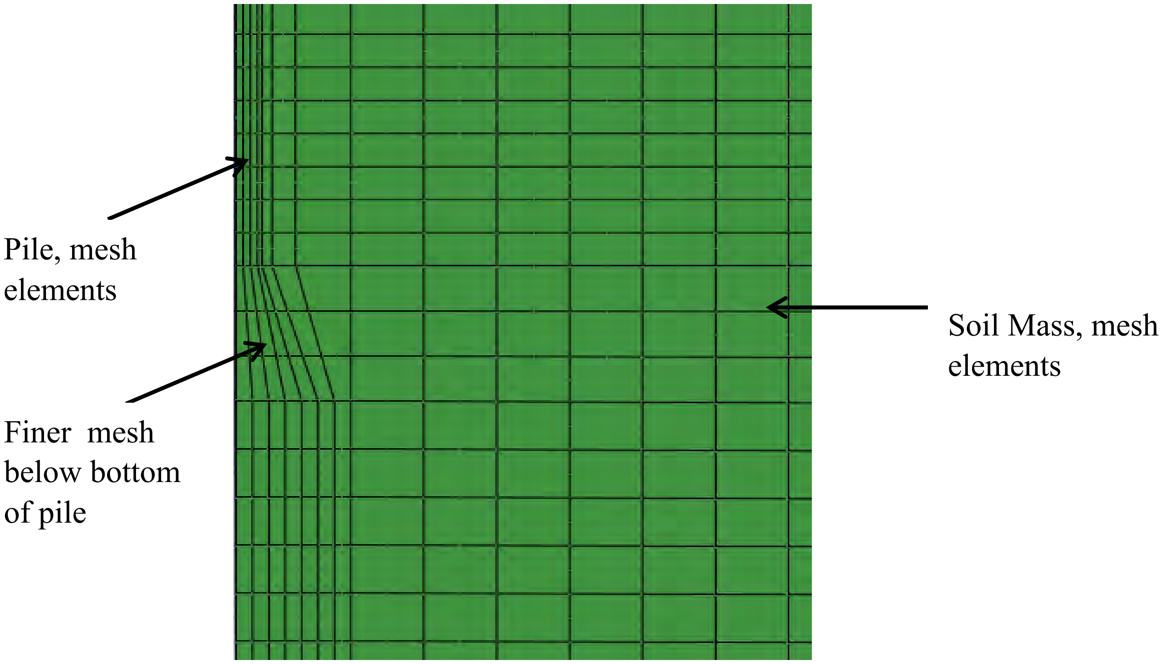


Figure 5.4 finer mesh below pile tip (mesh type 2)

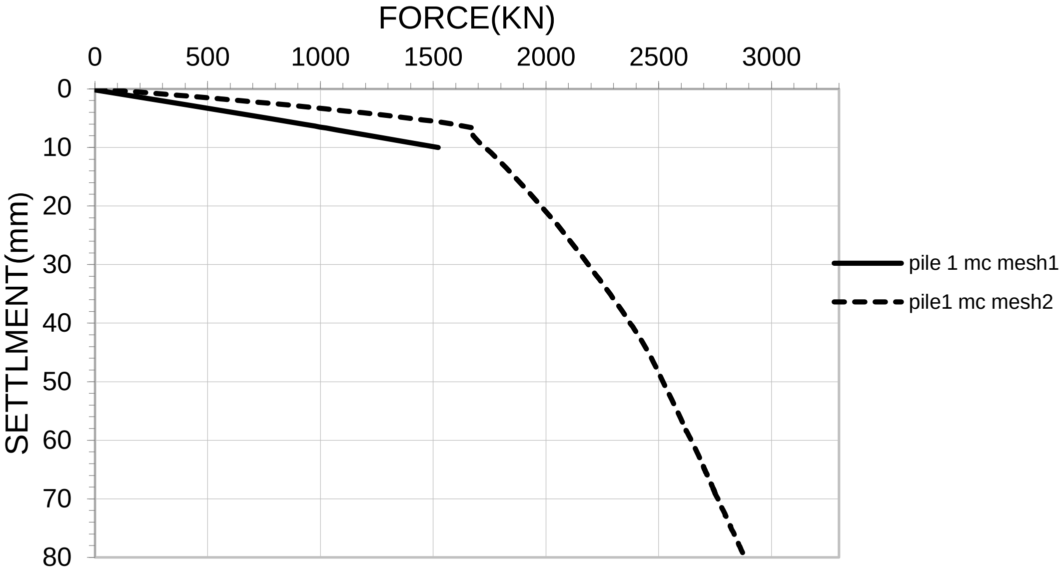


Figure 5.5 Effect of mesh type on load settlement curve

Fig.5.5, shows that coarser mesh elements below the pile tip results in reduced capacity of the pile, numerical instability, and termination of analysis at lower loading steps. But the second mesh type shown on fig. 5.4, gives better result in terms of the ultimate capacity of the pile and giving analysis results further in the plastic deformation of the soil.

5.3.3 Simulation of Un-Drained Condition

As pile loading test demands a short period of time as compared to the time required for consolidation to take place in soil layers, there will be no time for excess pore water to drain out of the soil. Hence the un-drained (short term) condition will best simulate the model with the actual pile load test.

Four main measures must be considered for a successful finite element analysis of soils considering their short-term (un-drained) behavior, Sam Helwamey 2007,
(1) The initial conditions of the soil strata (initial geostatic stresses, initial pore water pressures, and initial void ratios) must be estimated carefully and implemented in the analysis. The initial conditions will determine the initial stiffness and strength of the soil strata;
(2) The boundary conditions must be defined carefully as being pervious or impervious;
(3) The long-term strength parameters of the soil must be used in an appropriate soil model; and
(4) loads must be applied quickly. Fast loading does not allow enough time for the pore water pressure to dissipate, thus invoking the short-term strength of the soil. This means that there is no need to input the short term strength parameters because the constitutive model will react to fast loading in an "un-drained" manner.

5.3.4 Analysis Steps

The general analysis step, which can be used to analyze linear and or non-linear response, is selected for the analysis, (Abaqus tutorial, 2013).

Analysis steps can be broadly categorized as initial step and analysis step. Abaqus/CAE creates a special initial step at the beginning of the model's step and allows defining boundary condition, predefined fields. Whereas the analysis step is followed by one or more subsequent analysis steps where each subsequent step is associated with a specific procedure that define the type of the analysis to be performed. In this study different analysis steps have been followed to simulate the actual pile loading tests as mentioned below:
. 1. Initial stress condition (geostatic condition)
The actual in situ stress initial condition of the soil mass is simulated in this step. Equilibrium condition of the internal stress is checked here.
2. Excavation:
In this step bore holes created for the pile to be installed are simulated by removing all the elements at the pile location. Any stress variation due to the excavation will be checked based on the equilibrium condition.
3. Pile installation:
In this step installation of the foundation piles is simulated by putting in place finite elements representing the pile at the location where the preceding step performed removal of soil elements.
4. Loading:

In this step the test loads are applied in incremental manner to simulate the actual pile load test procedures. There are two types of load application in numerical models. The first is by applying direct loads on the nodal points and the second one is by applying a predetermined displacement on the top of the pile.

Typical pile loading test procedure requires more than 30 load incremental steps for the full cycle of loading and unloading procedures, as indicated in chapter 3. This means at least 30 numerical analysis steps are required to simulate the pile load test using FEM methods. Whereas the later method of applying predefined displacement requires only one analysis step. This is a big difference in terms of computational costs.

Both methods give identical results, and this is proved by taking one typical pile from K.K. project and subject it to loading by both methods: Mohr- Coulomb model was used for the analysis, and as the result shown on fig 5.6, the two methods gives identically the same output on the load vs settlement graph.

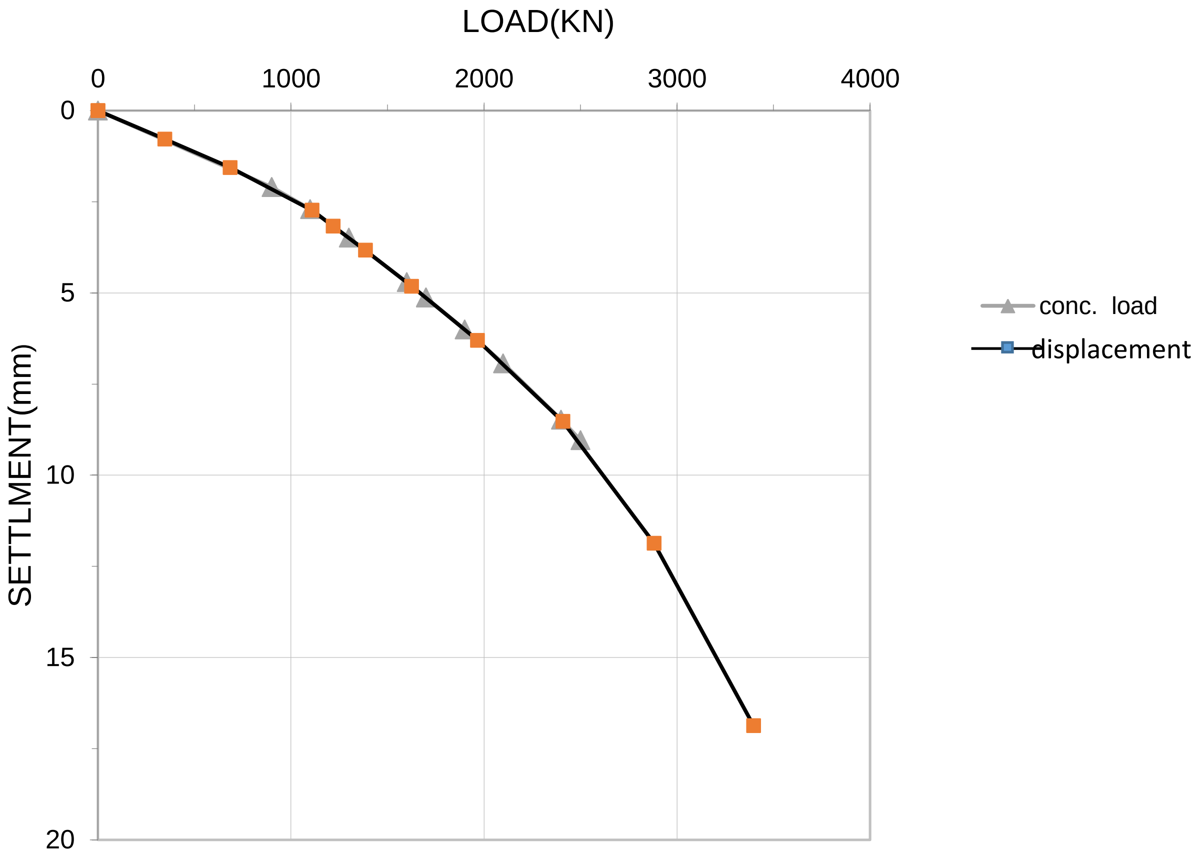


Figure 5.6 Comparison of method of load application

6 ANALYSIS RESULT AND PARAMETRIC STUDY

6.1 Validation of FEM Result

Computation of sum of vertical load along the bottom datum line of the soil mass is performed to compare results from FEM model with simple hand calculation method. Pile-1 from K.K. and Dire Dawa and pile 2 from Gidabo project have been selected to check the validation of the FEM model. Both Mohr Coulomb and Cap constitutive models have been used for the comparison. Tables 6.1 to 6.3 show that the difference between FEM and hand calculation results are insignificant, hence the chosen FEM models can be taken as applicable for further analysis in this study.

Table 6.1 Comparison of total vertical load (FEM vs. hand calculations), Dire dawa project

Model type	Analysis steps	Total Vertical Load (KN)		Difference (%)
		Theoretical	FEM	
Cap model	Initial Stress	1227508.2	1227508.2	0.00
	Final loading	1232186.3	1234590.8	0.19
MC model	Initial Stress	1227508.2	1227508.2	0.00
	Loading	1230388.9	1232044.5	0.13

Table 6.2 Comparision of total vertical load (FEM vs. hand calculations), K.K. project

Model type	Analysis steps	Total Vertical Load (KN)		Difference (%)
		Theoretical	FEM	
Cap model	Initial Stress	633737.78	633737.78	0.00
	Final loading	640564.24	645614.96	0.78
MC model	Initial Stress	633737.78	633737.78	0.00
	Loading	642640.63	644321.01	0.26

Table 6.3 Comparision of total vertical load (FEM vs. hand calculations), Gidabo project

Model type	Analysis steps	Total Vertical Load (KN)		Difference (%)
		Theoretical	FEM	
Cap model	Initial Stress	5908278.05	5908278.05	0.00
	Final loading	5925360.45	5930921.80	0.09
MC model	Initial Stress	5908278.05	5908278.05	0.00
	Loading	5916636.48	5919554.3	0.05

6.2 FEM Analysis result and Comparison with actual pile load test

6.2.1 Vertical stress analysis result

Typical analysis results of vertical stress values expressed in units of ‘kN/m^2’, at intial geostatic state are shown below in Fig. 6.1 to Fig.6.3 for Gidabo pile1, K.K. pile2 and Dire Dawa pile1, respectively.

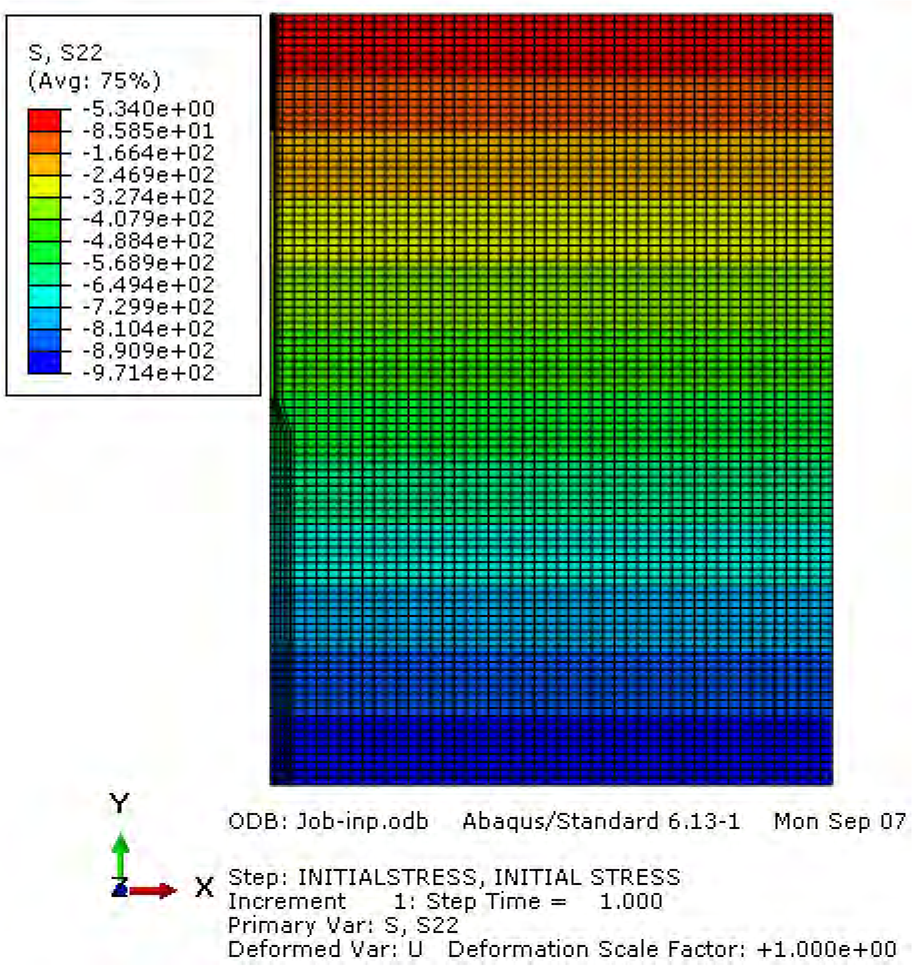


Figure 6.1 Vertical stress at initial condition, Gidabo pile 1 with cap model

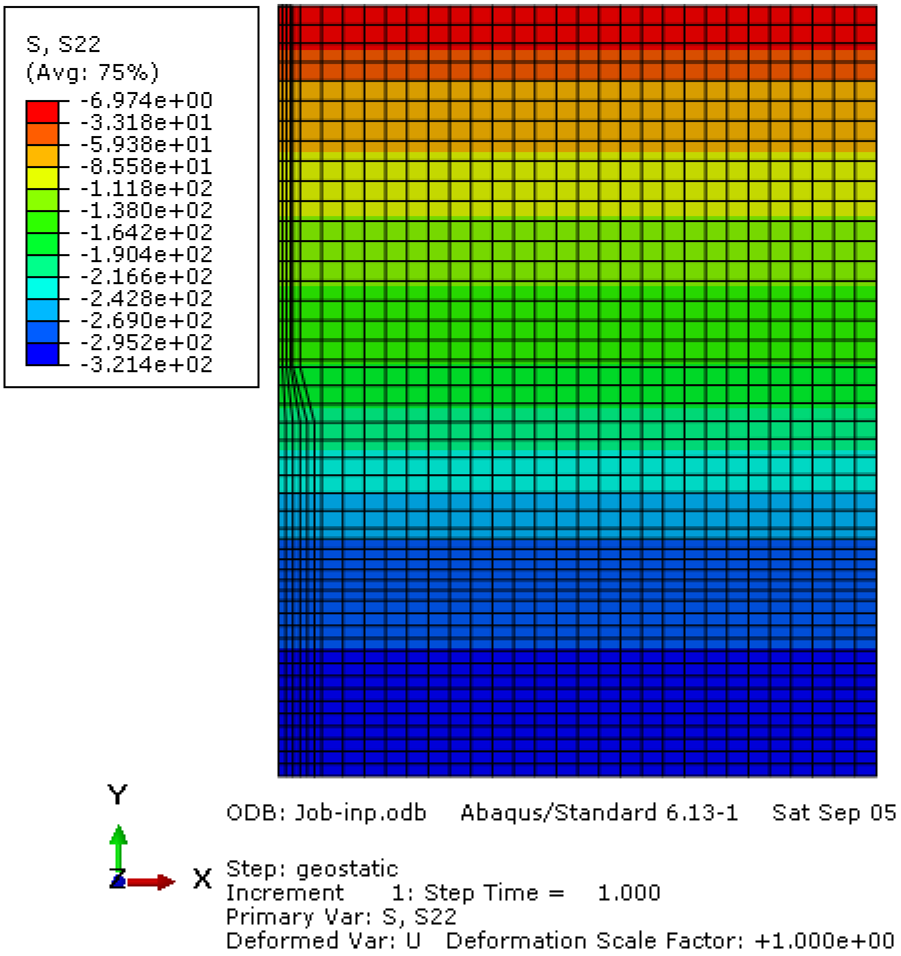


Figure 6.2 Vertical stress at initial condition, K.K. Pile 2 with cap model

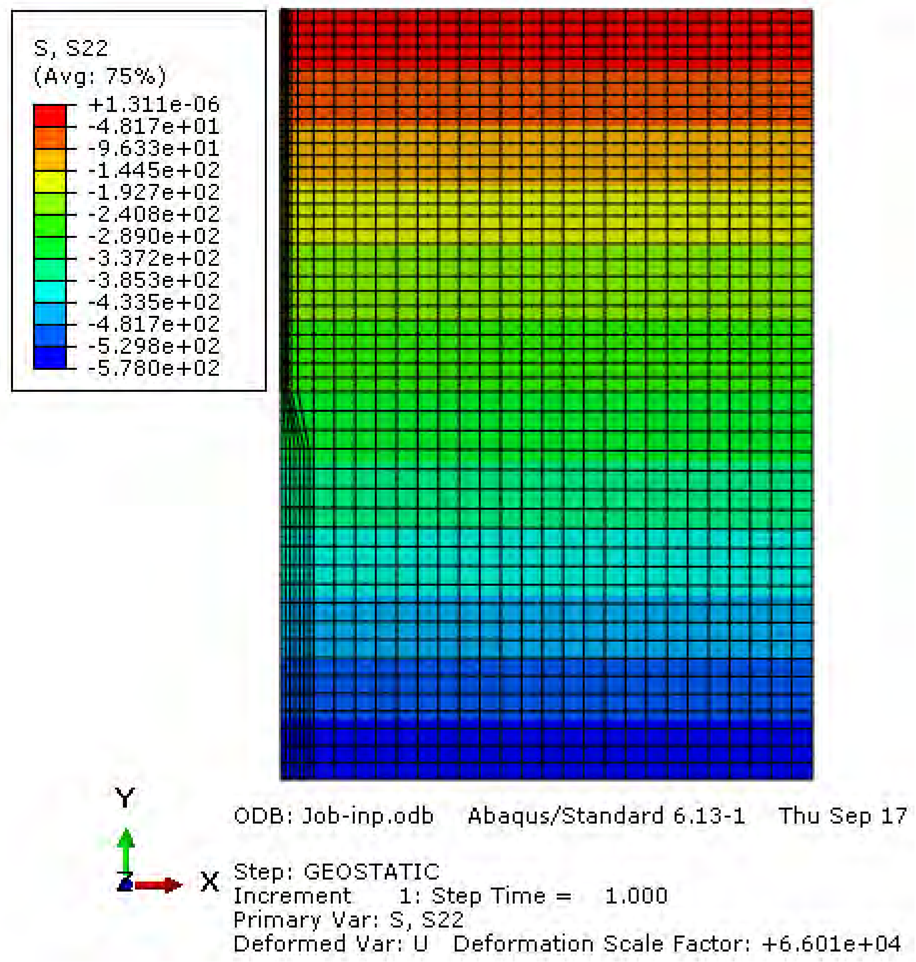


Figure 6.3 Vertical stress at initial condition, Dire Dawa pile 1 with cap model

6.2.2 Load settlement analysis result

Load settlement graph of both MC and Cap models numerical analysis results are compared with the respective actual pile load test as shown below in Fig. 6.4 to Fig. 6.9.

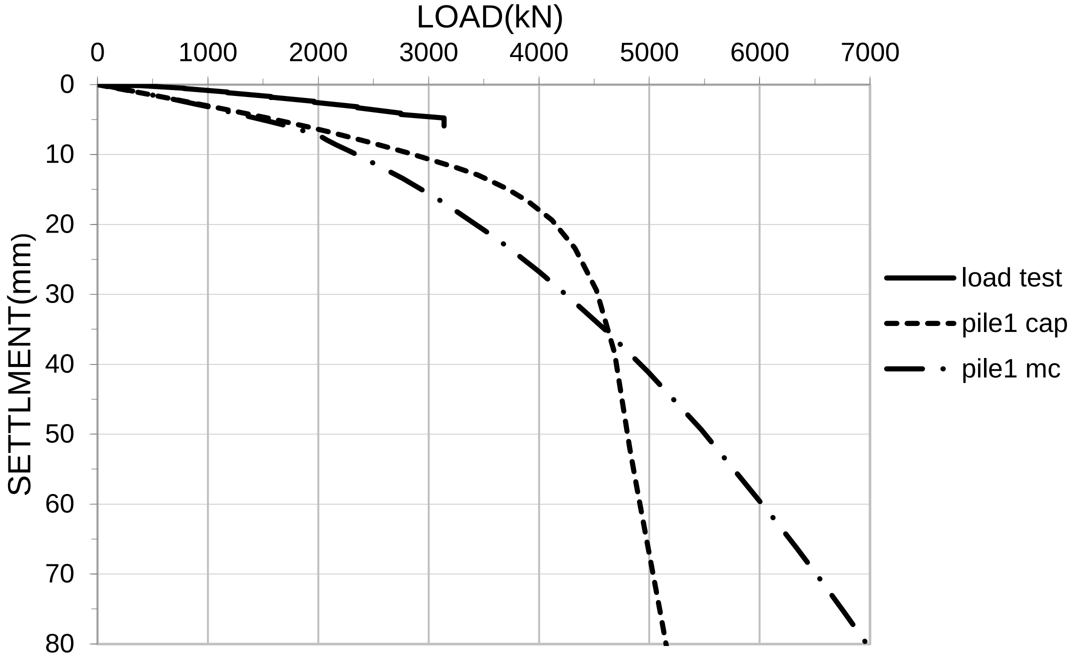


Figure 6.4 comparison of FEM result with pile load test (Dire Dawa pile1)

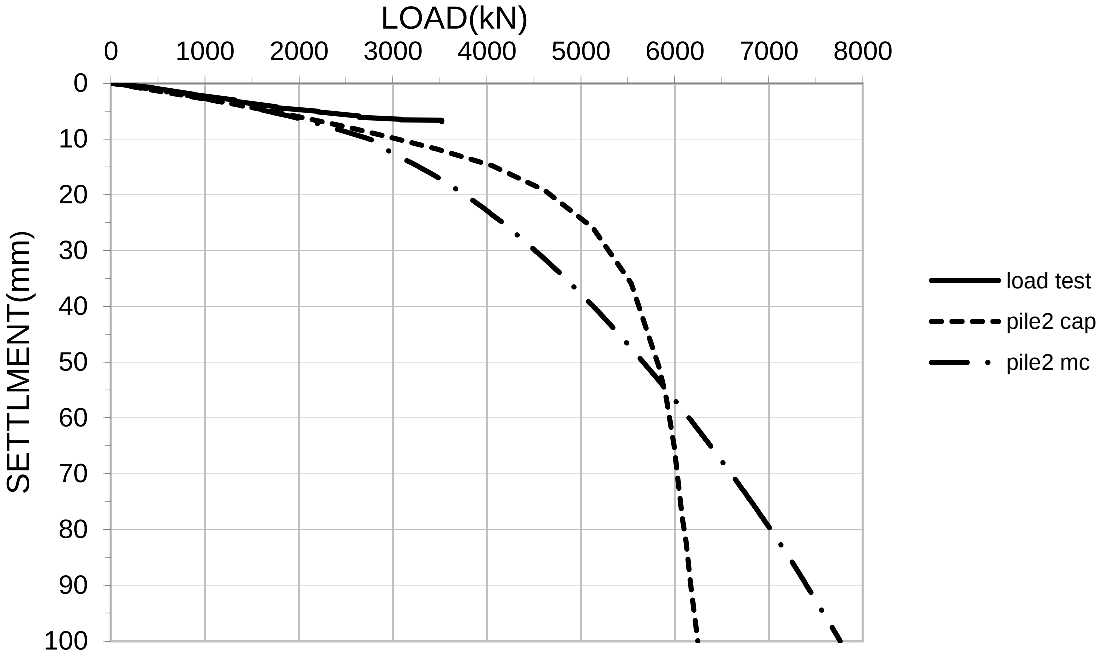


Figure 6.5 comparison of FEM result with actual pile load test (Dire Dawa pile2)

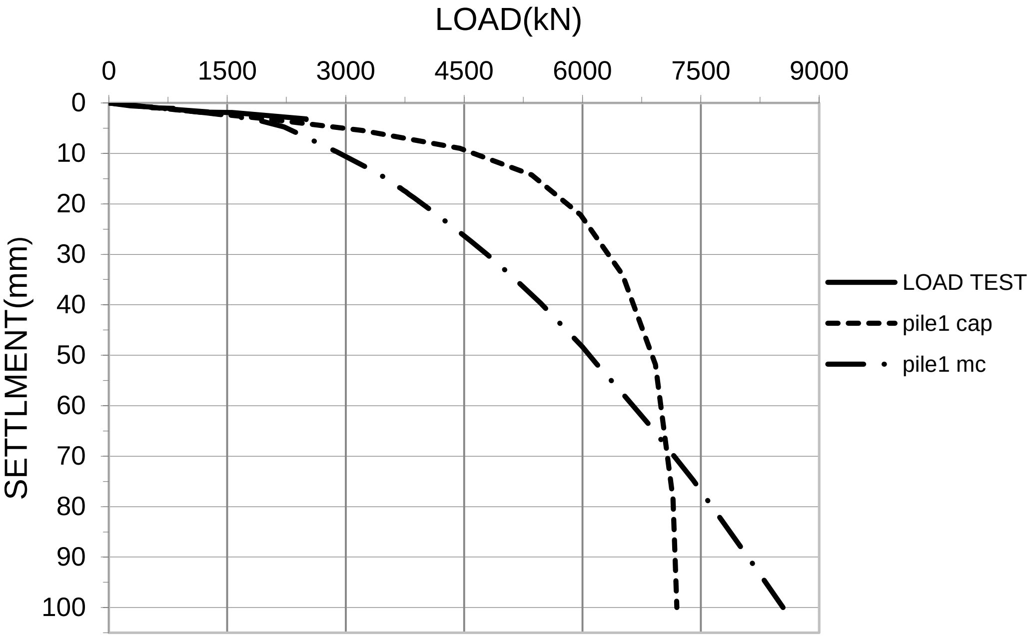


Figure 6.6 comparison of FEM result with actual pile load test (K.K pile 1)

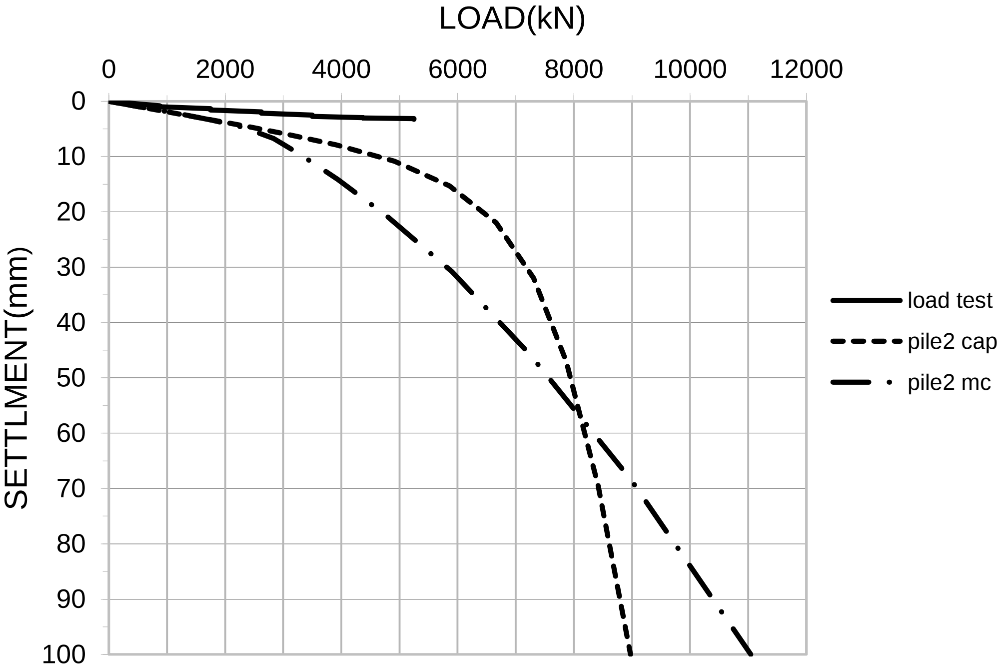


Figure 6.7 comparison of FEM result with actual pile load test (K.K pile 2)

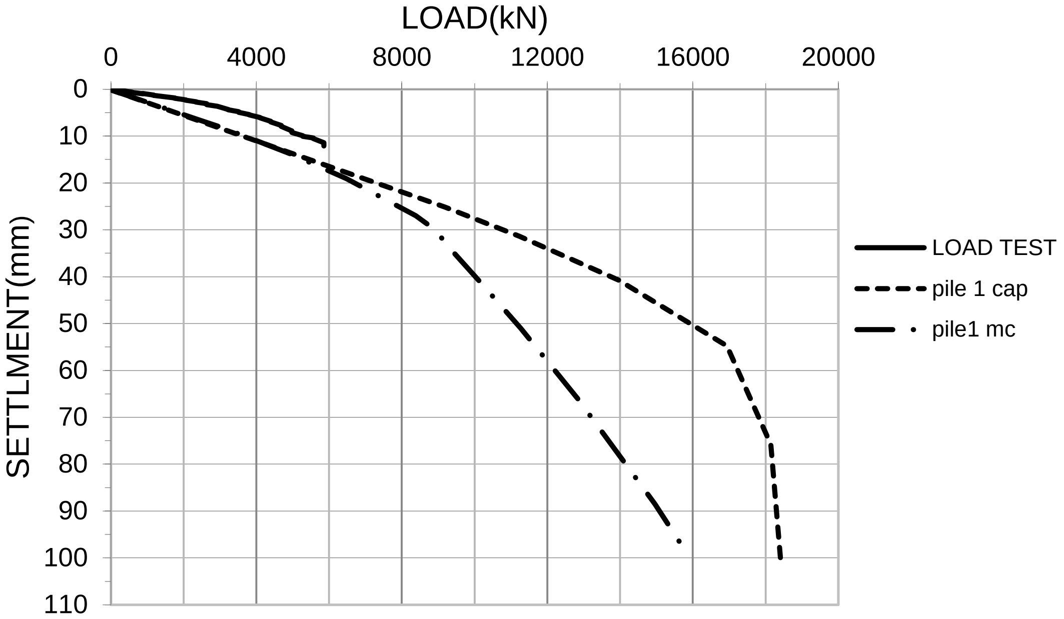


Figure 6.8 comparison of FEM result with actual pile load test (Gidabo pile 1)

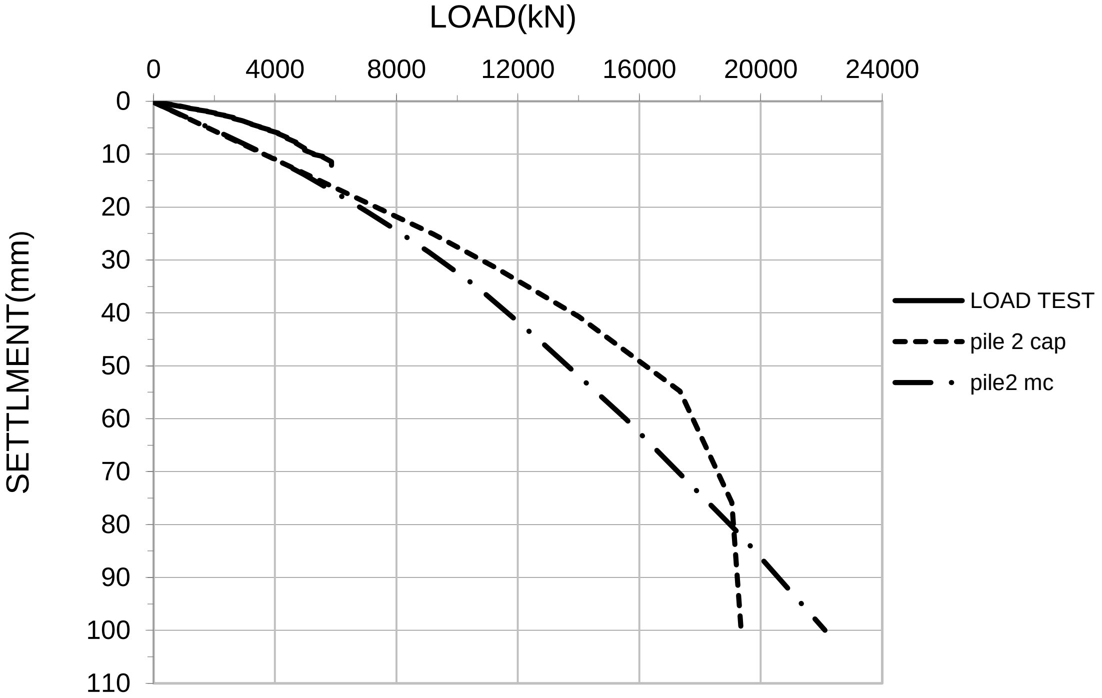


Figure 6.9 comparison of FEM result with actual pile load test (Gidabo pile 2)

As shown in Fig. 6.4 and 6.5, for both piles (pile1 and pile 2 from Dire Dawa project), numerical analysis results obtained based on cap model shows a closer result towards the actual load test as compared with the other results obtained based on Mohr Coulomb model. Both piles with Mohr Coulomb model show similar nature in the elastic deformation range, whereas once the plastic deformation started, a continuous increase in the ultimate resistance of the pile is observed. Load settlement curves of analysis result obtained using the cap model show similar nature of deformation in both piles. The piles seem to be reaching their apparent yielding capacity in cap models. Comparison with actual load test could not be done in the plastic deformation as the load test stops early at 200 percent of the working load.

Analysis results of piles from K.K. project, which are shown in Fig. 6.6 and 6.7, indicate that the cap model gives closer simulation result towards the actual load test in all elastic deformation ranges and in some portion of the plastic deformation. The results from cap models show apparent yielding tendency, whereas the Mohr Coulomb model results shows no tendency of failure but a continuous increase in the ultimate resistance of piles. Pile load test results are stopped early at 200 percent of the working load and no full comparison of load settlement curve is possible.

Fig. 6.8 and 6.9 shows comparison of analysis result of piles from Gidabo project with the respective load tests. Analysis result of pile1 with cap model gives better simulation result than the Mohr Coulomb model. In case of pile 2 both models show comparable result but still the cap model curve is above the Mohr Coulomb in all elastic and most part of plastic deformation regions.

In general, the cap model shows a better simulation result towards the load test and the pile reached is yielding point due to the elliptical cap yield surface that bounds the yield surface of the cap model. However, the Mohr Coulomb model shows no distinct failure but a continuous increase in the resistance force.

6.3 PARAMETRIC STUDY

6.3.1 Stiffness Modulus

Parametric study on the FEM analysis has been carried out by varying parameter like stiffness modulus of the soil layers above and below the base of the pile tip. For this study only the analysis done using cap model has been considered on selected pile from the K.K. project (pile2). Results are shown in Fig. 6.10 to 6.13.

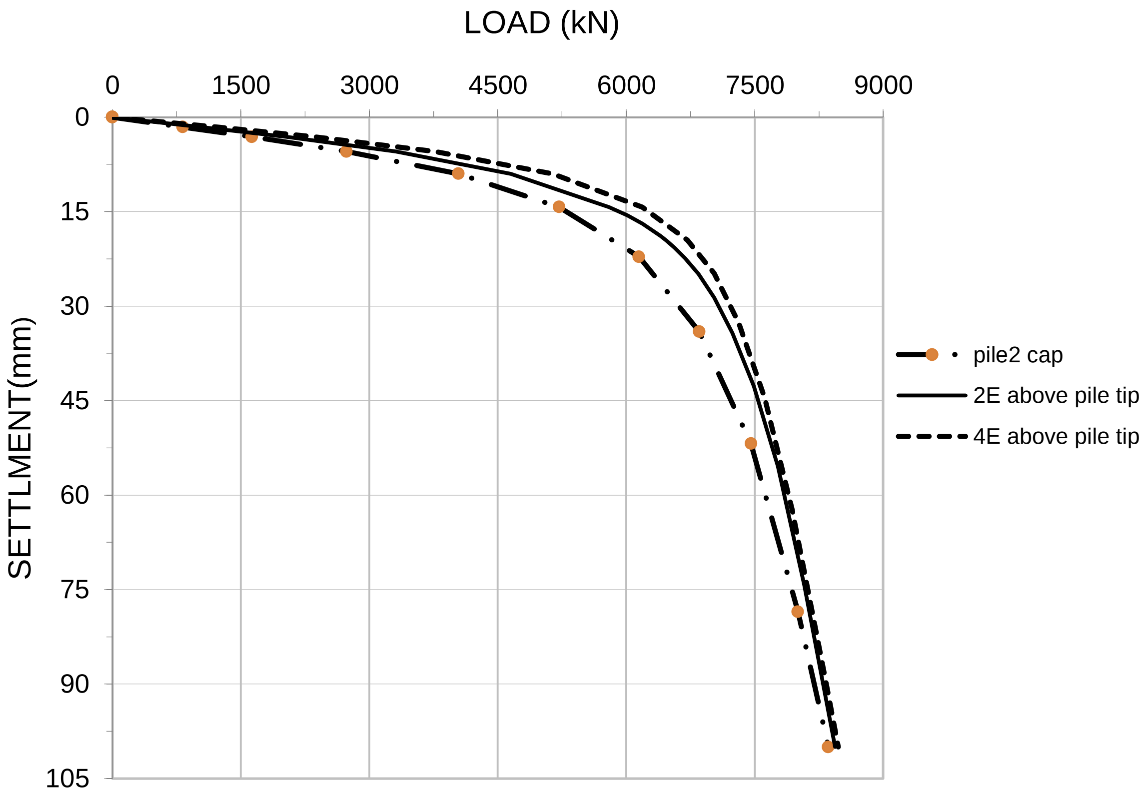


Figure 6.10 Effect of stiffness modulus of soil above bottom of pile.

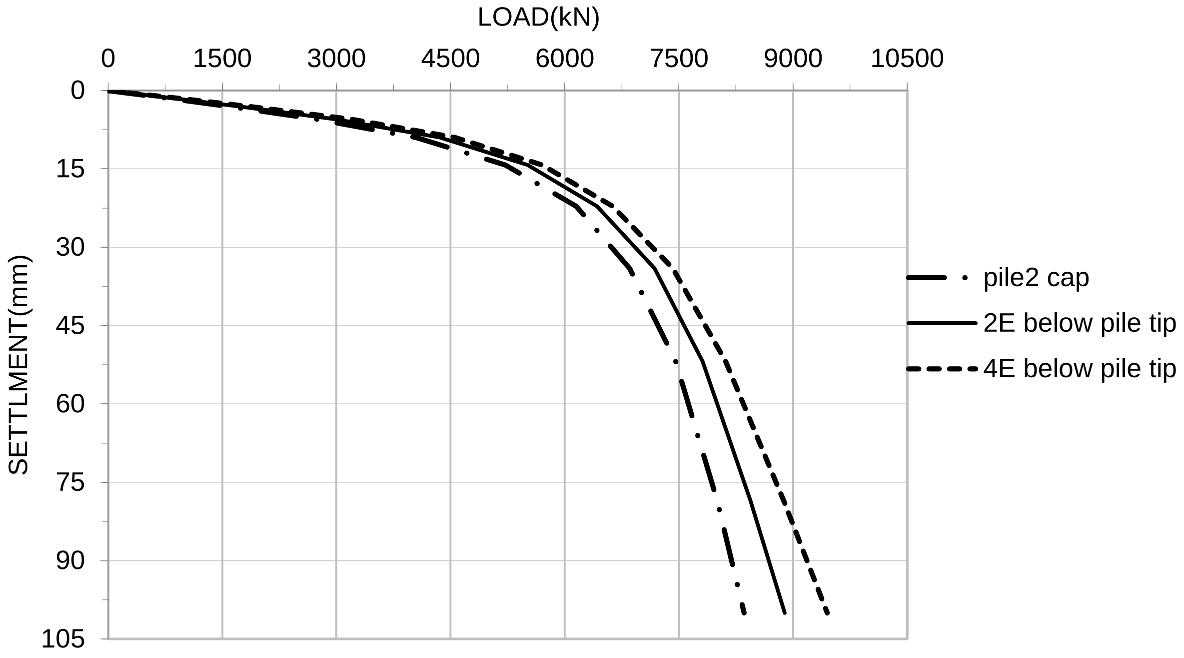


Figure 6.11 Effect of stiffness modulus of soil below bottom of pile.

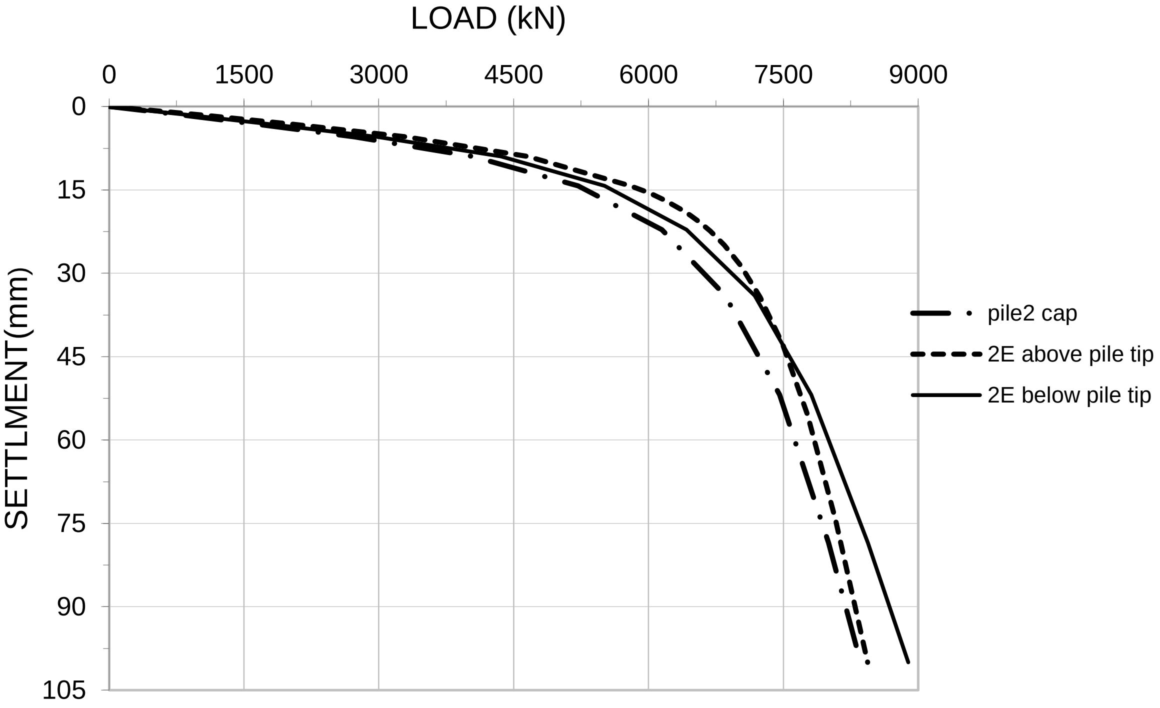


Figure 6.12 Effect of stiffness modulus of soil above and below of bottom of pile (2E)

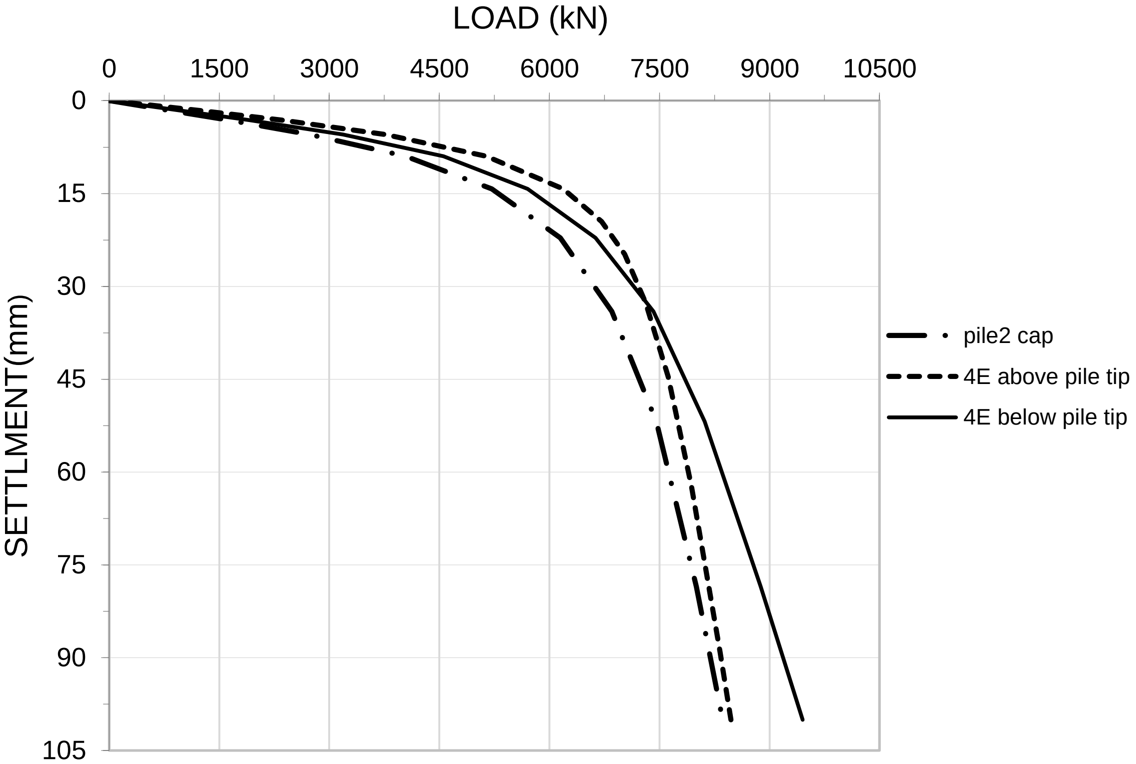


Figure 6.13 Effect of stiffness modulus of soil above and below of bottom of pile(4E)

The result shows that the stiffer the material above the pile tip the better will be the ultimate capacity of the pile, this result shows that since the piles are friction piles stiffness below the pile tip do not contribute much to the bearing capacity of the pile.

6.3.2 Soil Dilatancy

Effect of soil dilatancy, ψ, was assessed by examining the load deflection response for varying soil dilatancy. The soil friction, $\emptyset$, and cohesion were used as obtained in chapter three. To study this, Mohr Coulomb soil model with Pile No. 1 from Dire Dawa Bridge project is taken. Different values of ψ, ranging from $\psi = 0$ to $\psi = \emptyset$ has been taken and the corresponding effect on load settlement curve is shown in Fig.6.14.

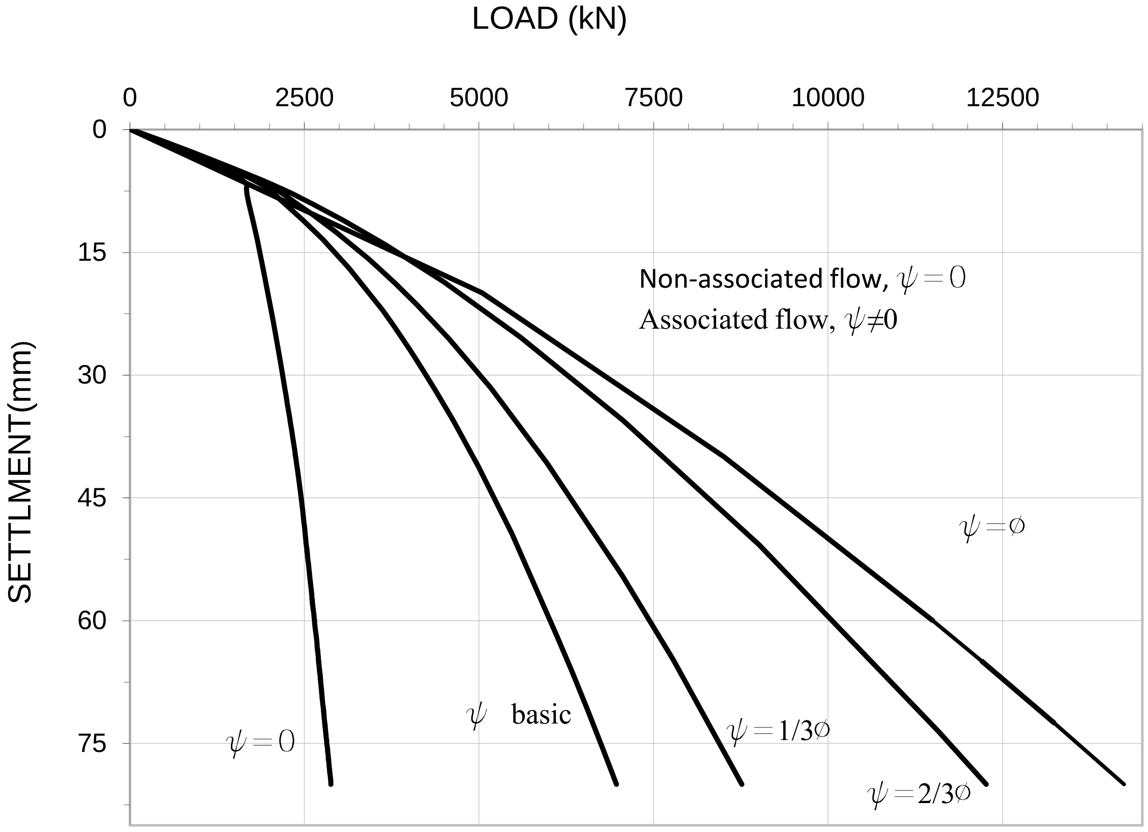


Figure 6.14 Effect of Dilatancy on load settlement curve

Pile capacity reached its yielding apparently in the non-dilatant soil $(\psi = 0)$, where the volume expansion is zero, resulting in much more realistic computational result. The associative flow rule with higher angle of dilatancy, exhibits no distinct failure, but rather a continuous increase in the resistance force.

6.3.3 Angle of Internal Friction

Effect of friction angle of a soil, $\varnothing$, on the load settlement graph of a soil model was seen by varying values of the friction angle as $\varnothing$, $1.2\varnothing$, $1.5\varnothing$ and $2\varnothing$. The result is presented in Fig. 6.15 below.

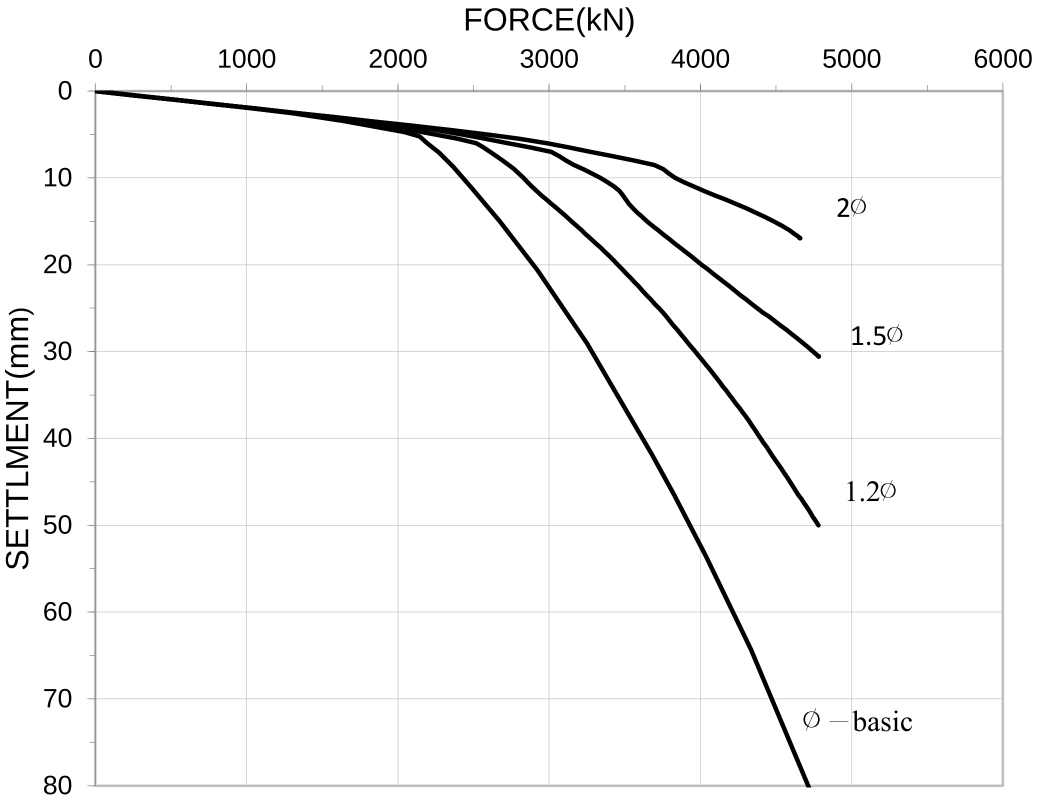


Figure 6.15 Effect of angle of internal friction on load settlement curve.

Increasing the friction angle of a soil do not help much in getting better simulation result as compared with the loading test. Rather it shows an increase in resistance foe a given settlement. The nature of the graph in the elastic deformation remains the same for all the cases of ∅ taken for the parametric study.

6.4 Conclusion

Numerical analysis method, such as finite element method, is widely used to predict the bearing capacity and settlement of pile foundation. ABAQUS, as a general-purpose finite element analysis software package, is commonly used in geotechnical engineering, due to its powerful capability in non-linear analysis. In this paper, simulation of pile load tests using FEM analysis was examined and the resulting load settlement curves are compared with actual pile load tests. The following conclusions and recommendations can be drawn from the result.

- Two constitutive models, Mohr Coulomb and Cap plasticity (modified Drucker Prager) models are used in the FEM analysis. In all the cases of this study, the cap model gives better simulation result than the Mohr Coulomb when compared with the actual load test. This is due to the different formulation of stiffness in the two models, in the MC model a constant stiffness is used while in the cap model a stress dependent stiffness is included. The total resistance for the MC model increases linearly up to the pick point for all cases. In the cap model of K.K. mixed use building and Dire Dawa Bridge projects, no linear increase of the resistance of the pile is observed. The shape of the curve is hyperbolic and the pick value is higher than the MC model.

- Modeling of the interface behavior between the pile and the soil is important in the analysis of pile under vertical loading. In the numerical analysis done, no specific interface element was used and the pile is assumed in perfect contact with adjacent soils, which is an ideal contact in the shear band zone. From the result obtained so far, it can be deduced that the ideal contact interface used in this study is capable of modeling the pile soil interaction mechanism.

- Applying finer mesh for soil elements just beneath the bottom of pile and meshing the pile accordingly, gives better simulation result.

- Parametric study, by varying the stiffness modulus, angle of dilatancy, and angle of internal friction of the soil, has been performed to see their effect on the behavior of load settlement curve. The stiffness parameter is one of the basic soil factors that has a significant effect on the ultimate resistance of the pile. Varying its value for the different soil layers located below and above the bottom of the pile is considered in this parametric study. As we can see from the result, increasing stiffness values of soil layer located above the bottom of the pile, gives a better simulation result closer to the actual load test than increasing stiffness of soils layers beneath the bottom of the pile. This is in good accordance with the expected behavior of friction piles.

When using Mohr Coulomb and Drucker Pragar model, attention has to be given to the angle of dilatancy, which is constant in the model. Pile capacity reached its yielding apparently in the non-dilatant soil $(\psi = 0)$, where the volume expansion is zero, resulting in much more realistic computational result. The associative flow rule with higher angle of dilatancy, exhibits no distinct failure, but rather a continuous increase in the resistance force.

Effect of angle of internal friction of the soil on the load settlement curve has also been examined by taking different values of $\emptyset$. Increasing the friction angle of a soil do not help in getting better simulation result as compared with the loading test. The nature of the graph in the elastic deformation remains the same for all the cases of $\emptyset$ taken for the parametric study.

- Finally, with the rapid growing numerical analysis using a finite element method, practicing engineers currently are making use of computational models to solve different geotechnical problems. Hence, by incorporating more real soil data and parameters representing the actual soil formation, and by selecting a proper constitutive models and theories, simulation results of pile load test can be taken as one alternative to estimate the ultimate capacity of a vertically loaded pile without performing the static load test.

6.5 Recommendations

- All pile load tests obtained for this study are not completely loaded till failure, only 200 percent of the design load is applied. Whereas the Numerical simulation models are loaded far beyond the working load as can been seen in the load settlement curves. Due to this comparison of ultimate bearing capacity of piles could not be obtained as the actual load test stops before reaching the settlement where the capacity of piles are computed. Therefore, it is recommended that loading tests performed in different construction areas of the country must incorporate at least one failure load test in addition to the test done on working piles.

- Most of soil parameters required for numerical analysis used in this study are obtained from correlation techniques set forth by different literatures. Only very limited parameters are directly taken from test result. This is due to lack of sufficient and adequate laboratory and field test results for the different soil layers at different depth. This can be raised as one significant drawback in this study. A better simulation result of FEM could have been obtained, if most soil parameters have been directly determined from laboratory and field test results.

From this a recommendation can be drawn that more has to be done on the way soil investigation tests are being carried out, in terms of need of more soil parameters and the way they are determined. Atterberg and shear strength parameters, which are most common in soil test results, are done on soil samples obtained from shallow depth and from certain part of the soil layer. Detailed investigation on all stratified soil formation at different depth shall be performed in order to obtain sufficient data for numerical analysis.

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.

APPENDIX –A

Pile Load Test Data

Pile load test data for pile no. 1(K.K. project)

Project K.K. mixed use building project

Pile Load Test No. 1 Page:

Pile No. 1 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	0	10	00		8000	8000	8000	8000	0		
273	25	22	-	10	00		7961	7946	7928	7949	0.540	-	
273	25	22	-		05		7960	7945	7928	7948	0.548	0.007	
273	25	22	-		10		7960	7945	7928	7948	0.548	0.000	
273	25	22	-		20		7959	7943	7928	7948	0.555	0.008	
273	25	22	-		30		7958	7942	7928	7946	0.565	0.010	
273	25	22	-		60		7957	7941	7926	7944	0.580	0.015	
545	50	43	-	11	00		7935	7919	7901	7919	0.815	-	
545	50	43	-		05		7933	7915	7896	7917	0.848	0.033	
545	50	43	-		10		7930	7913	7892	7915	0.875	0.028	
545	50	43	-		20		7928	7912	7891	7913	0.890	0.015	
545	50	43	-		30		7927	7911	7890	7913	0.898	0.007	
545	50	43	-		60		7925	7909	7888	7910	0.920	0.023	
818	75	65	-	12	00		7909	7892	7872	7892	1.088	-	
818	75	65	-		05		7906	7889	7868	7890	1.118	0.030	
818	75	65	-		10		7906	7889	7868	7890	1.118	0.000	
818	75	65	-		20		7895	7876	7856	7876	1.243	0.125	
818	75	65	-		30		7893	7873	7854	7874	1.265	0.023	
818	75	65	-		60		7892	7871	7852	7871	1.285	0.020	
1250	100	100	-	13	00		7843	7818	7802	7819	1.795	-	
1250	100	100	-		05		7843	7815	7801	7818	1.808	0.013	
1250	100	100	-		10		7841	7813	7800	7817	1.823	0.015	
1250	100	100	-		20		7841	7812	7799	7817	1.828	0.005	
1250	100	100	-		30		7841	7812	7799	7817	1.828	0.000	
1250	100	100	-		60		7841	7812	7799	7817	1.828	0.000	
818	50	43	-	14	00		7876	7857	7836	7863	1.420	-	
818	50	43	-		05		7876	7857	7836	7863	1.420	0.000	
818	50	43	-		10		7877	7858	7836	7863	1.415	-0.005	
818	50	43	-		20		7877	7858	7836	7863	1.415	0.000	
818	50	43	-		30		7879	7861	7848	7869	1.358	-0.058	
818	50	43	-		60		7883	7865	7849	7869	1.335	-0.023	
0	0	0	-	15	00		7959	7915	7910	7907	0.773	-	
0	0	0	-		05		7960	7917	7909	7907	0.768	-0.005	
0	0	0	-		10		7961	7920	7909	7908	0.755	-0.013	
0	0	0	-		20		7964	7922	7910	7910	0.735	-0.020	
0	0	0	-		30		7965	7924	7911	7911	0.723	-0.013	
0	0	0	-		60		7984	7947	7925	7929	0.538	-0.185	
625	50	50	-	16	00		7883	7829	7893	7938	1.143	-	
625	50	50	-		05		7882	7828	7893	7937	1.150	0.007	
625	50	50	-		10		7882	7827	7893	7936	1.155	0.005	
625	50	50	-		20		7881	7825	7892	7934	1.170	0.015	
625	50	50	-		30		7880	7825	7892	7934	1.173	0.003	
625	50	50	-		60		7879	7824	7891	7933	1.183	0.010	
937.5	75	75	-	17	00		7857	7802	7866	7908	1.418	-	
937.5	75	75	-		05		7857	7801	7866	7907	1.423	0.005	
937.5	75	75	-		10		7857	7796	7865	7904	1.445	0.023	
937.5	75	75	-		20		7856	7795	7864	7902	1.458	0.013	
937.5	75	75	-		30		7855	7793	7863	7902	1.468	0.010	
937.5	75	75	-		60		7853	7792	7862	7900	1.483	0.015	
1250	100	100	-	18	00		7810	7753	7821	7861	1.888	-	
1250	100	100	-		05		7810	7752	7820	7861	1.893	0.005	
1250	100	100	-		10		7810	7752	7820	7861	1.893	0.000	
1250	100	100	-		20		7811	7751	7819	7860	1.898	0.005	
1250	100	100	-		30		7811	7751	7819	7860	1.898	0.000	
1250	100	100	-		60		7812	7750	7818	7859	1.903	0.005	

1562.5	125	125	-	19	00		7811	7748	7814	7856	1.928	-	
1562.5	125	125	-		05		7811	7748	7813	7856	1.930	0.002	
1562.5	125	125	-		10		7811	7748	7813	7855	1.933	0.003	
1562.5	125	125	-		20		7810	7747	7812	7854	1.943	0.010	
1562.5	125	125	-		30		7810	7747	7811	7853	1.948	0.005	
1562.5	125	125	-		60		7810	7747	7810	7853	1.950	0.002	
1875	150	150	-	20	00		7808	7745	7806	7850	1.978	-	
1875	150	150	-		05		7805	7744	7805	7849	1.993	0.015	
1875	150	150	-		10		7803	7742	7804	7848	2.008	0.015	
1875	150	150	-		20		7800	7738	7801	7847	2.035	0.028	
1875	150	150	-		30		7794	7732	7796	7842	2.090	0.055	
1875	150	150	-		60		7781	7723	7780	7832	2.210	0.120	
2500	200	199	-	21	00		7744	7665	7742	7668	2.953	-	
2500	200	199	-		05		7744	7665	7742	7668	2.953	0.000	
2500	200	199	-		10		7743	7665	7742	7667	2.958	0.005	
2500	200	199	-		20		7741	7665	7741	7664	2.973	0.015	
2500	200	199	-		30		7737	7664	7738	7658	3.008	0.035	
2500	200	199	-	22	00		7727	7663	7735	7656	3.048	0.040	
2500	200	199	-	23	00		7726	7661	7732	7654	3.068	0.020	
2500	200	199	-	00	00		7725	7659	7731	7651	3.085	0.018	
2500	200	199	-	01	00		7725	7658	7729	7650	3.095	0.010	
2500	200	199	-	02	00		7725	7656	7724	7645	3.125	0.030	
2500	200	199	-	03	00		7725	7655	7723	7643	3.135	0.010	
2500	200	199	-	04	00		7725	7654	7721	7642	3.145	0.010	
2500	200	199	-	05	00		7724	7653	7720	7641	3.155	0.010	
2500	200	199	-	06	00		7724	7651	7720	7638	3.168	0.013	
2500	200	199	-	07	00		7724	7651	7720	7637	3.170	0.002	
2500	200	199	-	08	00		7723	7650	7720	7637	3.175	0.005	
2500	200	199	-	09	00		7720	7639	7719	7635	3.218	0.043	
2500	200	199	-	10	00		7712	7626	7707	7631	3.310	0.093	
1875	150	150	-		05		7731	7646	7731	7741	2.878	-	
1875	150	150	-		10		7732	7649	7732	7745	2.855	-0.023	
1875	150	150	-		20		7735	7653	7734	7748	2.825	-0.030	
1875	150	150	-		30		7736	7655	7735	7752	2.805	-0.020	
1875	150	150	-		60		7737	7660	7738	7766	2.748	-0.058	
1250	100	100	-	11	00		7766	7693	7771	7802	2.420	-	
1250	100	100	-		05		7766	7693	7771	7802	2.420	0.000	
1250	100	100	-		10		7766	7694	7772	7802	2.415	-0.005	
1250	100	100	-		20		7766	7694	7772	7802	2.415	0.000	
1250	100	100	-		30		7766	7694	7772	7802	2.415	0.000	
1250	100	100	-		60		7767	7697	7774	7805	2.393	-0.023	
625	50	50	-	12	00		7798	7731	7819	7844	2.020	-	
625	50	50	-		05		7799	7732	7819	7845	2.013	-0.007	
625	50	50	-		10		7799	7732	7819	7845	2.013	0.000	
625	50	50	-		20		7799	7732	7820	7846	2.008	-0.005	
625	50	50	-		30		7799	7732	7820	7846	2.008	0.000	
625	50	50	-		60		7799	7733	7821	7846	2.003	-0.005	
0	0	0	-	13	00		7900	7895	7947	7943	0.788	-	
0	0	0	-		05		7901	7897	7945	7943	0.785	-0.002	
0	0	0	-		10		7901	7898	7945	7943	0.783	-0.003	
0	0	0	-		20		7902	7900	7944	7943	0.778	-0.005	
0	0	0	-		30		7903	7901	7944	7944	0.770	-0.007	
0	0	0	-	14	00		7906	7906	7943	7945	0.750	-0.020	
0	0	0	-	15	00		7909	7912	7947	7951	0.703	-0.048	
0	0	0	-	16	00		7927	7931	7962	7968	0.530	-0.173	
0	0	0	-	17	00		7940	7945	7972	7980	0.408	-0.123	
0	0	0	-	18	00		7948	7954	7979	7988	0.328	-0.080	
0	0	0	-	19	00		7954	7960	7984	7994	0.270	-0.058	
0	0	0	-	20	00		7959	7962	7988	7999	0.230	-0.040	
0	0	0	-	21	00		7962	7970	7990	8002	0.190	-0.040	
0	0	0	-	22	00		7965	7973	7993	8004	0.163	-0.028	

Pile load test data for pile no. 2 (K.K. project)

Project K.K. mixed use building project

Pile Load Test No. 2 Page: ____________
2
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.	^0C	mm	mm	mm	mm	mm	mm	
0	0	0	0	7	00		8000	8000	8000	8000	0		
875	25	70	-	7	00		7970	7880	7846	7972	0.830	-	
875	25	70	-		05		7969	7878	7845	7972	0.840	0.010	
875	25	70	-		10		7969	7878	7844	7973	0.840	0.000	
875	25	70	-		20		7971	7877	7844	7973	0.838	-0.002	
875	25	70	-		30		7966	7874	7840	7964	0.890	0.053	
875	25	70	-		60		7955	7861	7832	7953	0.998	0.108	
1750	50	140	-	8	00		7919	7828	7798	7922	1.333	-	
1750	50	140	-		05		7915	7822	7798	7919	1.365	0.033	
1750	50	140	-		10		7910	7818	7790	7912	1.425	0.060	
1750	50	140	-		20		7907	7811	7788	7911	1.458	0.033	
1750	50	140	-		30		7902	7805	7777	7906	1.525	0.067	
1750	50	140	-		60		7894	7795	7768	7897	1.615	0.090	
2625	75	209	-	9	00		7865	7759	7729	7867	1.950	-	
2625	75	209	-		05		7863	7752	7728	7865	1.980	0.030	
2625	75	209	-		10		7862	7750	7726	7863	1.998	0.018	
2625	75	209	-		20		7859	7745	7721	7861	2.035	0.038	
2625	75	209	-		30		7856	7741	7718	7857	2.070	0.035	
2625	75	209	-		60		7847	7729	7707	7849	2.170	0.100	
3500	100	279	-	10	00		7811	7690	7671	7821	2.518	-	
3500	100	279	-		05		7815	7687	7668	7820	2.525	0.007	
3500	100	279	-		10		7813	7684	7665	7818	2.550	0.025	
3500	100	279	-		20		7811	7679	7661	7815	2.585	0.035	
3500	100	279	-		30		7810	7677	7659	7814	2.600	0.015	
3500	100	279	-		60		7802	7668	7650	7802	2.695	0.095	
1750	50	140	-	11	00		7839	7709	7679	7831	2.355	-	
1750	50	140	-		05		7840	7710	7680	7832	2.345	-0.010	
1750	50	140	-		10		7840	7711	7680	7832	2.343	-0.003	
1750	50	140	-		20		7840	7710	7680	7831	2.348	0.005	
1750	50	140	-		30		7838	7709	7679	7830	2.360	0.012	
1750	50	140	-		60		7836	7707	7676	7827	2.385	0.025	
0	0	0	-	12	00		7906	7865	7859	7894	1.190	-	
0	0	0	-		05		7907	7868	7861	7894	1.175	-0.015	
0	0	0	-		10		7907	7868	7861	7894	1.175	0.000	
0	0	0	-		20		7907	7870	7863	7894	1.165	-0.010	
0	0	0	-		30		7907	7871	7864	7894	1.160	-0.005	
0	0	0	-		60		7907	7872	7864	7893	1.160	0.000	
1750	50	140	-	13	00		7837	7715	7683	7826	2.348	-	
1750	50	140	-		05		7837	7715	7683	7826	2.348	0.000	
1750	50	140	-		10		7837	7715	7683	7826	2.348	0.000	
1750	50	140	-		20		7836	7714	7682	7826	2.355	0.007	
1750	50	140	-		30		7836	7713	7682	7826	2.358	0.002	
1750	50	140	-		60		7835	7712	7681	7826	2.365	0.008	
2625	75	209	-	14	00		7812	7687	7662	7810	2.573	-	
2625	75	209	-		05		7813	7686	7662	7810	2.573	0.000	
2625	75	209	-		10		7813	7681	7662	7810	2.585	0.013	
2625	75	209	-		20		7811	7684	7660	7809	2.590	0.005	
2625	75	209	-		30		7812	7684	7661	7810	2.583	-0.007	
2625	75	209	-		60		7813	7685	7644	7811	2.618	0.035	
3500	100	279	-	15	00		7794	7662	7643	7799	2.755	-	
3500	100	279	-		05		7793	7660	7643	7798	2.765	0.010	
3500	100	279	-		10		7793	7660	7643	7798	2.765	0.000	

3500	100	279	-		20		7793	7660	7643	7798	2.765	0.000	
3500	100	279	-		30		7793	7659	7642	7798	2.770	0.005	
3500	100	279	-		60		7794	7659	7643	7800	2.760	-0.010	
4375	125	349	-	16	00		7773	7631	7620	7783	2.983	-	
4375	125	349	-		05		7773	7631	7620	7783	2.983	0.000	
4375	125	349	-		10		7773	7631	7620	7783	2.983	0.000	
4375	125	349	-		20		7774	7626	7617	7782	3.003	0.020	
4375	125	349	-		30		7772	7627	7617	7779	3.013	0.010	
4375	125	349	-		60		7774	7626	7616	7781	3.008	-0.005	
5250	150	418	-	17	00		7772	7596	7594	7768	3.175	-	
5250	150	418	-		05		7750	7593	7593	7765	3.248	0.073	
5250	150	418	-		10		7746	7589	7590	7763	3.280	0.032	
5250	150	418	-		20		7746	7589	7590	7763	3.280	0.000	
5250	150	418	-		30		7748	7587	7589	7766	3.275	-0.005	
5250	150	418	-	18	00		7757	7591	7594	7774	3.210	-0.065	
5250	150	418	-	19	00		7763	7597	7602	7782	3.140	-0.070	
5250	150	418	-	20	00		7767	7600	7605	7786	3.105	-0.035	
5250	150	418	-	21	00		7770	7602	7608	7790	3.075	-0.030	
5250	150	418	-	22	00		7773	7605	7611	7792	3.048	-0.028	
5250	150	418	-	23	00		7775	7606	7612	7794	3.033	-0.015	
5250	150	418	-	00	00		7778	7609	7614	7797	3.005	-0.028	
5250	150	418	-	01	00		7782	7613	7619	7801	2.963	-0.043	
5250	150	418	-	02	00		7783	7615	7621	7803	2.945	-0.018	
5250	150	418	-	03	00		7785	7617	7623	7805	2.925	-0.020	
5250	150	418	-	04	00		7789	7620	7626	7807	2.895	-0.030	
5250	150	418	-	05	00		7790	7622	7628	7809	2.878	-0.018	
3500	100	279	-	05	00		7822	7659	7652	7829	2.595	-	
3500	100	279	-		05		7823	7660	7652	7830	2.588	-0.008	
3500	100	279	-		10		7824	7661	7653	7831	2.578	-0.010	
3500	100	279	-		20		7825	7662	7654	7832	2.568	-0.010	
3500	100	279	-		30		7826	7663	7655	7833	2.558	-0.010	
3500	100	279	-		60		7827	7664	7656	7834	2.548	-0.010	
1750	50	140	-	6	00		7876	7721	7693	7865	2.113	-	
1750	50	140	-		05		7877	7722	7694	7866	2.103	-0.010	
1750	50	140	-		10		7877	7723	7695	7866	2.098	-0.005	
1750	50	140	-		20		7879	7725	7697	7867	2.080	-0.018	
1750	50	140	-		30		7879	7726	7697	7867	2.078	-0.002	
1750	50	140	-		60		7881	7727	7699	7869	2.060	-0.018	
0	0	0	-	7	00		7953	7849	7844	7929	1.063	-	
0	0	0	-		05		7954	7852	7844	7929	1.053	-0.010	
0	0	0	-		10		7955	7855	7848	7930	1.030	-0.023	
0	0	0	-		20		7958	7865	7852	7933	0.980	-0.050	
0	0	0	-		30		7958	7867	7852	7933	0.975	-0.005	
0	0	0	-	8	00		7961	7867	7854	7935	0.958	-0.018	
0	0	0	-	9	00		7966	7870	7855	7935	0.935	-0.023	
0	0	0	-	10	00		7964	7869	7857	7938	0.930	-0.005	
0	0	0	-	11	00		7967	7871	7857	7936	0.923	-0.008	
0	0	0	-	12	00		7968	7871	7856	7939	0.915	-0.007	
0	0	0	-	13	00		7969	7871	7858	7939	0.908	-0.008	
0	0	0	-	14	00		7971	7874	7860	7939	0.890	-0.018	
0	0	0	-	15	00		7971	7875	7861	7941	0.880	-0.010	
0	0	0	-	16	00		7969	7880	7864	7940	0.868	-0.013	
0	0	0		17	00		7973	7882	7867	7942	0.840	-0.028	
0	0	0		18	00		7979	7890	7875	7954	0.755	-0.085	
0	0	0		19	00		7982	7893	7877	7953	0.738	-0.018	

Pile load test data for pile no. 1(Dire Dawa. project)

Project TAIWAN BRIDGE
Pile Load Test No. __01_ Page: __01-04_
Pile No. P1/WP-03 Date: 26-27 APRIL 2014

Load	**work load**	**Pressure**	**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	14	00		8000	8000	8000	8000	0	0	
393	25%	33	14	00		7931	7911	7989	8000	0.4225	0.4225	
393	25%	33	14	05		7942	7924	7999	8011	0.31	-0.1125	
393	25%	33	14	10		7947	7928	8002	8015	0.27	-0.04	
393	25%	33	14	20		7946	7929	8004	8017	0.26	-0.01	
393	25%	33	14	30		7953	7937	8010	8024	0.19	-0.07	
393	25%	33	14	60		7956	7938	8012	8024	0.175	-0.015	
785	50%	65	15	00		7935	7916	7969	7968	0.53	0.355	
785	50%	65	15	05		7940	7916	7968	7964	0.53	0	
785	50%	65	15	10		7940	7915	7968	7962	0.5375	0.0075	
785	50%	65	15	20		7941	7915	7967	7960	0.5425	0.005	
785	50%	65	15	30		7941	7914	7966	7959	0.55	0.0075	
785	50%	65	15	60		7941	7913	7966	7957	0.5575	0.0075	
1178	75%	97	16	00		7939	7865	7916	7844	1.09	0.5325	
1178	75%	97	16	05		7941	7862	7914	7836	1.1175	0.0275	
1178	75%	97	16	10		7942	7861	7913	7833	1.1275	0.01	
1178	75%	97	16	20		7944	7861	7912	7831	1.13	0.0025	
1178	75%	97	16	30		7945	7860	7910	7826	1.1475	0.0175	
1178	75%	97	16	60		7943	7857	7908	7823	1.1725	0.025	
1570	100%	129	17	00		7909	7805	7859	7735	1.73	0.5575	
1570	100%	129	17	05		7908	7793	7853	7722	1.81	0.08	
1570	100%	129	17	10		7910	7793	7854	7719	1.81	0	
1570	100%	129	17	20		7910	7789	7849	7713	1.8475	0.0375	
1570	100%	129	17	30		7911	7790	7849	7712	1.845	-0.0025	
1570	100%	129	17	60		7915	7793	7851	7713	**1.82**	-0.025	
785	50%	65	18	00		7975	7820	7902	7760	1.3575	-0.4625	
785	50%	65	18	05		7978	7823	7903	7760	1.34	-0.0175	
785	50%	65	18	10		7980	7825	7905	7762	1.32	-0.02	
785	50%	65	18	20		7981	7825	7905	7764	1.3125	-0.0075	
785	50%	65	18	30		7982	7826	7908	7765	1.2975	-0.015	
785	50%	65	18	60		7986	7830	7910	7770	1.26	-0.0375	
0	0%	0	19	00		8056	7935	8014	7874	0.3025	-0.9575	
0	0%	0	19	05		8055	7927	8024	7879	0.2875	-0.015	
0	0%	0	19	10		8056	7927	8025	7881	0.2775	-0.01	

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0	0%	0	19	20		8056	7929	8025	7882	0.27	-0.0075	
0	0%	0	19	30		8056	7932	8025	7882	0.2625	-0.0075	
0	0%	0	19	60		8056	7935	8024	7883	0.255	-0.0075	
785	50%	65	20	00		7992	7864	7921	7887	0.84	0.84	
785	50%	65	20	05		7990	7863	7920	7886	0.8525	0.0125	
785	50%	65	20	10		7990	7863	7920	7886	0.8525	0	
785	50%	65	20	20		7990	7863	7920	7886	0.8525	0	
785	50%	65	20	30		7990	7863	7919	7885	0.8575	0.005	
785	50%	65	20	60		7989	7861	7917	7884	0.8725	0.015	
1178	75%	97	21	00		7951	7840	7878	7750	1.4525	0.58	
1178	75%	97	21	05		7949	7839	7876	7748	1.47	0.0175	
1178	75%	97	21	10		7949	7839	7875	7747	1.475	0.005	
1178	75%	97	21	20		7948	7839	7874	7746	1.4825	0.0075	
1178	75%	97	21	30		7948	7839	7873	7746	1.485	0.0025	
1178	75%	97	21	60		7949	7840	7872	7745	1.485	0	
1570	100%	129	22	00		7903	7813	7830	7713	1.8525	0.3675	
1570	100%	129	22	05		7902	7813	7826	7705	1.885	0.0325	
1570	100%	129	22	10		7901	7812	7825	7704	1.895	0.01	
1570	100%	129	22	20		7901	7812	7824	7703	1.9	0.005	
1570	100%	129	22	30		7901	7812	7822	7702	1.9075	0.0075	
1570	100%	129	22	60		7900	7811	7819	7699	1.9275	0.02	
1963	125%	162	23	00		7841	7765	7773	7655	2.415	0.4875	
1963	125%	162	23	05		7836	7759	7772	7653	2.45	0.035	
1963	125%	162	23	10		7836	7757	7770	7650	2.4675	0.0175	
1963	125%	162	23	20		7835	7757	7769	7648	2.4775	0.01	
1963	125%	162	23	30		7835	7756	7768	7647	2.485	0.0075	
1963	125%	162	23	60		7830	7749	7762	7640	2.5475	0.0625	
2355	150%	194	0	00		7760	7682	7705	7583	3.175	0.6275	
2355	150%	194	0	05		7758	7675	7704	7579	3.21	0.035	
2355	150%	194	0	10		7756	7672	7701	7576	3.2375	0.0275	
2355	150%	194	0	20		7755	7669	7699	7572	3.2625	0.025	
2355	150%	194	0	30		7753	7666	7696	7569	3.29	0.0275	
2355	150%	194	0	60		7751	7663	7692	7565	3.3225	0.0325	
2748	175%	226	1	00		7699	7512	7637	7506	4.1150	4.1150	
2748	175%	226	1	05		7699	7506	7635	7501	4.1475	0.0325	
2748	175%	226	1	10		7698	7501	7633	7496	4.1800	0.0325	
2748	175%	226	1	20		7698	7497	7630	7490	4.2125	0.0325	
2748	175%	226	1	30		7697	7494	7629	7488	4.2300	0.0175	
2748	175%	226	1	60		7696	7489	7625	7480	4.2750	0.0450	
3140	200%	258	2	00		7632	7510	7553	7395	**4.775**	0.5000	
3140	200%	258	2	05		7628	7500	7547	7383	4.855	0.0800	
3140	200%	258	2	10		7627	7498	7545	7380	4.875	0.0200	

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3140	200%	258	2	20		7626	7498	7544	7378	4.885	0.010
3140	200%	258	2	30		7625	7493	7542	7375	4.913	0.027
3140	200%	258	3	00		7609	7472	7520	7352	5.118	0.205
3140	200%	258	4	00		7606	7461	7515	7344	5.185	0.067
3140	200%	258	5	00		7605	7464	7512	7341	5.195	0.010
3140	200%	258	6	00		7600	7460	7505	7335	5.250	0.055
3140	200%	258	7	00		7589	7448	7491	7322	5.375	0.125
3140	200%	258	8	00		7574	7433	7476	7307	5.525	0.150
3140	200%	258	9	00		7566	7425	7468	7299	5.605	0.080
3140	200%	258	10	00		7559	7419	7460	7291	5.678	0.072
3140	200%	258	11	00		7549	7401	7450	7281	5.798	0.120
3140	200%	258	12	00		7533	7391	7435	7264	5.943	0.145
3140	200%	258	13	00		7534	7394	7433	7262	**5.943**	0.000
3140	200%	258	14	00		7534	7395	7434	7266	5.928	-0.015
2355	150%	194	14	00		7593	7439	7496	7321	5.378	-0.550
2355	150%	194	14	05		7596	7440	7497	7322	5.363	-0.015
2355	150%	194	14	10		7599	7444	7501	7325	5.328	-0.035
2355	150%	194	14	20		7605	7450	7506	7331	5.270	-0.058
2355	150%	194	14	30		7614	7469	7520	7368	5.073	-0.198
2355	150%	194	14	60		7636	7479	7537	7360	4.970	-0.103
1570	100%	129	15	00		7712	7526	7626	7425	4.278	-0.693
1570	100%	129	15	05		7713	7526	7614	7425	4.305	0.027
1570	100%	129	15	10		7713	7526	7614	7425	4.305	0.000
1570	100%	129	15	20		7711	7524	7613	7424	4.320	0.015
1570	100%	129	15	30		7710	7523	7612	7423	4.330	0.010
1570	100%	129	15	60		7709	7524	7611	7423	4.333	0.002
785	50%	65	16	00		7811	7590	7708	7501	3.475	3.475
785	50%	65	16	05		7812	7591	7709	7502	3.465	-0.010
785	50%	65	16	10		7813	7593	7710	7503	3.453	-0.012
785	50%	65	16	20		7816	7595	7713	7506	3.425	-0.028
785	50%	65	16	30		7824	7604	7721	7514	3.343	-0.083
785	50%	65	16	60		7827	7607	7724	7516	3.315	-0.027
0	0%	0	17	00		7985	7744	7880	7634	**1.893**	-1.423
0	0%	0	17	05		7985	7742	7887	7640	1.865	-0.028
0	0%	0	17	10		7982	7743	7890	7644	1.853	-0.013
0	0%	0	17	20		7990	7745	7895	7648	1.805	-0.048
0	0%	0	17	30		7988	7745	7897	7650	1.800	-0.005
0	0%	0	18	00		7989	7745	7902	7655	1.773	-0.028
0	0%	0	19	00		7989	7745	7906	7658	1.755	-0.018
0	0%	0	20	00		7988	7744	7909	7661	1.745	-0.010
0	0%	0	21	00		7998	7753	7919	7670	1.650	-0.095
0	0%	0	22	00		7999	7754	7918	7670	1.648	-0.002
0	0%	0	23	00		7998	7753	7920	7670	1.648	0.000
0	0%	0	0	00		7998	7748	7917	7666	1.678	0.030
0	0%	0	1	00						80.000	78.323
0	0%	0	2	00						**80.000**	0.000
0	0%	0	3	00						80.000	0.000
0	0%	0	4	00						80.000	0.000
0	0%	0	5	00						**80.000**	0.000

Pile load test data for pile no. 2(Dire Dawa. project)

Project TAIWAN BRIDGE
Pile Load Test No. _02_ Page: __01-04_
Pile No. A1/WP-29 Date: 3-04 APRIL 201

Load	**work load**	**Pressure**	**Load Cell**	**time**		**temp.**	**reading dial gauges**				**settlement**	
				ctual 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		16	00		8000	8000	8000	8000	0	0
440	25%	37		16	00		7888	7971	7927	7896	0.795	0.795
440	25%	37		16	05		7886	7972	7925	7892	0.8125	0.0175
440	25%	37		16	10		7886	7972	7922	7889	0.8275	0.015
440	25%	37		16	20		7886	7973	7922	7888	0.8275	0
440	25%	37		16	30		7888	7975	7922	7888	0.8175	-0.01
440	25%	37		16	60		7888	7976	7920	7886	0.825	0.0075
880	50%	73		17	00		7780	7927	7800	7710	1.9575	1.1325
880	50%	73		17	05		7772	7927	7799	7705	1.9925	0.035
880	50%	73		17	10		7770	7927	7799	7702	2.005	0.0125
880	50%	73		17	20		7768	7927	7799	7699	2.0175	0.0125
880	50%	73		17	30		7767	7927	7799	7697	2.025	0.0075
880	50%	73		17	60		7766	7929	7799	7695	2.0275	0.0025
1320	75%	109		18	00		7664	7813	7696	7624	3.0075	0.98
1320	75%	109		18	05		7651	7803	7689	7615	3.105	0.0975
1320	75%	109		18	10		7653	7803	7687	7609	3.12	0.015
1320	75%	109		18	20		7642	7802	7686	7604	3.165	0.045
1320	75%	109		18	30		7639	7800	7686	7602	3.1825	0.0175
1320	75%	109		18	60		7635	7797	7680	7595	3.2325	0.05
1760	100%	145		19	00		7514	7720	7547	7527	4.23	0.9975
1760	100%	145		19	05		7509	7715	7540	7519	4.2925	0.0625
1760	100%	145		19	10		7507	7715	7540	7516	4.305	0.0125
1760	100%	145		19	20		7505	7713	7538	7512	4.33	0.025
1760	100%	145		19	30		7504	7712	7536	7508	4.35	0.02
1760	100%	145		19	60		7504	7711	7534	7504	**4.3675**	0.0175
880	50%	73		20	00		7663	7767	7620	7563	3.4675	-0.9
880	50%	73		20	05		7663	7767	7619	7564	3.4675	0
880	50%	73		20	10		7664	7767	7619	7564	3.465	-0.0025
880	50%	73		20	20		7664	7767	7620	7564	3.4625	-0.0025
880	50%	73		20	30		7664	7765	7620	7568	3.4575	-0.005
880	50%	73		20	60		7665	7768	7621	7565	3.4525	-0.005
0	0%	0		21	00		7876	7899	7764	7788	1.6825	-1.77
0	0%	0		21	05		7881	7902	7790	7765	1.655	-0.0275
0	0%	0		21	10		7882	7903	7790	7766	1.6475	-0.0075

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0	0%	0		21	20		7882	7905	7792	7765	1.64	-0.0075
0	0%	0		21	30		7883	7905	7792	7767	1.6325	-0.0075
0	0%	0		21	60		7883	7906	7794	7767	1.625	-0.0075
880	50%	73		22	00		7651	7877	7662	7577	3.0825	3.0825
880	50%	73		22	05		7650	7876	7662	7576	3.09	0.0075
880	50%	73		22	10		7650	7876	7662	7575	3.0925	0.0025
880	50%	73		22	20		7650	7876	7662	7575	3.0925	0
880	50%	73		22	30		7650	7877	7662	7575	3.09	-0.0025
880	50%	73		22	60		7651	7877	7662	7575	3.0875	-0.0025
1320	75%	109		23	00		7621	7742	7602	7538	3.7425	0.655
1320	75%	109		23	05		7620	7740	7602	7536	3.755	0.0125
1320	75%	109		23	10		7620	7740	7602	7536	3.755	0
1320	75%	109		23	20		7620	7740	7601	7535	3.76	0.005
1320	75%	109		23	30		7620	7740	7601	7535	3.76	0
1320	75%	109		23	60		7621	7740	7600	7536	3.7575	-0.0025
1760	100%	145		0	00		7587	7696	7526	7485	4.265	0.5075
1760	100%	145		0	05		7583	7693	7525	7483	4.29	0.025
1760	100%	145		0	10		7582	7692	7524	7481	4.3025	0.0125
1760	100%	145		0	20		7581	7691	7524	7480	4.31	0.0075
1760	100%	145		0	30		7580	7690	7523	7479	4.32	0.01
1760	100%	145		0	60		7579	7688	7522	7477	4.335	0.015
2200	125%	181		1	00		7554	7637	7411	7402	4.99	0.655
2200	125%	181		1	05		7549	7633	7410	7400	5.02	0.03
2200	125%	181		1	10		7546	7632	7408	7396	5.045	0.025
2200	125%	181		1	20		7544	7630	7405	7390	5.0775	0.0325
2200	125%	181		1	30		7542	7628	7403	7386	5.1025	0.025
2200	125%	181		1	60		7539	7626	7400	7381	5.135	0.0325
2640	150%	217		2	00		7507	7569	7275	7292	5.8925	0.7575
2640	150%	217		2	05		7501	7565	7268	7281	5.9625	0.07
2640	150%	217		2	10		7497	7563	7264	7274	6.005	0.0425
2640	150%	217		2	20		7495	7561	7261	7270	6.0325	0.0275
2640	150%	217		2	30		7494	7560	7260	7268	6.045	0.0125
2640	150%	217		2	60		7491	7557	7255	7262	6.0875	0.0425
3080	175%	253		3	00		7519	7534	7155	7215	6.4425	6.4425
3080	175%	253		3	05		7519	7534	7149	7212	6.4650	0.0225
3080	175%	253		3	10		7519	7533	7145	7209	6.4850	0.0200
3080	175%	253		3	20		7519	7532	7141	7206	6.5050	0.0200
3080	175%	253		3	30		7519	7532	7138	7204	6.5175	0.0125
3080	175%	253		3	60		7518	7531	7132	7200	6.5475	0.0300
3520	200%	289		4	00		7604	7527	7028	7186	**6.6375**	0.0900
3520	200%	289		4	05		7609	7525	7019	7184	6.6575	0.0200
3520	200%	289		4	10		7610	7524	7017	7184	6.6625	0.0050

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3520	200%	289		4	20		7612	7524	7014	7184	6.665	0.0025
3520	200%	289		4	30		7614	7523	7012	7184	6.6675	0.0025
3520	200%	289		5	00		7619	7522	7012	7184	6.6575	-0.0100
3520	200%	289		6	00		7621	7521	7005	7179	6.6850	0.0275
3520	200%	289		7	00		7620	7520	7000	7173	6.7175	0.0325
3520	200%	289		8	00		7620	7517	6993	7169	6.7525	0.0350
3520	200%	289		9	00		7620	7513	6986	7162	6.7975	0.0450
3520	200%	289		10	00		7610	7510	6982	7157	6.8525	0.0550
3520	200%	289		11	00		7608	7507	6978	7153	6.8850	0.0325
3520	200%	289		12	00		7606	7506	6977	7152	6.8975	0.0125
3520	200%	289		13	00		7607	7506	6975	7151	6.9025	0.0050
3520	200%	289		14	00		7608	7506	6975	7151	6.9000	-0.0025
3520	200%	289		15	00		7608	7506	6974	7151	**6.9025**	0.0025
3520	200%	289		16	00		7610	7506	6974	7151	6.8975	-0.0050
2640	150%	217		16	00		7605	7504	6982	7149	6.9000	0.0025
2640	150%	217		16	05		7605	7504	6982	7149	6.9000	0.0000
2640	150%	217		16	10		7605	7504	6982	7149	6.9000	0.0000
2640	150%	217		16	20		7605	7504	6982	7149	6.9000	0.0000
2640	150%	217		16	30		7605	7504	6982	7149	6.9000	0.0000
2640	150%	217		16	60		7606	7504	6983	7149	6.8950	-0.0050
1760	100%	145		17	00		7589	7501	7008	7149	6.8825	-0.0125
1760	100%	145		17	05		7590	7501	7008	7150	6.8775	-0.0050
1760	100%	145		17	10		7590	7501	7009	7150	6.8750	-0.0025
1760	100%	145		17	20		7590	7501	7010	7150	6.8725	-0.0025
1760	100%	145		17	30		7590	7501	7010	7150	6.8725	0.0000
1760	100%	145		17	60		7591	7501	7010	7150	6.8700	-0.0025
880	50%	73		18	00		7661	7581	7146	7256	5.8900	5.8900
880	50%	73		18	05		7662	7581	7148	7258	5.8775	-0.0125
880	50%	73		18	10		7663	7582	7149	7260	5.8650	-0.0125
880	50%	73		18	20		7664	7583	7151	7261	5.8525	-0.0125
880	50%	73		18	30		7665	7583	7152	7263	5.8425	-0.0100
880	50%	73		18	60		7666	7585	7155	7265	5.8225	-0.0200
0	0%	0		19	00		7742	7853	7571	7465	**3.4225**	-2.4000
0	0%	0		19	05		7742	7856	7587	7465	3.3750	-0.0475
0	0%	0		19	10		7742	7859	7591	7467	3.3525	-0.0225
0	0%	0		19	20		7744	7868	7602	7470	3.2900	-0.0625
0	0%	0		19	30		7745	7872	7606	7472	3.2625	-0.0275
0	0%	0		20	00		7747	7880	7616	7476	3.2025	-0.0600
0	0%	0		21	00		7748	7887	7626	7480	3.1475	-0.0550
0	0%	0		22	00		7747	7889	7629	7480	3.1375	-0.0100
0	0%	0		23	00		7749	7893	7633	7482	3.1075	-0.0300
0	0%	0		0	00		7745	7891	7632	7480	3.1300	0.0225
0	0%	0		0	00						80.0000	76.8700
0	0%	0		2	00						80.0000	0.0000
0	0%	0		3	00						80.0000	0.0000
0	0%	0		4	00						**80.0000**	0.0000
0	0%	0		5	00						80.0000	0.0000
0	0%	0		6	00						80.0000	0.0000
0	0%	0		7	00						**80.0000**	0.0000

Pile load test data for pile no. 1 on GITPBH 2(Gidabo Dam Project)

Project	GIDABO DAM				
Pile Load Test No.	_01_	Page:		Pile Dim.	600mm
Pile No.	TP-01	Date:	16-Nov-13		

Load	**work load**	**Pressure**	**Load Cell**	**time**		**temp.**	**reading dial gauges**				**settlement**	
				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		8000	8000	8000	8000	0	0
293	10%	12		12	00		7918	7985	7983	8000	0.285	0.285
293				12	15		7915	7979	7979	7998	0.323	0.0375
586	20%	23.5		12	15		7835	7974	7969	7979	0.608	0.285
586				12	30		7833	7964	7961	7973	0.673	0.065
879	30%	35		12	30		7791	7945	7932	7952	0.95	0.2775
879				12	45		7801	7946	7931	7954	0.92	-0.03
1172	40%	47		12	45		7760	7923	7896	7919	1.255	0.335
1172				13	00		7754	7918	7888	7917	1.308	0.0525
1465	50%	58.5		13	00		7722	7898	7858	7892	1.575	0.2675
1465				13	15		7718	7896	7855	7890	1.603	0.0275
1758	60%	70		13	15		7700	7871	7817	7863	1.873	0.27
1758				13	30		7695	7865	7812	7861	1.918	0.045
2051	70%	82		13	30		7670	7834	7767	7828	2.253	0.335
2051				13	45		7661	7827	7758	7822	2.33	0.0775
2344	80%	93.5		13	45		7634	7793	7713	7790	2.675	0.345
2344				14	00		7623	7786	7706	7786	2.748	0.0725
2637	90%	105		14	00		7592	7752	7662	7753	3.103	0.355
2637				14	15		7571	7734	7642	7736	3.293	0.19
2929	100%	117		14	15		7539	7698	7597	7702	3.66	0.3675
2929				14	30		7526	7688	7585	7694	3.768	0.1075
2929				14	45		7519	7680	7576	7688	3.843	0.075
2929				15	00		7520	7681	7578	7691	3.825	-0.0175
2929				15	15		7520	7682	7579	7692	**3.818**	-0.0075
2197	75%	87.5		15	15		7515	7674	7610	7719	3.705	3.705
2197				15	30		7514	7676	7613	7724	3.683	-0.0225
1465	50%	58.5		15	30		7577	7739	7700	7791	2.983	-0.7
1465				15	45		7570	7735	7695	7786	3.035	0.0525
733	25%	29.5		15	45		7679	7824	7800	7862	2.088	-0.9475
733				16	00		7688	7832	7806	7867	2.018	-0.07
0	0%	0		16	00		7887	7908	7901	7920	0.96	-1.0575
0				16	15		7889	7914	7909	7926	0.905	-0.055

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0				16	30		7892	7919	7915	7931	0.858	-0.0475
733	25%	29.5		16	30		7761	7898	7872	7904	1.413	0.555
733				16	45		7758	7897	7871	7905	1.423	0.01
1465	50%	58.5		16	45		7666	7736	7781	7840	2.443	1.02
1465				17	00		7661	7733	7778	7839	2.473	0.03
2197	75%	87.5		17	00		7582	7756	7676	7764	3.055	0.5825
2197				17	15		7576	7753	7673	7762	3.09	0.035
2929	100%	117		17	15		7506	7675	7572	7688	3.898	0.8075
2929				17	30		7498	7667	7564	7683	3.97	0.0725
3222	110%	129		17	30		7473	7637	7525	7650	4.288	0.3175
3222				17	45		7464	7628	7517	7645	4.365	0.0775
3515	120%	140		17	45		7425	7583	7463	7599	4.825	0.46
3515				18	00		7415	7574	7453	7591	4.918	0.0925
3808	130%	152		18	00		7375	7525	7397	7545	5.395	0.4775
3808				18	15		7361	7514	7387	7535	5.508	0.1125
4101	140%	164		18	15		7318	7464	7329	7485	6.01	0.5025
4101				18	30		7308	7453	7318	7478	6.108	0.0975
4394	150%	175		18	30		7243	7378	7240	7410	6.823	0.715
4394				18	45		7223	7358	7220	7395	7.01	0.1875
4687	160%	187		18	45		7160	7285	7140	7322	7.733	0.7225
4687				19	00		7137	7260	7120	7307	7.94	7.94
4980	170%	199		19	00		7048	7165	7015	7205	8.918	0.9775
4980				19	15		7016	7130	6978	7184	9.23	0.3125
5273	180%	210		19	15		6952	7060	6899	7118	9.928	0.6975
5273				19	30		6945	7049	6885	7110	10.03	0.1
5565	190%	222		19	30		6920	7010	6832	7068	10.43	0.3975
5565				19	45		6909	6999	6816	7059	10.54	0.1175
5858	200%	234		19	45		6830	6925	6710	6970	11.41	0.87
5858				20	00		6821	6894	6682	6955	11.62	0.2075
5858				20	30		6825	6892	6679	6955	11.62	0.0025
5858				21	00		6825	6882	6654	6942	11.74	0.12
5858				22	00		6822	6880	6649	6942	11.77	0.025
5858				23	00		6828	6872	6630	6932	11.85	0.0775
5858				00	00		6824	6862	6610	6920	11.96	0.115
5858				01	00		6820	6854	6597	6910	12.05	0.0875
5858				02	00		6820	6850	6590	6906	12.09	0.0375
5858				03	00		6820	6849	6583	6906	12.11	0.02
5858				04	00		6820	6848	6583	6906	12.11	0.0025
5858				05	00		6821	6848	6583	6904	12.11	0.0025
5858				06	00		6822	6848	6582	6904	12.11	0
4394	150%	175		06	00		6827	6849	6623	6942	11.9	-0.2125
4394				06	15		6827	6849	6624	6942	11.9	-0.0025

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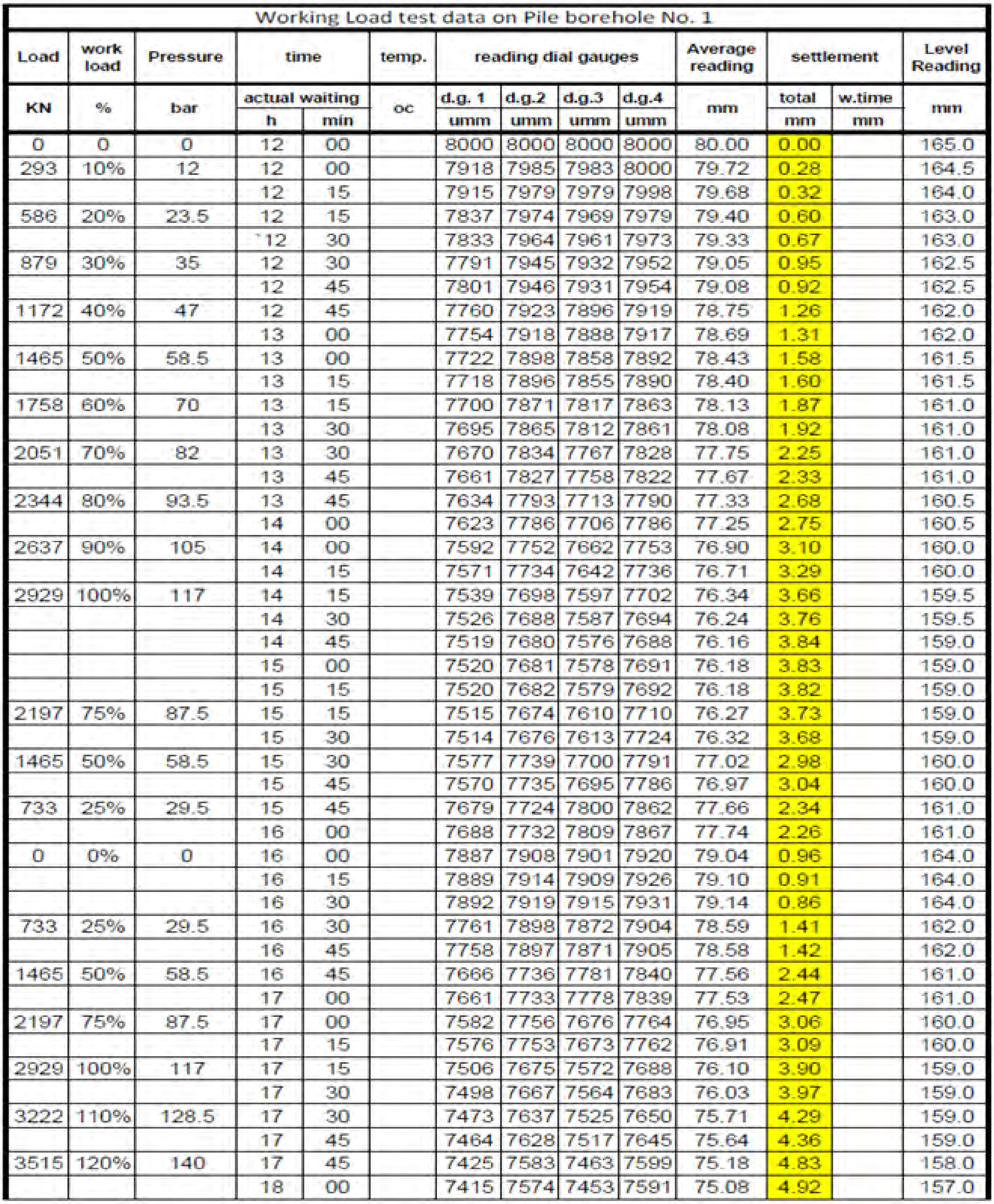
2929	100%	117		06	15		6837	6865	6728	7016	11.39	-0.51
2929				06	30		6845	6878	6732	7019	11.32	11.315
1465	50%	58.5		06	30		6929	7009	6904	7131	10.07	-1.2475
1465				06	45		6934	7015	6915	7138	9.995	-0.0725
0	0%	0		06	45		7368	7350	7290	7410	6.455	-3.54
0				07	00		7366	7370	7324	7422	6.295	-0.16
0				07	15		7497	7415	7383	7444	5.653	-0.6425
0				07	30		7500	7420	7390	7447	5.608	-0.045
0				07	45		7500	7419	7394	7450	5.593	-0.015
0				08	45		7490	7419	7396	7443	5.63	0.0375
0				09	45		7481	7416	7402	7426	5.688	0.0575
0				10	45		7146	7234	7155	7041	8.56	2.8725
0				11	45		7143	7233	7155	7039	8.575	0.015

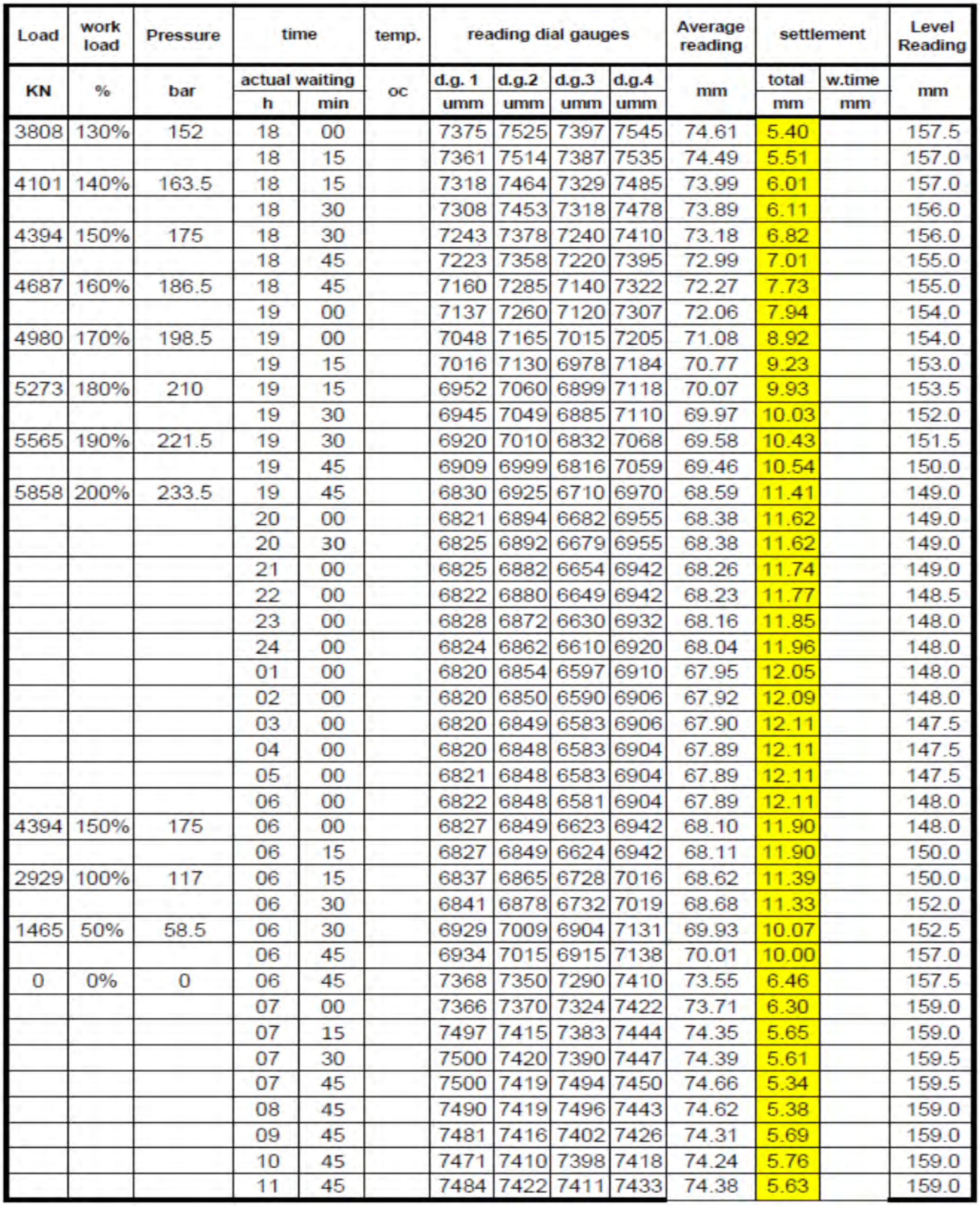
MIDROC Foundation WWDSE

Pile load test data for pile no.2 (Gidabo Dam Project)



Working Load test data on Pile borehole No. 1

Load	work load	Pressure	time		temp.	reading dial gauges				Average reading	settlement		Level Reading
KN	%	bar	actual waiting		oc	d.g. 1	d.g.2	d.g.3	d.g.4	mm	total	w.time	mm
			h	min		umm	umm	umm	umm		mm	mm	
0	0	0	12	00		8000	8000	8000	8000	80.00	0.00		165.0
293	10%	12	12	00		7918	7985	7983	8000	79.72	0.28		164.5
			12	15		7915	7979	7979	7998	79.68	0.32		164.0
586	20%	23.5	12	15		7837	7974	7969	7979	79.40	0.60		163.0
			12	30		7833	7964	7961	7973	79.33	0.67		163.0
879	30%	35	12	30		7791	7945	7932	7952	79.05	0.95		162.5
			12	45		7801	7946	7931	7954	79.08	0.92		162.5
1172	40%	47	12	45		7760	7923	7896	7919	78.75	1.26		162.0
			13	00		7754	7918	7888	7917	78.69	1.31		162.0
1465	50%	58.5	13	00		7722	7898	7858	7892	78.43	1.58		161.5
			13	15		7718	7896	7855	7890	78.40	1.60		161.5
1758	60%	70	13	15		7700	7871	7817	7863	78.13	1.87		161.0
			13	30		7695	7865	7812	7861	78.08	1.92		161.0
2051	70%	82	13	30		7670	7834	7767	7828	77.75	2.25		161.0
			13	45		7661	7827	7758	7822	77.67	2.33		161.0
2344	80%	93.5	13	45		7634	7793	7713	7790	77.33	2.68		160.5
			14	00		7623	7786	7706	7786	77.25	2.75		160.5
2637	90%	105	14	00		7592	7752	7662	7753	76.90	3.10		160.0
			14	15		7571	7734	7642	7736	76.71	3.29		160.0
2929	100%	117	14	15		7539	7698	7597	7702	76.34	3.66		159.5
			14	30		7526	7688	7587	7694	76.24	3.76		159.5
			14	45		7519	7680	7576	7688	76.16	3.84		159.0
			15	00		7520	7681	7578	7691	76.18	3.83		159.0
			15	15		7520	7682	7579	7692	76.18	3.82		159.0
2197	75%	87.5	15	15		7515	7674	7610	7710	76.27	3.73		159.0
			15	30		7514	7676	7613	7724	76.32	3.68		159.0
1465	50%	58.5	15	30		7577	7739	7700	7791	77.02	2.98		160.0
			15	45		7570	7735	7695	7786	76.97	3.04		160.0
733	25%	29.5	15	45		7679	7724	7800	7862	77.66	2.34		161.0
			16	00		7688	7732	7809	7867	77.74	2.26		161.0
0	0%	0	16	00		7887	7908	7901	7920	79.04	0.96		164.0
			16	15		7889	7914	7909	7926	79.10	0.91		164.0
			16	30		7892	7919	7915	7931	79.14	0.86		164.0
733	25%	29.5	16	30		7761	7898	7872	7904	78.59	1.41		162.0
			16	45		7758	7897	7871	7905	78.58	1.42		162.0
1465	50%	58.5	16	45		7666	7736	7781	7840	77.56	2.44		161.0
			17	00		7661	7733	7778	7839	77.53	2.47		161.0
2197	75%	87.5	17	00		7582	7756	7676	7764	76.95	3.06		160.0
			17	15		7576	7753	7673	7762	76.91	3.09		160.0
2929	100%	117	17	15		7506	7675	7572	7688	76.10	3.90		159.0
			17	30		7498	7667	7564	7683	76.03	3.97		159.0
3222	110%	128.5	17	30		7473	7637	7525	7650	75.71	4.29		159.0
			17	45		7464	7628	7517	7645	75.64	4.36		159.0
3515	120%	140	17	45		7425	7583	7463	7599	75.18	4.83		158.0
			18	00		7415	7574	7453	7591	75.08	4.92		157.0

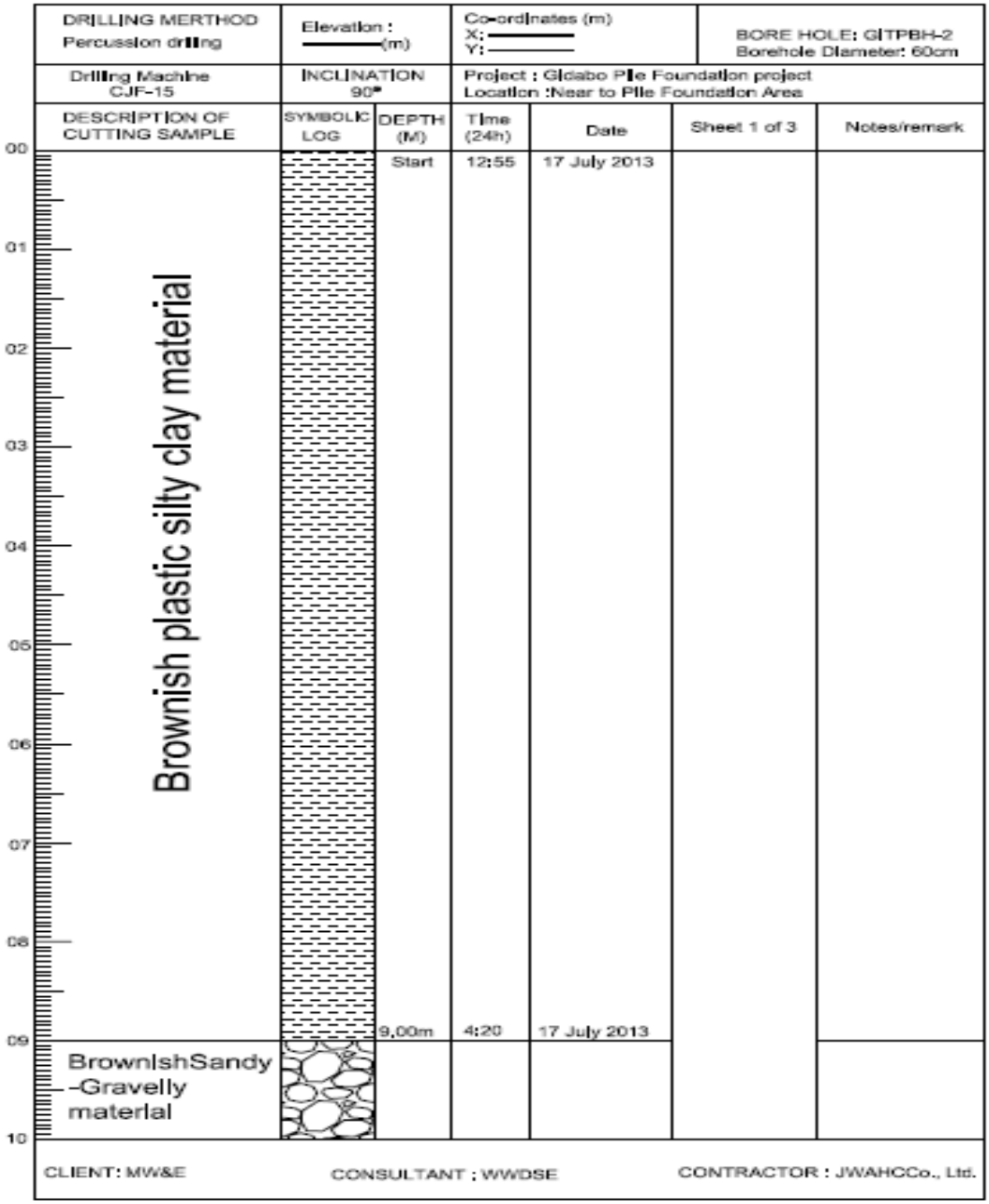


Load	work load	Pressure	time		temp.	reading dial gauges				Average reading	settlement		Level Reading
KN	%	bar	actual waiting		oc	d.g. 1	d.g.2	d.g.3	d.g.4	mm	total	w.time	mm
			h	min		umm	umm	umm	umm		mm	mm	
3808	130%	152	18	00		7375	7525	7397	7545	74.61	5.40		157.5
			18	15		7361	7514	7387	7535	74.49	5.51		157.0
4101	140%	163.5	18	15		7318	7464	7329	7485	73.99	6.01		157.0
			18	30		7308	7453	7318	7478	73.89	6.11		156.0
4394	150%	175	18	30		7243	7378	7240	7410	73.18	6.82		156.0
			18	45		7223	7358	7220	7395	72.99	7.01		155.0
4687	160%	186.5	18	45		7160	7285	7140	7322	72.27	7.73		155.0
			19	00		7137	7260	7120	7307	72.06	7.94		154.0
4980	170%	198.5	19	00		7048	7165	7015	7205	71.08	8.92		154.0
			19	15		7016	7130	6978	7184	70.77	9.23		153.0
5273	180%	210	19	15		6952	7060	6899	7118	70.07	9.93		153.5
			19	30		6945	7049	6885	7110	69.97	10.03		152.0
5565	190%	221.5	19	30		6920	7010	6832	7068	69.58	10.43		151.5
			19	45		6909	6999	6816	7059	69.46	10.54		150.0
5858	200%	233.5	19	45		6830	6925	6710	6970	68.59	11.41		149.0
			20	00		6821	6894	6682	6955	68.38	11.62		149.0
			20	30		6825	6892	6679	6955	68.38	11.62		149.0
			21	00		6825	6882	6654	6942	68.26	11.74		149.0
			22	00		6822	6880	6649	6942	68.23	11.77		148.5
			23	00		6828	6872	6630	6932	68.16	11.85		148.0
			24	00		6824	6862	6610	6920	68.04	11.96		148.0
			01	00		6820	6854	6597	6910	67.95	12.05		148.0
			02	00		6820	6850	6590	6906	67.92	12.09		148.0
			03	00		6820	6849	6583	6906	67.90	12.11		147.5
			04	00		6820	6848	6583	6904	67.89	12.11		147.5
			05	00		6821	6848	6583	6904	67.89	12.11		147.5
			06	00		6822	6848	6581	6904	67.89	12.11		148.0
4394	150%	175	06	00		6827	6849	6623	6942	68.10	11.90		148.0
			06	15		6827	6849	6624	6942	68.11	11.90		150.0
2929	100%	117	06	15		6837	6865	6728	7016	68.62	11.39		150.0
			06	30		6841	6878	6732	7019	68.68	11.33		152.0
1465	50%	58.5	06	30		6929	7009	6904	7131	69.93	10.07		152.5
			06	45		6934	7015	6915	7138	70.01	10.00		157.0
0	0%	0	06	45		7368	7350	7290	7410	73.55	6.46		157.5
			07	00		7366	7370	7324	7422	73.71	6.30		159.0
			07	15		7497	7415	7383	7444	74.35	5.65		159.0
			07	30		7500	7420	7390	7447	74.39	5.61		159.5
			07	45		7500	7419	7494	7450	74.66	5.34		159.5
			08	45		7490	7419	7496	7443	74.62	5.38		159.0
			09	45		7481	7416	7402	7426	74.31	5.69		159.0
			10	45		7471	7410	7398	7418	74.24	5.76		159.0
			11	45		7484	7422	7411	7433	74.38	5.63		159.0

APPENDIX – B

Bore hole loge data

Borehole log data for failure load tested pile borehole No. GITPBH-2, Gidabo



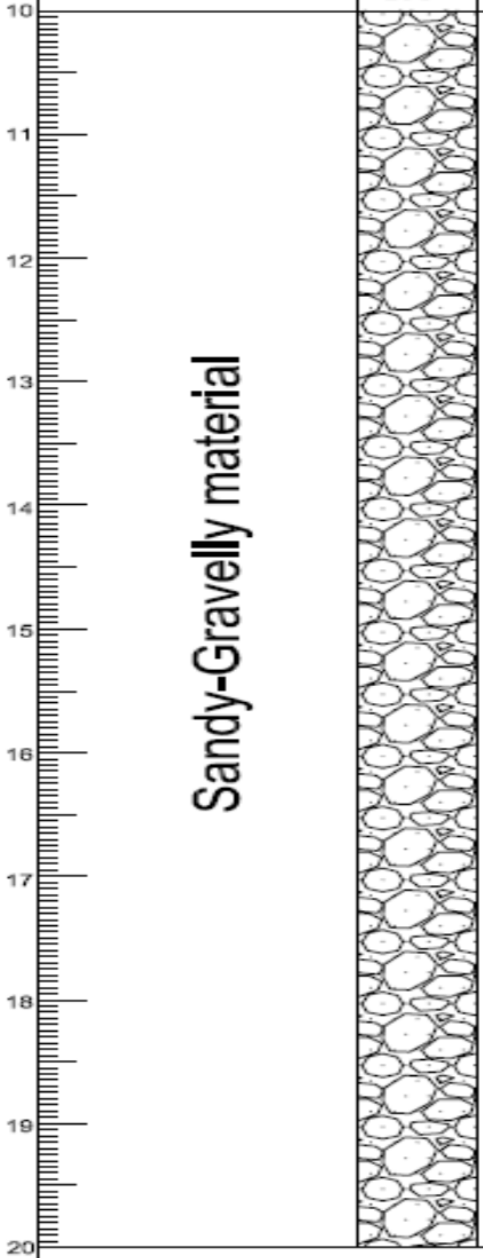
DRILLING MERTHOD Percussion drilling	Elevation : ———(m)	Co-ordinates (m) X: ——— Y: ———	BORE HOLE: GITPBH-2 Borehole Diameter: 60cm
Drilling Machine CJF-15	INCLINATION 90°	Project : Gidabo Pile Foundation project Location :Near to Pile Foundation Area	

DESCRIPTION OF CUTTING SAMPLE	SYMBOLIC LOG	DEPTH (M)	Time (24h)	Date	Sheet 1 of 3	Notes/remark
Brownish plastic silty clay material		Start 9,00m	12:55 4:20	17 July 2013 17 July 2013		
BrownishSandy -Gravelly material						

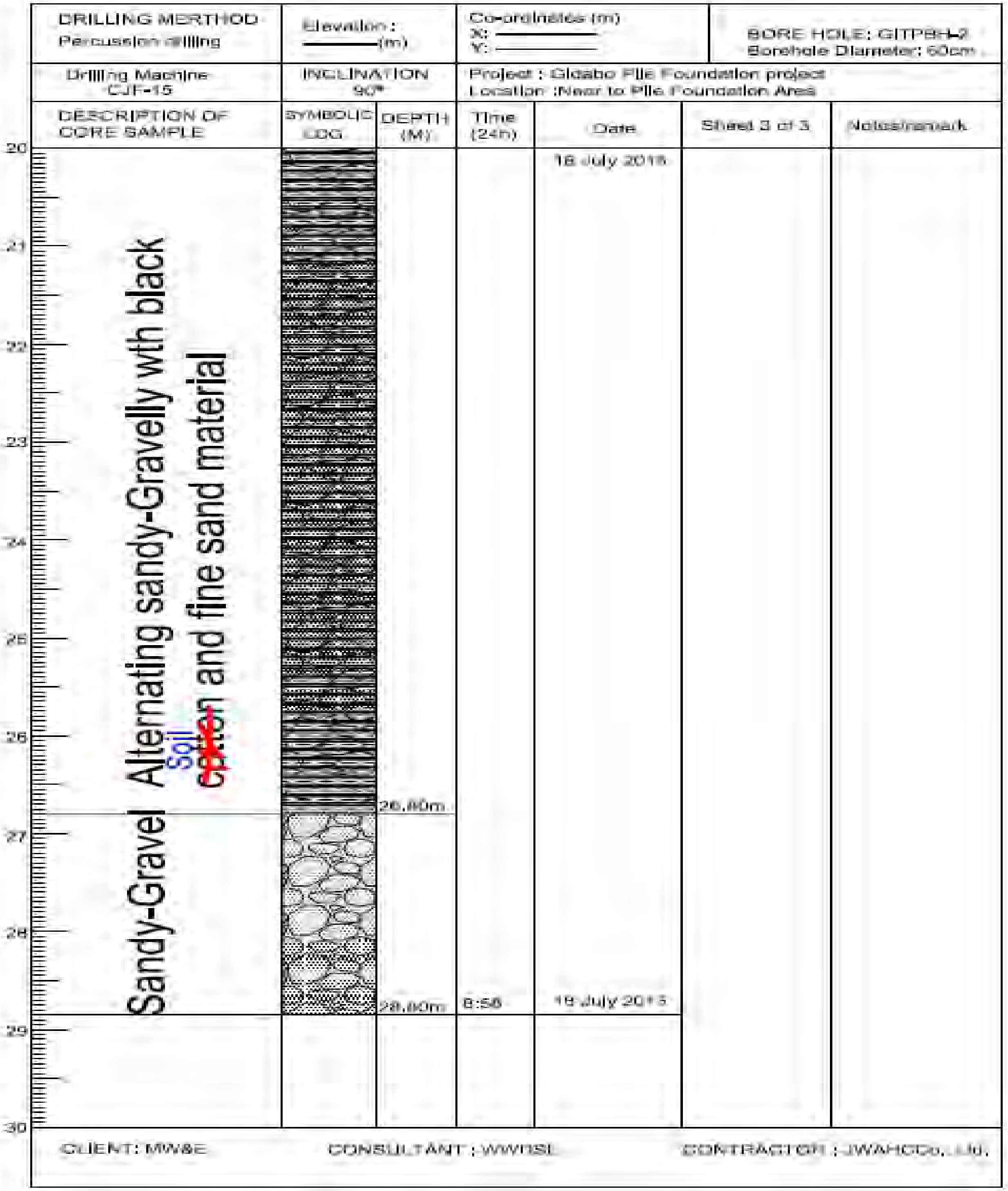
CLIENT: MW&E CONSULTANT ; WWDSE CONTRACTOR : JWAHCCo., Ltd.

DRILLING MERTHOD Percussion drilling	Elevation : ——————(m)	Co-ordinates (m) X: —————— Y: ——————	BORE HOLE: GITPBH-2 Borehole Diameter; 60cm
Drilling Machine CJF-15	INCLINATION 90°	Project : Gidabo Pile Foundation project Location :Near to Pile Foundation Area	

DESCRIPTION OF CORE SAMPLE	SYMBOLIC LOG	DEPTH (M)	Time (24h)	Date	Sheet 2 of 3	Notes/remark
Sandy-Gravelly material				17 July 2013		



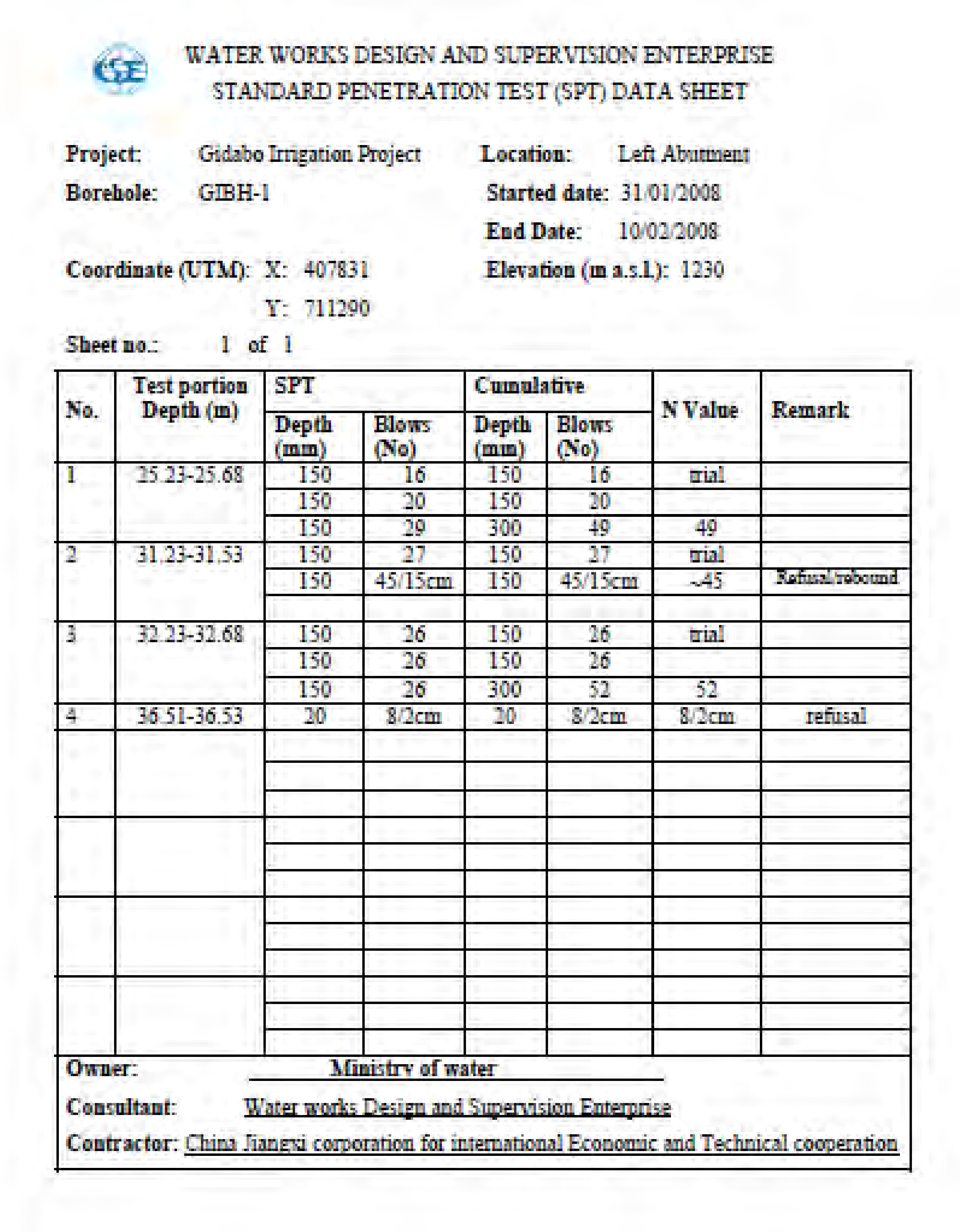
CLIENT: MW&E CONSULTANT : WWDSE CONTRACTOR : JWAHCCo., Ltd.



DRILLING MERTHOD Percussion drilling	Elevation : ————(m)	Co-ordinates (m) X: ———— Y: ————	BORE HOLE: GITPBH-2 Borehole Diameter: 60cm
Drilling Machine CJF-15	INCLINATION 90°	Project : Gidabo Pile Foundation project Location :Near to Pile Foundation Area	

DESCRIPTION OF CORE SAMPLE	SYMBOLIC LOG	DEPTH (M)	Time (24h)	Date	Sheet 3 of 3	Notes/remark
Alternating sandy-Gravelly wth black ~~cotton~~ Soil and fine sand material		26.80m		18 July 2015		
Sandy-Gravel		28.80m	8:58	18 July 2015		

CLIENT: MW&E CONSULTANT : WWDSE CONTRACTOR : JWAHCCo., Ltd.

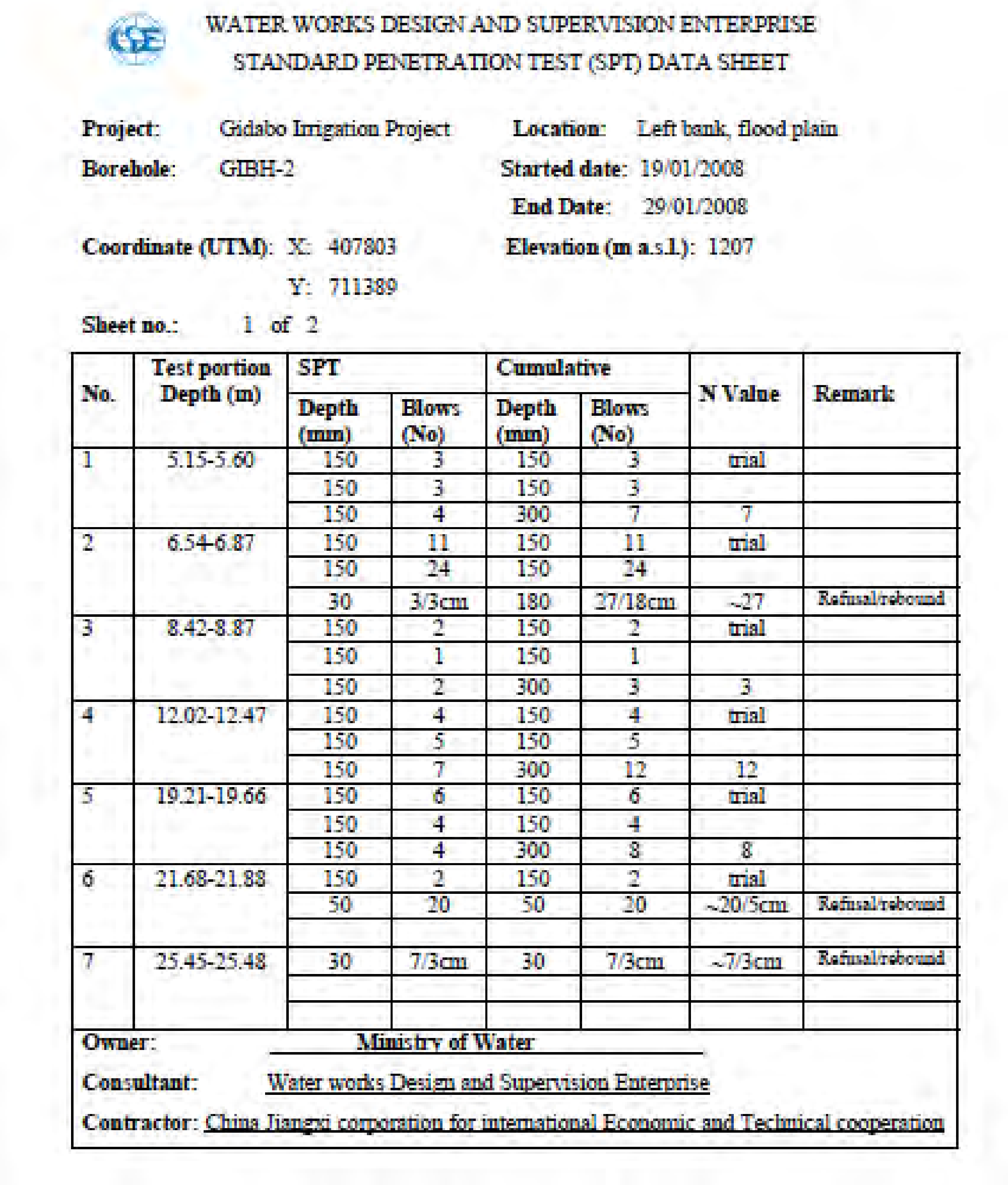


WATER WORKS DESIGN AND SUPERVISION ENTERPRISE
STANDARD PENETRATION TEST (SPT) DATA SHEET

Project: Gidabo Irrigation Project **Location:** Left Abutment
Borehole: GIBH-1 **Started date:** 31/01/2008
End Date: 10/02/2008
Coordinate (UTM): X: 407831 **Elevation (m a.s.l.):** 1230
Y: 711290
Sheet no.: 1 of 1

No.	Test portion Depth (m)	SPT		Cumulative		N Value	Remark
		Depth (mm)	Blows (No)	Depth (mm)	Blows (No)		
1	25.23-25.68	150	16	150	16	trial	
		150	20	150	20		
		150	29	300	49	49	
2	31.23-31.53	150	27	150	27	trial	
		150	45/15cm	150	45/15cm	~45	Refusal/rebound
3	32.23-32.68	150	26	150	26	trial	
		150	26	150	26		
		150	26	300	52	52	
4	36.51-36.53	20	8/2cm	20	8/2cm	8/2cm	refusal

Owner: Ministry of water
Consultant: Water works Design and Supervision Enterprise
Contractor: China Jiangxi corporation for international Economic and Technical cooperation



WATER WORKS DESIGN AND SUPERVISION ENTERPRISE
STANDARD PENETRATION TEST (SPT) DATA SHEET

Project: Gidabo Irrigation Project **Location:** Left bank, flood plain

Borehole: GIBH-2 **Started date:** 19/01/2008

End Date: 29/01/2008

Coordinate (UTM): X: 407803 **Elevation (m a.s.l.):** 1207

Y: 711389

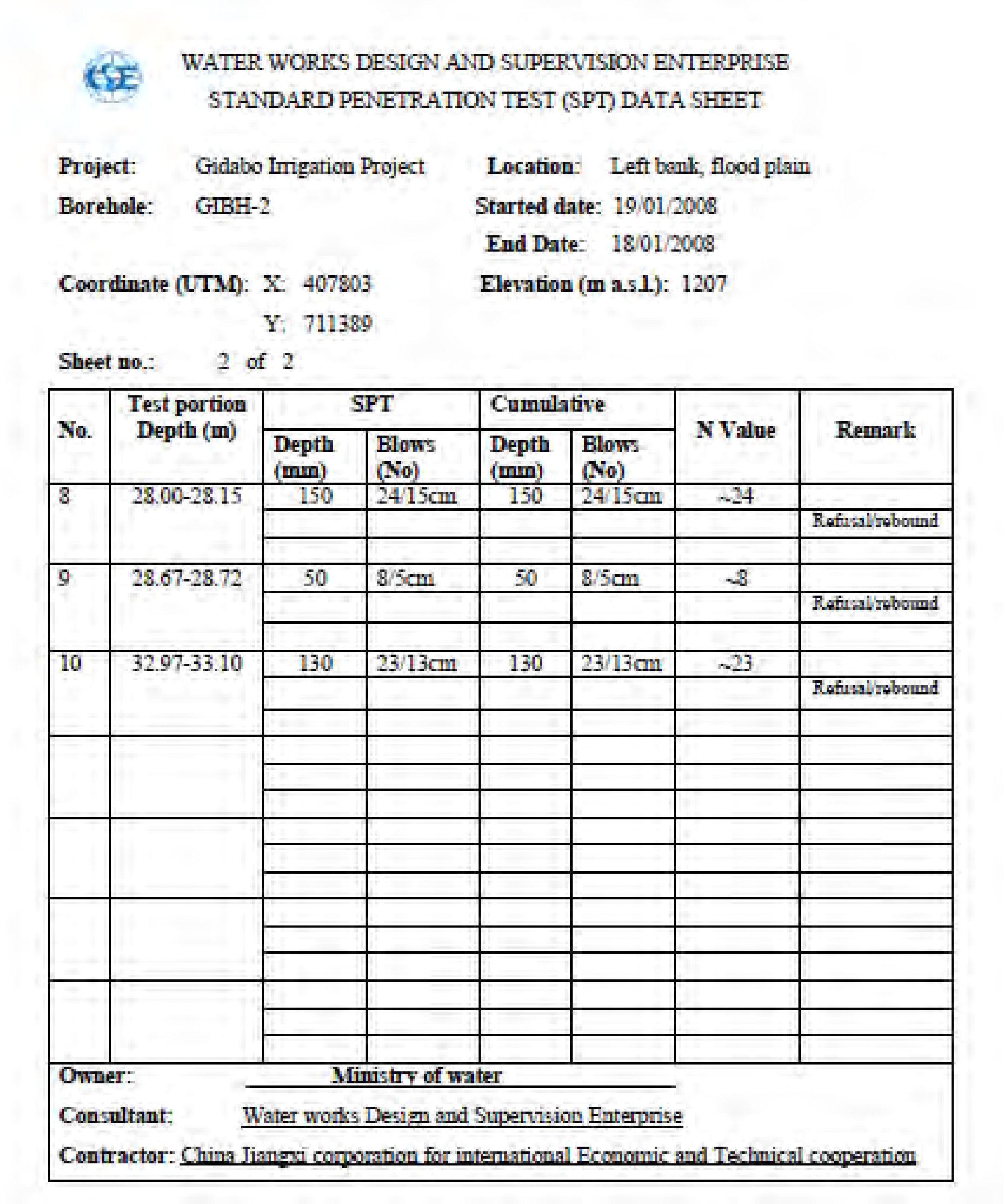
Sheet no.: 1 of 2

No.	**Test portion Depth (m)**	**SPT**		**Cumulative**		**N Value**	**Remark**
		Depth (mm)	**Blows (No)**	**Depth (mm)**	**Blows (No)**		
1	5.15-5.60	150	3	150	3	trial	
		150	3	150	3		
		150	4	300	7	7	
2	6.54-6.87	150	11	150	11	trial	
		150	24	150	24		
		30	3/3cm	180	27/18cm	~27	Refusal/rebound
3	8.42-8.87	150	2	150	2	trial	
		150	1	150	1		
		150	2	300	3	3	
4	12.02-12.47	150	4	150	4	trial	
		150	5	150	5		
		150	7	300	12	12	
5	19.21-19.66	150	6	150	6	trial	
		150	4	150	4		
		150	4	300	8	8	
6	21.68-21.88	150	2	150	2	trial	
		50	20	50	20	~20/5cm	Refusal/rebound
7	25.45-25.48	30	7/3cm	30	7/3cm	~7/3cm	Refusal/rebound

Owner: Ministry of Water

Consultant: Water works Design and Supervision Enterprise

Contractor: China Jiangxi corporation for international Economic and Technical cooperation



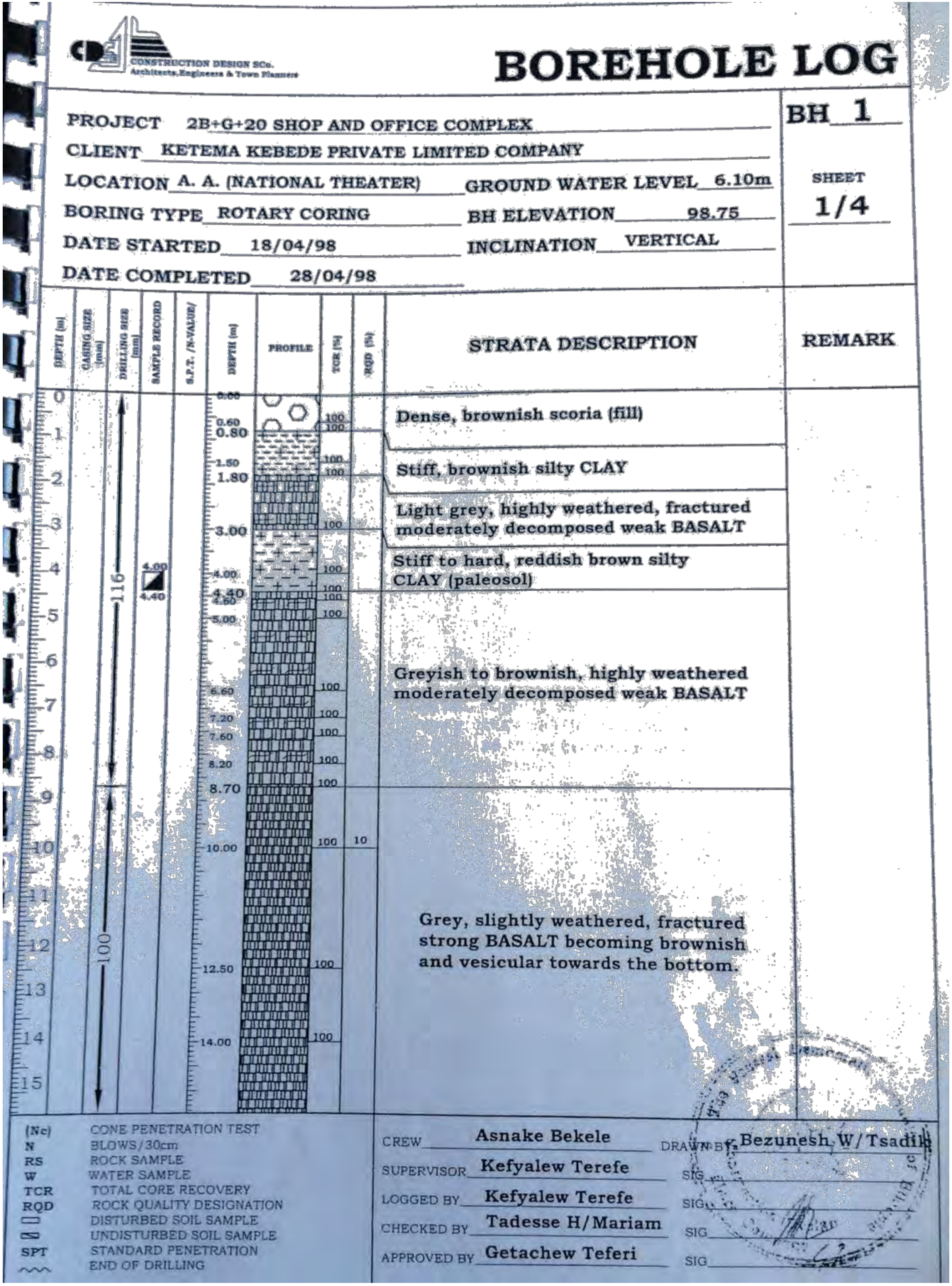
WATER WORKS DESIGN AND SUPERVISION ENTERPRISE
STANDARD PENETRATION TEST (SPT) DATA SHEET

Project: Gidabo Irrigation Project
Location: Left bank, flood plain
Borehole: GIBH-2
Started date: 19/01/2008
End Date: 18/01/2008
Coordinate (UTM): X: 407803
Y: 711389
Elevation (m a.s.l.): 1207
Sheet no.: 2 of 2

No.	Test portion Depth (m)	SPT		Cumulative		N Value	Remark
		Depth (mm)	Blows (No)	Depth (mm)	Blows (No)		
8	28.00-28.15	150	24/15cm	150	24/15cm	~24	
							Refusal/rebound
9	28.67-28.72	50	8/5cm	50	8/5cm	~8	
							Refusal/rebound
10	32.97-33.10	130	23/13cm	130	23/13cm	~23	
							Refusal/rebound

Owner: Ministry of water
Consultant: Water works Design and Supervision Enterprise
Contractor: China Jiangxi corporation for international Economic and Technical cooperation

Borehole log data for working pile load test, K.K. project



CONSTRUCTION DESIGN SCo.
Architects, Engineers & Town Planners

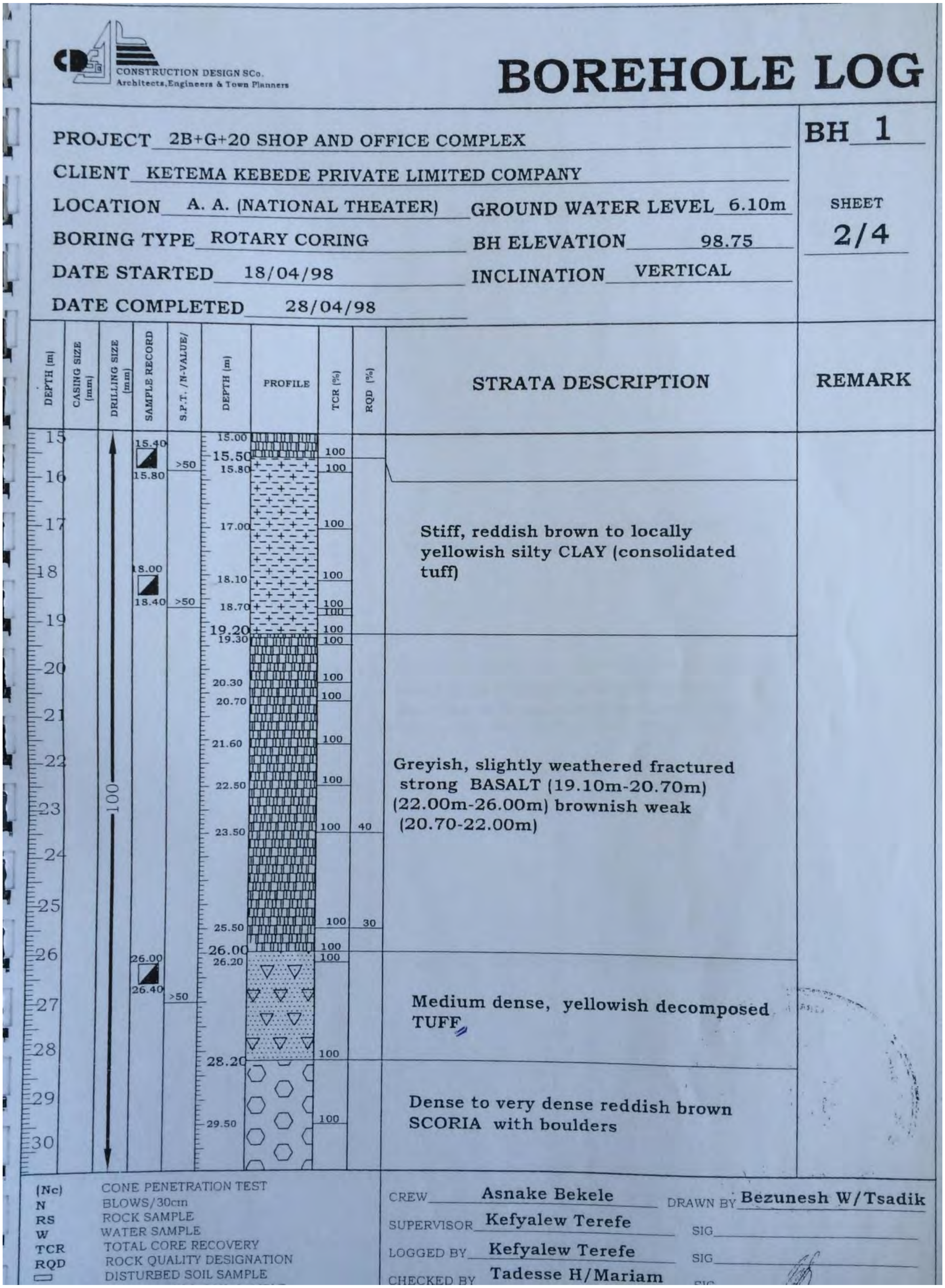
BOREHOLE LOG

PROJECT 2B+G+20 SHOP AND OFFICE COMPLEX
CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY
LOCATION A. A. (NATIONAL THEATER)
GROUND WATER LEVEL 6.10m
BORING TYPE ROTARY CORING
BH ELEVATION 98.75
DATE STARTED 18/04/98
INCLINATION VERTICAL
DATE COMPLETED 28/04/98
BH 1
SHEET 1/4

DEPTH (m)	DRILLING SIZE (mm)	SAMPLE RECORD	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
0.00	116				Dense, brownish scoria (fill)	
0.60			100			
0.80			100		Stiff, brownish silty CLAY	
1.50			100			
1.80			100		Light grey, highly weathered, fractured moderately decomposed weak BASALT	
3.00			100		Stiff to hard, reddish brown silty CLAY (paleosol)	
4.00		4.00	100			
4.40		4.40	100		Greyish to brownish, highly weathered moderately decomposed weak BASALT	
4.60			100			
5.00			100			
6.60			100			
7.20			100			
7.60			100			
8.20			100			
8.70	100		100		Grey, slightly weathered, fractured strong BASALT becoming brownish and vesicular towards the bottom.	
10.00			100	10		
12.50			100			
14.00			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 Asnake Bekele
DRAWN BY Bezunesh W/Tsadik
SUPERVISOR Kefyalew Terefe
SIG
LOGGED BY Kefyalew Terefe
SIG
CHECKED BY Tadesse H/Mariam
SIG
APPROVED BY Getachew Teferi
SIG



CONSTRUCTION DESIGN SCo.
Architects,Engineers & Town Planners

BOREHOLE LOG

PROJECT 2B+G+20 SHOP AND OFFICE COMPLEX

CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY

LOCATION A. A. (NATIONAL THEATER) GROUND WATER LEVEL 6.10m

BORING TYPE ROTARY CORING BH ELEVATION 98.75

DATE STARTED 18/04/98 INCLINATION VERTICAL

DATE COMPLETED 28/04/98

BH 1

SHEET 2/4

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	15.40–15.80	>50	15.00					
					15.50		100			
16					15.80		100		Stiff, reddish brown to locally yellowish silty CLAY (consolidated tuff)	
17					17.00		100			
18			18.00–18.40	>50	18.10		100			
					18.70		100			
							100			
19					19.20		100		Greyish, slightly weathered fractured strong BASALT (19.10m-20.70m) (22.00m-26.00m) brownish weak (20.70-22.00m)	
					19.30		100			
20					20.30		100			
21					20.70		100			
22					21.60		100			
23					22.50		100			
24					23.50		100	40		
25					25.50		100	30		
26			26.00–26.40	>50	26.00		100		Medium dense, yellowish decomposed TUFF	
27					26.20		100			
28					28.20		100		Dense to very dense reddish brown SCORIA with boulders	
29					29.50		100			
30										

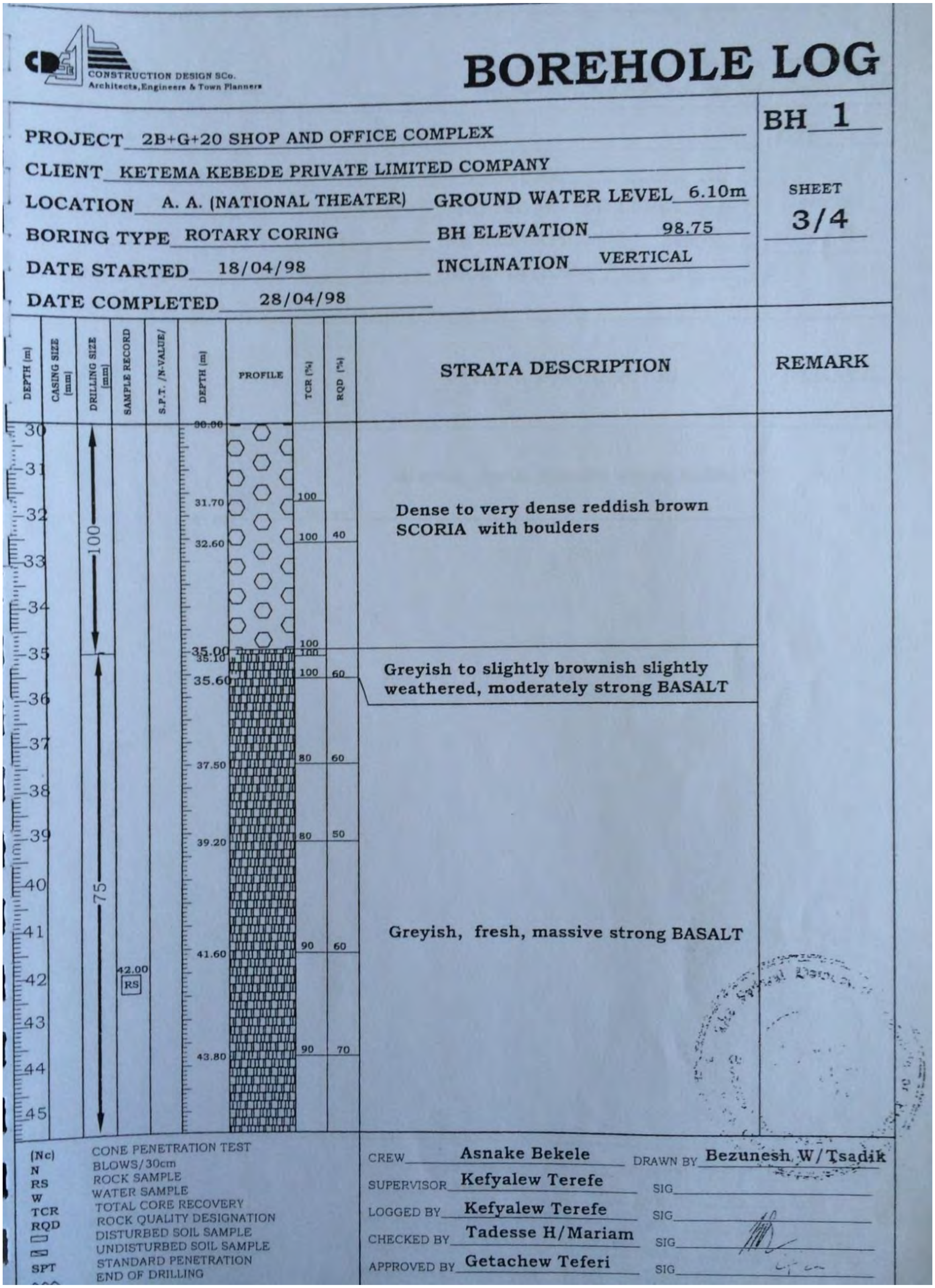
(Nc)	CONE PENETRATION TEST
N	BLOWS/30cm
RS	ROCK SAMPLE
W	WATER SAMPLE
TCR	TOTAL CORE RECOVERY
RQD	ROCK QUALITY DESIGNATION
▭	DISTURBED SOIL SAMPLE

CREW Asnake Bekele DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Kefyalew Terefe SIG

LOGGED BY Kefyalew Terefe SIG

CHECKED BY Tadesse H/Mariam



CONSTRUCTION DESIGN SCo.
Architects, Engineers & Town Planners

BOREHOLE LOG

BH 1

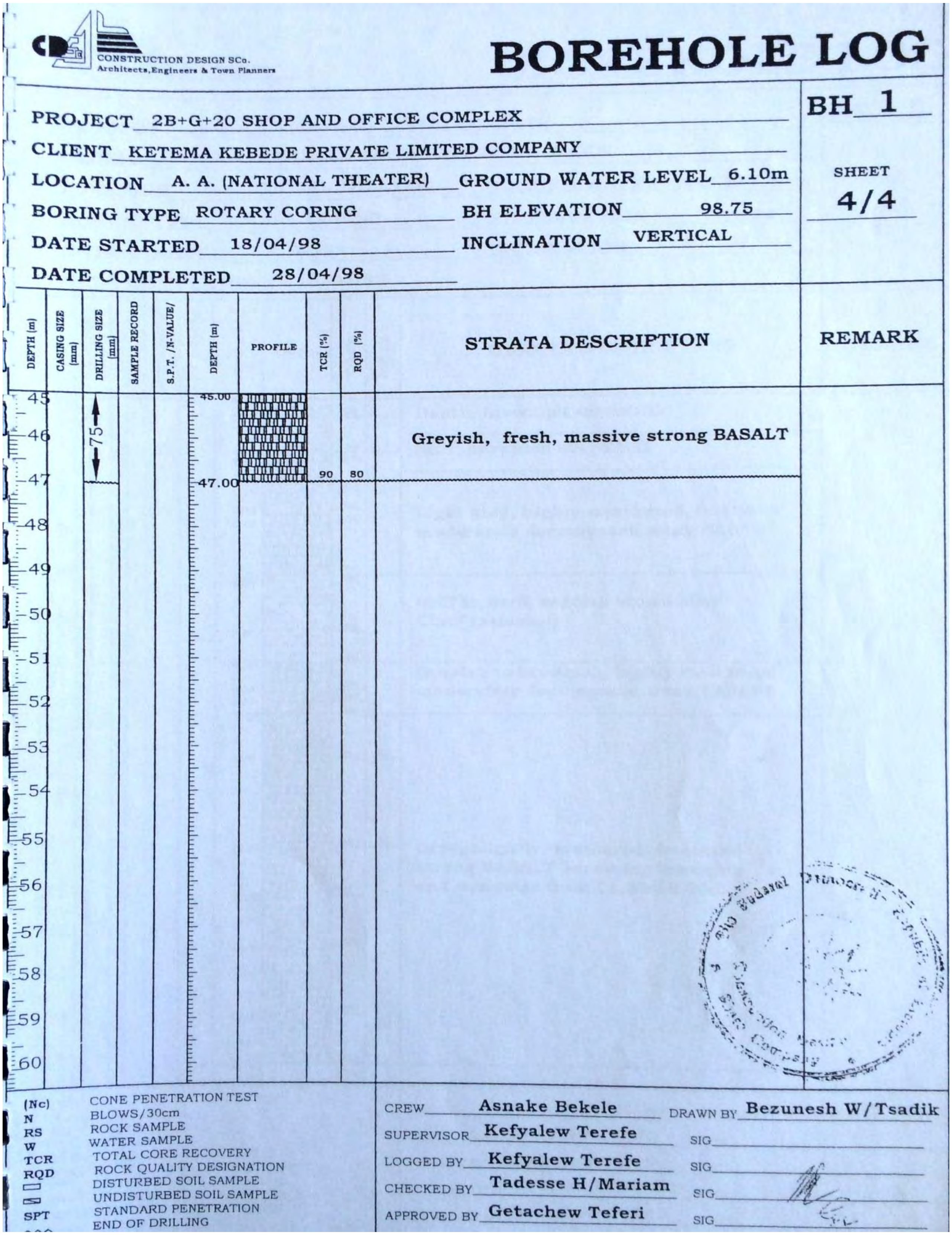
SHEET 3/4

PROJECT 2B+G+20 SHOP AND OFFICE COMPLEX
CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY
LOCATION A. A. (NATIONAL THEATER) GROUND WATER LEVEL 6.10m
BORING TYPE ROTARY CORING BH ELEVATION 98.75
DATE STARTED 18/04/98 INCLINATION VERTICAL
DATE COMPLETED 28/04/98

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
30		100			30.00				Dense to very dense reddish brown SCORIA with boulders	
31										
					31.70		100			
32										
					32.60		100	40		
33										
34										
35		75			35.00		100			
					35.10		100		Greyish to slightly brownish slightly weathered, moderately strong BASALT	
					35.60		100	60		
36										
37										
					37.50		80	60	Greyish, fresh, massive strong BASALT	
38										
39										
					39.20		80	50		
40										
41										
					41.60		90	60		
42			42.00 RS							
43										
					43.80		90	70		
44										
45										

(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 Asnake Bekele DRAWN BY Bezunesh W/Tsadik
SUPERVISOR Kefyalew Terefe SIG
LOGGED BY Kefyalew Terefe SIG
CHECKED BY Tadesse H/Mariam SIG
APPROVED BY Getachew Teferi SIG



CONSTRUCTION DESIGN SCo.
Architects,Engineers & Town Planners

BOREHOLE LOG

BH 1

SHEET 4/4

PROJECT 2B+G+20 SHOP AND OFFICE COMPLEX

CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY

LOCATION A. A. (NATIONAL THEATER) **GROUND WATER LEVEL** 6.10m

BORING TYPE ROTARY CORING **BH ELEVATION** 98.75

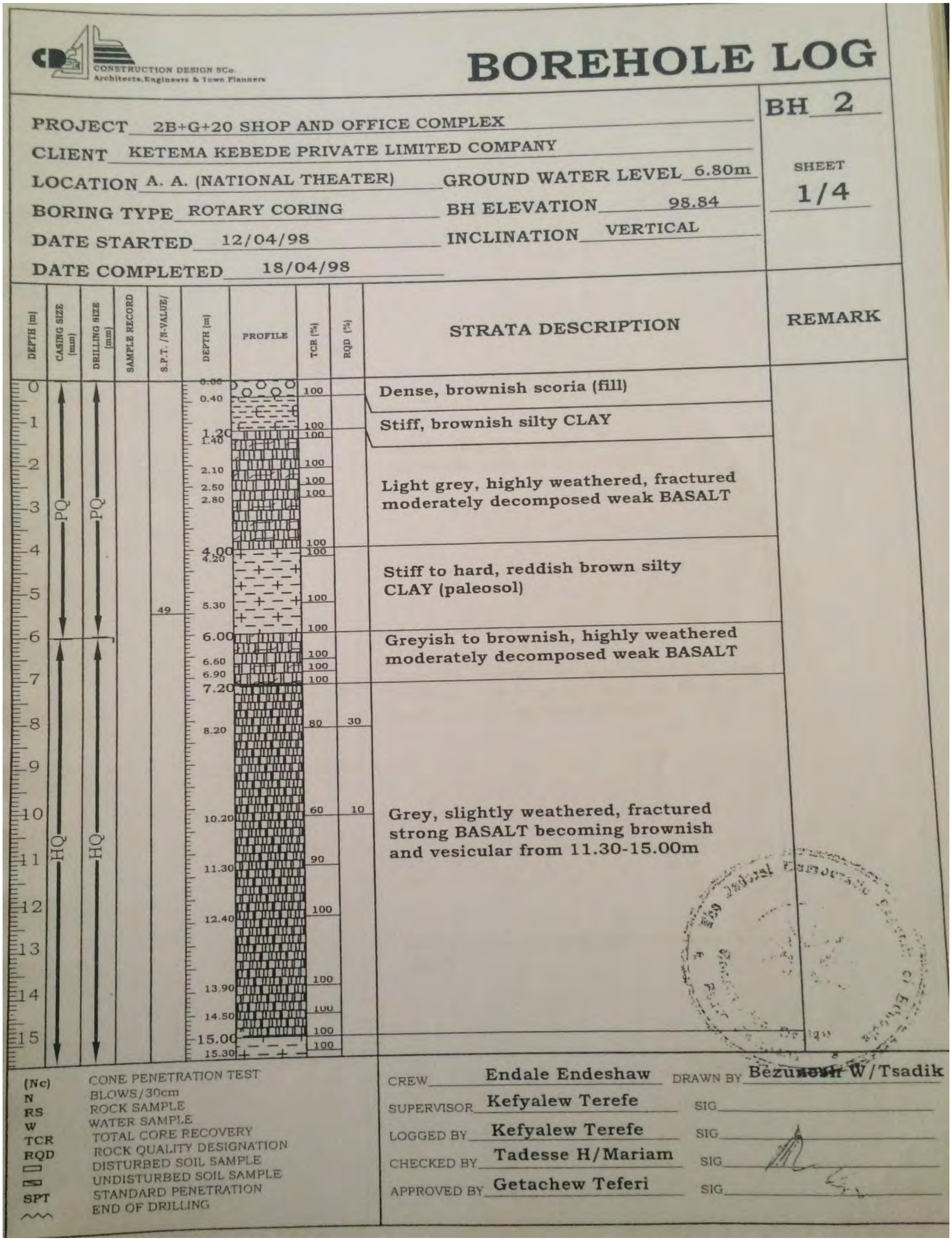
DATE STARTED 18/04/98 **INCLINATION** VERTICAL

DATE COMPLETED 28/04/98

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
45–47		75			45.00–47.00	▨	90	80	Greyish, fresh, massive strong BASALT	
47–60										

(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 Asnake Bekele DRAWN BY Bezunesh W/Tsadik
SUPERVISOR Kefyalew Terefe SIG
LOGGED BY Kefyalew Terefe SIG
CHECKED BY Tadesse H/Mariam SIG
APPROVED BY Getachew Teferi SIG



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BOREHOLE LOG

BH 2

SHEET 1/4

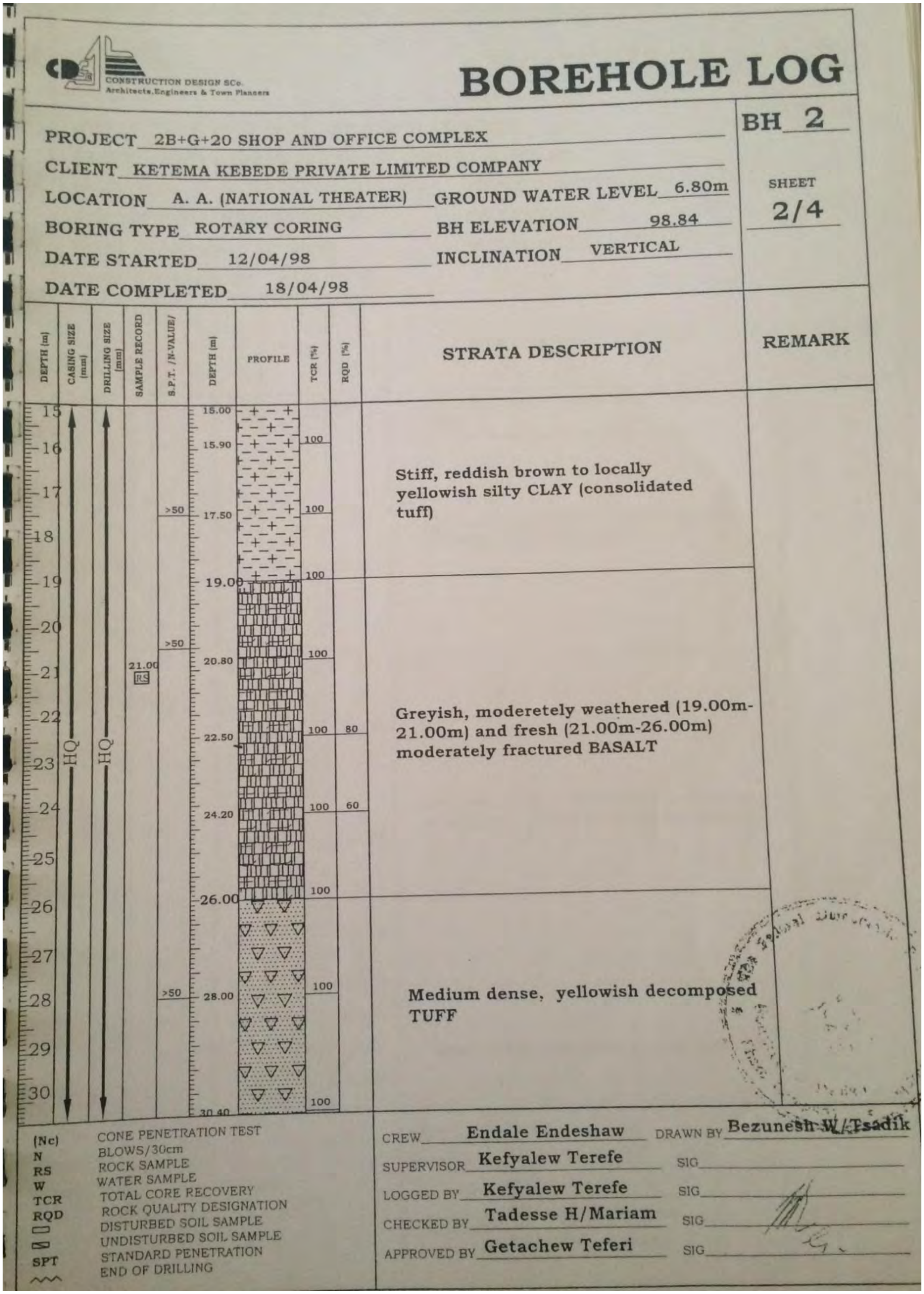
PROJECT 2B+G+20 SHOP AND OFFICE COMPLEX
CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY
LOCATION A. A. (NATIONAL THEATER) GROUND WATER LEVEL 6.80m
BORING TYPE ROTARY CORING BH ELEVATION 98.84
DATE STARTED 12/04/98 INCLINATION VERTICAL
DATE COMPLETED 18/04/98

DEPTH [m]	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH [m]	TCR (%)	RQD (%)	STRATA DESCRIPTION
0	PQ	PQ			0.00			Dense, brownish scoria (fill)
					0.40	100		
1					1.20	100		Stiff, brownish silty CLAY
					1.40	100		Light grey, highly weathered, fractured moderately decomposed weak BASALT
2					2.10	100		
					2.50	100		
3					2.80	100		
4					4.00	100		Stiff to hard, reddish brown silty CLAY (paleosol)
					4.20	100		
5				49	5.30	100		
6	HQ	HQ			6.00	100		Greyish to brownish, highly weathered moderately decomposed weak BASALT
					6.60	100		
					6.90	100		
7					7.20	100		Grey, slightly weathered, fractured strong BASALT becoming brownish and vesicular from 11.30-15.00m
8					8.20	80	30	
10					10.20	60	10	
11					11.30	90		
12					12.40	100		
13					13.90	100		
14					14.50	100		
15					15.00	100		
					15.30	100		

REMARK

(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 Endale Endeshaw DRAWN BY Bezuneh W/Tsadik
SUPERVISOR Kefyalew Terefe SIG
LOGGED BY Kefyalew Terefe SIG
CHECKED BY Tadesse H/Mariam SIG
APPROVED BY Getachew Teferi SIG



CONSTRUCTION DESIGN SCo.
Architects, Engineers & Town Planners

BOREHOLE LOG

BH 2

PROJECT 2B+G+20 SHOP AND OFFICE COMPLEX

CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY

LOCATION A. A. (NATIONAL THEATER) GROUND WATER LEVEL 6.80m

BORING TYPE ROTARY CORING BH ELEVATION 98.84

DATE STARTED 12/04/98 INCLINATION VERTICAL

DATE COMPLETED 18/04/98

SHEET 2/4

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
15	HQ	HQ			15.00				Stiff, reddish brown to locally yellowish silty CLAY (consolidated tuff)	
16					15.90		100			
17				>50	17.50		100			
18										
19					19.00		100		Greyish, moderetely weathered (19.00m-21.00m) and fresh (21.00m-26.00m) moderately fractured BASALT	
20				>50						
21			21.00 RS		20.80		100			
22					22.50		100	80		
23										
24					24.20		100	60		
25										
26					26.00		100		Medium dense, yellowish decomposed TUFF	
27										
28				>50	28.00		100			
29										
30					30.40		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

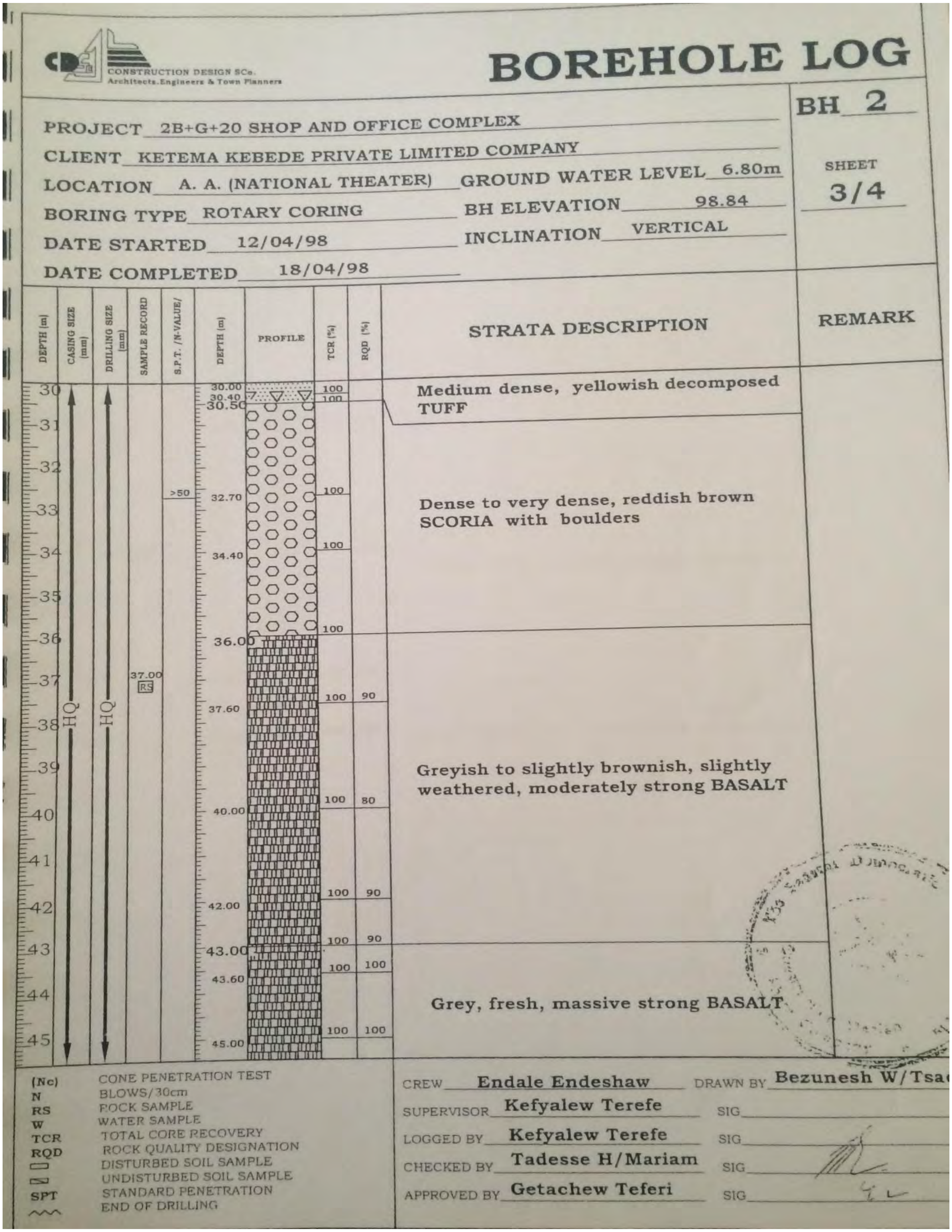
CREW Endale Endeshaw DRAWN BY Bezunesh W/Tsadik

SUPERVISOR Kefyalew Terefe SIG

LOGGED BY Kefyalew Terefe SIG

CHECKED BY Tadesse H/Mariam SIG

APPROVED BY Getachew Teferi SIG



CONSTRUCTION DESIGN SCo.
Architects.Engineers & Town Planners

BOREHOLE LOG

BH 2

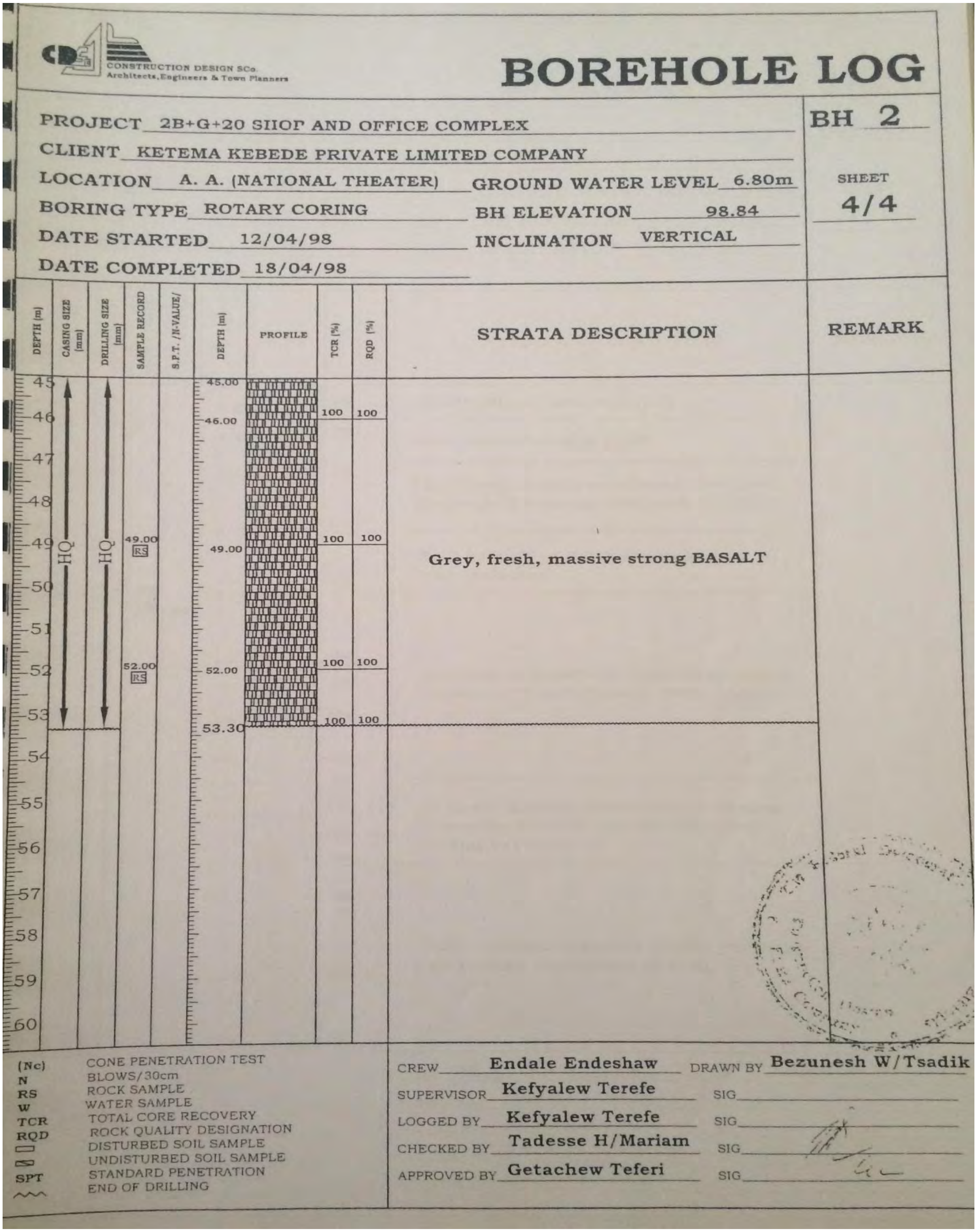
SHEET 3/4

PROJECT 2B+G+20 SHOP AND OFFICE COMPLEX
CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY
LOCATION A. A. (NATIONAL THEATER) GROUND WATER LEVEL 6.80m
BORING TYPE ROTARY CORING BH ELEVATION 98.84
DATE STARTED 12/04/98 INCLINATION VERTICAL
DATE COMPLETED 18/04/98

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
30	HQ	HQ			30.00	100		Medium dense, yellowish decomposed TUFF	
					30.40	100			
					30.50			Dense to very dense, reddish brown SCORIA with boulders	
				>50	32.70	100			
					34.40	100			
					36.00	100		Greyish to slightly brownish, slightly weathered, moderately strong BASALT	
			37.00 RS						
					37.60	100	90		
					40.00	100	80		
					42.00	100	90		
					43.00	100	90	Grey, fresh, massive strong BASALT	
					43.60	100	100		
45					45.00	100	100		

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

CREW Endale Endeshaw DRAWN BY Bezunesh W/Tsa
SUPERVISOR Kefyalew Terefe SIG
LOGGED BY Kefyalew Terefe SIG
CHECKED BY Tadesse H/Mariam SIG
APPROVED BY Getachew Teferi SIG



CONSTRUCTION DESIGN SCo.
Architects, Engineers & Town Planners

BOREHOLE LOG

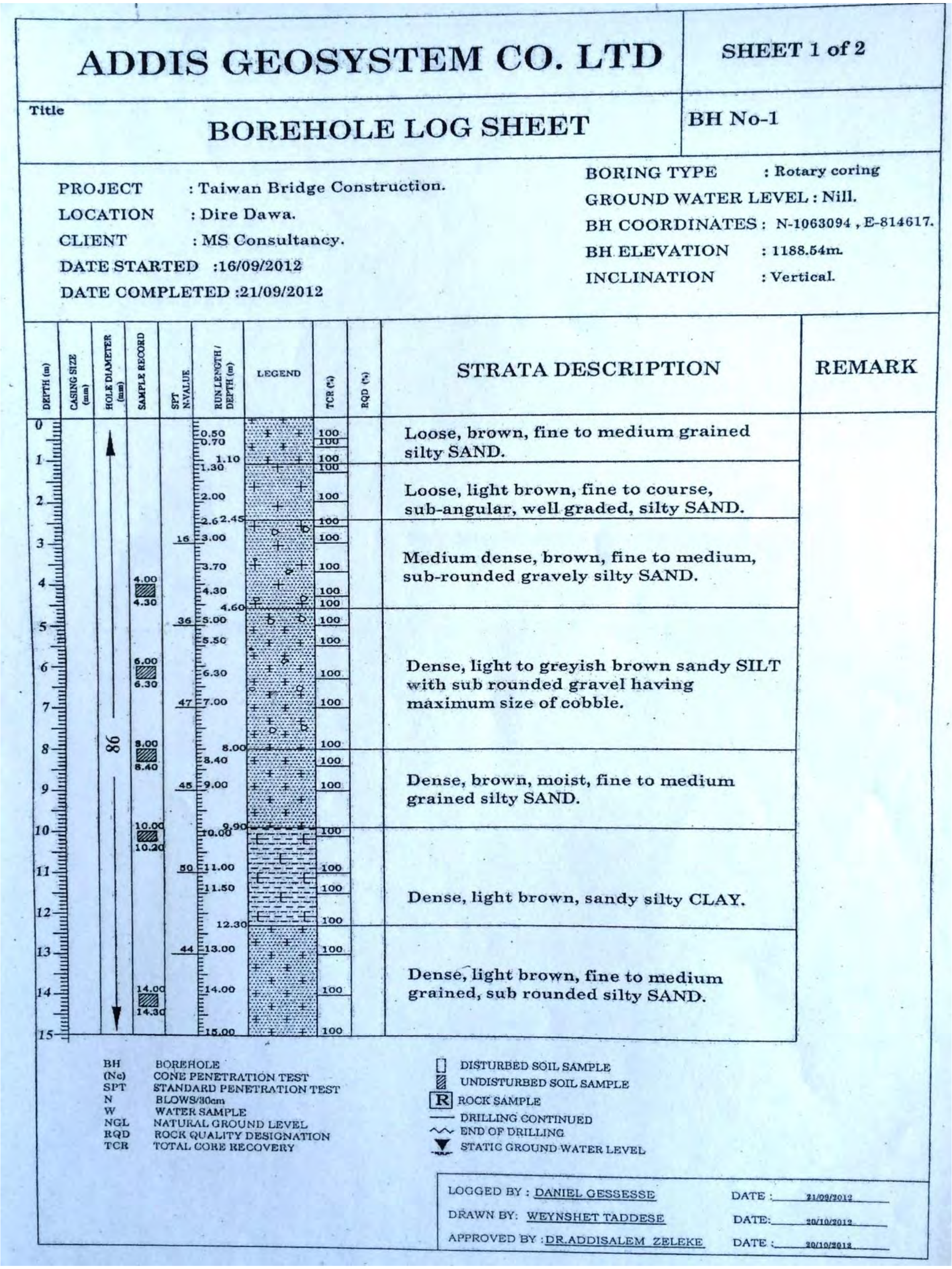
PROJECT 2B+G+20 SIIOP AND OFFICE COMPLEX
CLIENT KETEMA KEBEDE PRIVATE LIMITED COMPANY
LOCATION A. A. (NATIONAL THEATER)
GROUND WATER LEVEL 6.80m
BORING TYPE ROTARY CORING
BH ELEVATION 98.84
DATE STARTED 12/04/98
INCLINATION VERTICAL
DATE COMPLETED 18/04/98
BH 2
SHEET 4/4

DEPTH (m)	CASING SIZE (mm)	DRILLING SIZE (mm)	SAMPLE RECORD	S.P.T. /N-VALUE/	DEPTH (m)	PROFILE	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
45	HQ	HQ			45.00				Grey, fresh, massive strong BASALT	
46					46.00		100	100		
47										
48										
49			49.00 RS		49.00		100	100		
50										
51										
52			52.00 RS		52.00		100	100		
53					53.30		100	100		
54										
55										
56										
57										
58										
59										
60										

(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 Endale Endeshaw
DRAWN BY Bezunesh W/Tsadik
SUPERVISOR Kefyalew Terefe
SIG
LOGGED BY Kefyalew Terefe
SIG
CHECKED BY Tadesse H/Mariam
SIG
APPROVED BY Getachew Teferi
SIG

Borehole log sheet data for working pile load test, Dire Dawa Taiwan Bridge Project



ADDIS GEOSYSTEM CO. LTD

SHEET 1 of 2

Title

BOREHOLE LOG SHEET

BH No-1

PROJECT : Taiwan Bridge Construction.
LOCATION : Dire Dawa.
CLIENT : MS Consultancy.
DATE STARTED :16/09/2012
DATE COMPLETED :21/09/2012

BORING TYPE : Rotary coring
GROUND WATER LEVEL : Nill.
BH COORDINATES : N-1063094 , E-814617.
BH ELEVATION : 1188.54m.
INCLINATION : Vertical.

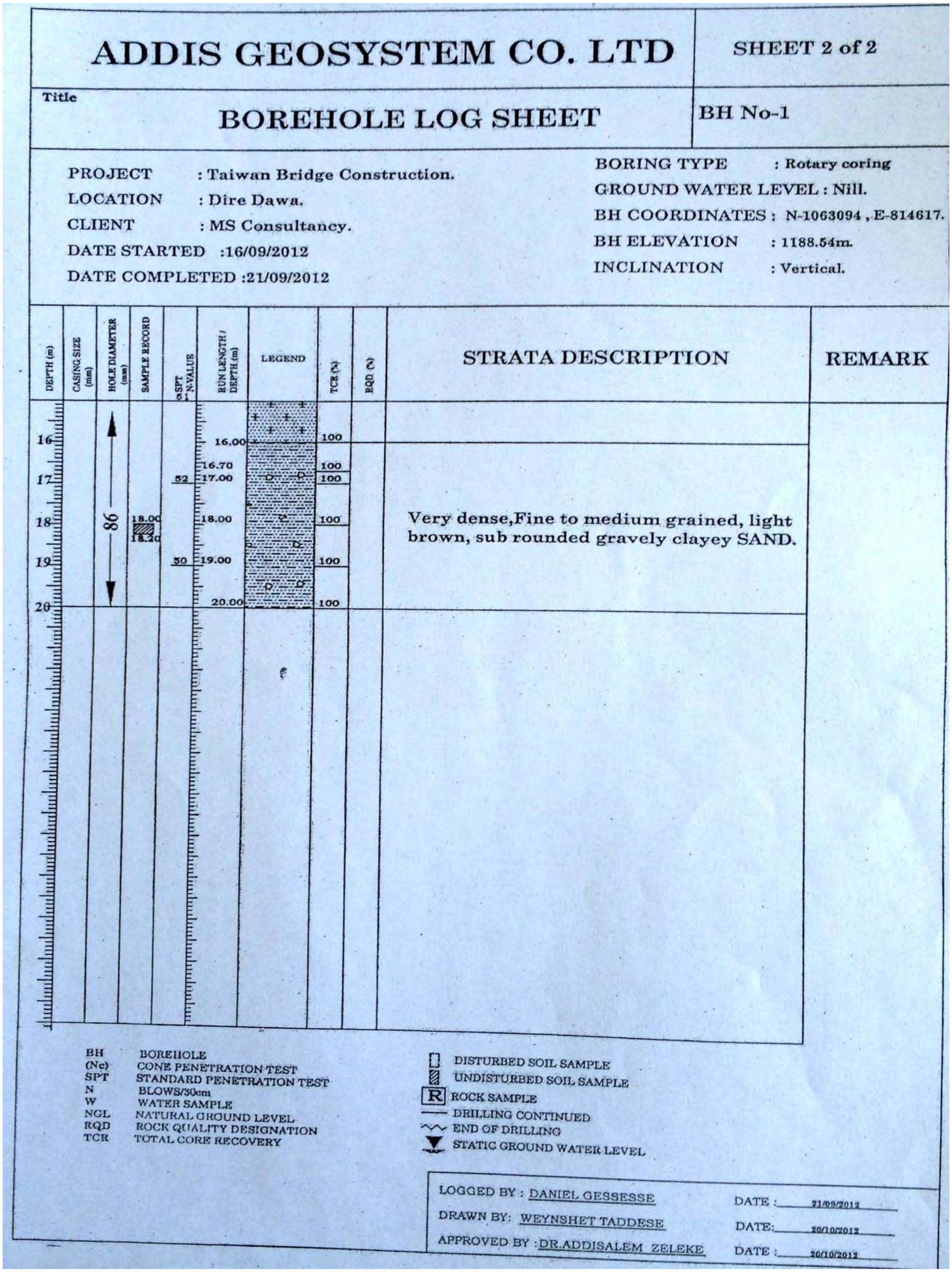
DEPTH (m)	CASING SIZE (mm)	HOLE DIAMETER (mm)	SAMPLE RECORD	SPT N-VALUE	RUN LENGTH / DEPTH (m)	TCR (%)	RQD (%)
0		86			0.50	100	
					0.70	100	
1					1.10	100	
					1.30	100	
2					2.00	100	
					2.6	100	
					2.45		
3				16	3.00	100	
					3.70	100	
4			4.00 ▨ 4.30		4.30	100	
					4.60	100	
5				36	5.00	100	
					5.50	100	
6			6.00 ▨ 6.30		6.30	100	
7				47	7.00	100	
8			8.00 ▨ 8.40		8.00	100	
					8.40	100	
9				45	9.00	100	
					9.90		
10			10.00 ▨ 10.20		10.00	100	
11				50	11.00	100	
					11.50	100	
12					12.30	100	
13				44	13.00	100	
14			14.00 ▨ 14.30		14.00	100	
15					15.00	100	

STRATA DESCRIPTION	REMARK
Loose, brown, fine to medium grained silty SAND.	
Loose, light brown, fine to course, sub-angular, well graded, silty SAND.	
Medium dense, brown, fine to medium, sub-rounded gravely silty SAND.	
Dense, light to greyish brown sandy SILT with sub rounded gravel having maximum size of cobble.	
Dense, brown, moist, fine to medium grained silty SAND.	
Dense, light brown, sandy silty CLAY.	
Dense, light brown, fine to medium grained, sub rounded silty SAND.	

BH	BOREHOLE
(Nc)	CONE PENETRATION TEST
SPT	STANDARD PENETRATION TEST
N	BLOWS/30cm
W	WATER SAMPLE
NGL	NATURAL GROUND LEVEL
RQD	ROCK QUALITY DESIGNATION
TCR	TOTAL CORE RECOVERY

☐	DISTURBED SOIL SAMPLE
▨	UNDISTURBED SOIL SAMPLE
R	ROCK SAMPLE
—	DRILLING CONTINUED
〰	END OF DRILLING
▼	STATIC GROUND WATER LEVEL

LOGGED BY : DANIEL GESSESSE DATE : 21/09/2012
DRAWN BY: WEYNSHET TADDESE DATE: 20/10/2012
APPROVED BY : DR.ADDISALEM ZELEKE DATE : 20/10/2012



ADDIS GEOSYSTEM CO. LTD

SHEET 2 of 2

Title

BOREHOLE LOG SHEET

BH No-1

PROJECT : Taiwan Bridge Construction.
LOCATION : Dire Dawa.
CLIENT : MS Consultancy.
DATE STARTED :16/09/2012
DATE COMPLETED :21/09/2012

BORING TYPE : Rotary coring
GROUND WATER LEVEL : Nill.
BH COORDINATES : N-1063094 , E-814617.
BH ELEVATION : 1188.54m.
INCLINATION : Vertical.

DEPTH (m)	CASING SIZE (mm)	HOLE DIAMETER (mm)	SAMPLE RECORD	SPT N-VALUE	RUN LENGTH / DEPTH (m)	LEGEND	TCR (%)	RQD (%)	STRATA DESCRIPTION	REMARK
16		86			16.00		100			
					16.70		100		Very dense,Fine to medium grained, light brown, sub rounded gravely clayey SAND.	
17				52	17.00		100			
18			18.00 ▨ 18.20		18.00		100			
19				50	19.00		100			
20					20.00		100			

BH	BOREHOLE
(Nc)	CONE PENETRATION TEST
SPT	STANDARD PENETRATION TEST
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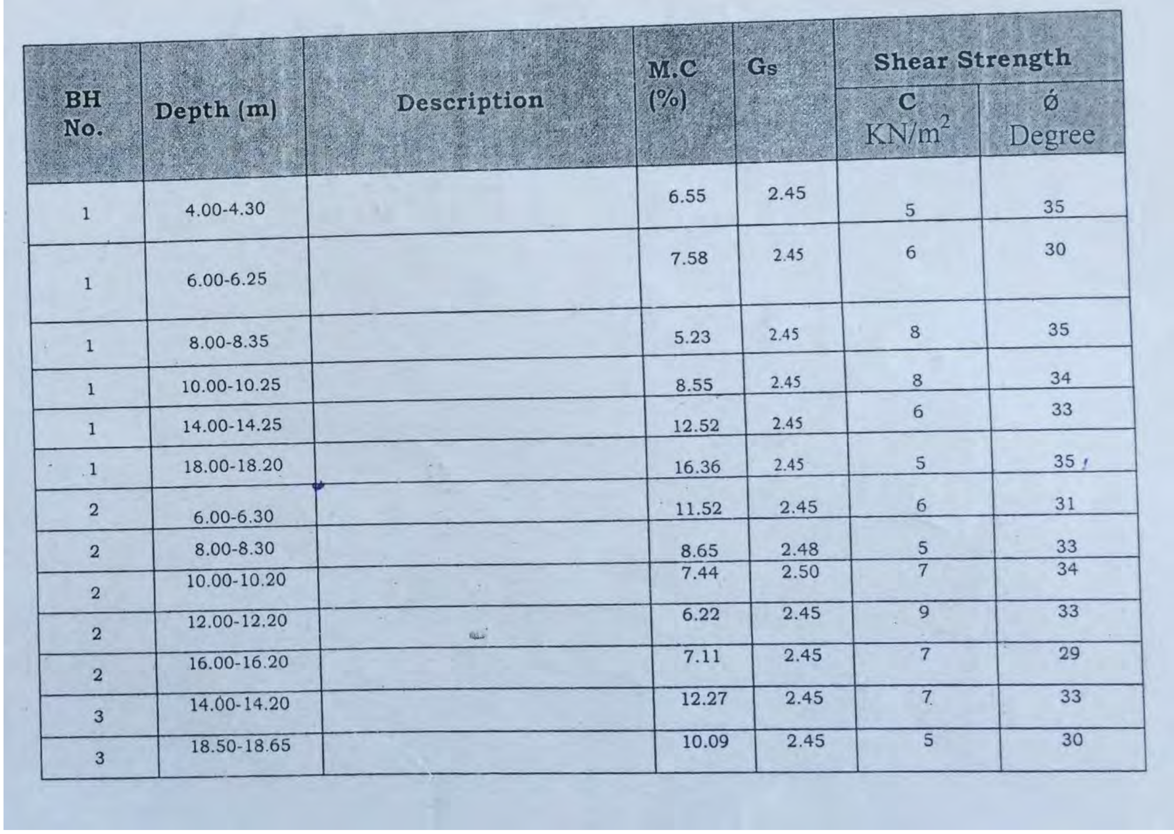
TCR	TOTAL CORE RECOVERY

▯	DISTURBED SOIL SAMPLE
▨	UNDISTURBED SOIL SAMPLE
R	ROCK SAMPLE
—	DRILLING CONTINUED
〰	END OF DRILLING
▼	STATIC GROUND WATER LEVEL

LOGGED BY : DANIEL GESSESSE DATE : 21/09/2012
DRAWN BY: WEYNSHET TADDESE DATE: 20/10/2012
APPROVED BY : DR.ADDISALEM ZELEKE DATE : 20/10/2012

Summary of laboratory test result for Dire Dawa Taiwan Bridge Project



BH No.	Depth (m)	Description	M.C (%)	Gs	Shear Strength	
					C KN/m^2	Ø Degree
1	4.00-4.30		6.55	2.45	5	35
1	6.00-6.25		7.58	2.45	6	30
1	8.00-8.35		5.23	2.45	8	35
1	10.00-10.25		8.55	2.45	8	34
1	14.00-14.25		12.52	2.45	6	33
1	18.00-18.20		16.36	2.45	5	35
2	6.00-6.30		11.52	2.45	6	31
2	8.00-8.30		8.65	2.48	5	33
2	10.00-10.20		7.44	2.50	7	34
2	12.00-12.20		6.22	2.45	9	33
2	16.00-16.20		7.11	2.45	7	29
3	14.00-14.20		12.27	2.45	7	33
3	18.50-18.65		10.09	2.45	5	30

APPENDIX C

Cap Hardening Calculation Sheets and Generated Graphs

Canculation sheet used to generate the cap hardening curve .

The mean effective stress is related with the plastic volumetric strain with the following equation

$$\varepsilon_v^p = \frac{\lambda - \kappa}{1 + e_0} \ln \frac{p'}{p'_0} = \frac{C_c - C_s}{2.3(1 + e_0)} \ln \frac{p'}{p'_0} \quad \& \quad x = \frac{C_c - C_s}{2.3(1 + e_0)}$$

The working load is taken from load test

DIRE DAWA project

to center	epth (z	Layer thickness	N70	N30	ν	γ	vertical stress	P_o	3140	E_s	E	K_o	σ_1	σ_3	P	x
1.5	3	3	16	37.33	450	17	51	25.5	666.33	85817.45	63750.1	0.43	691.83	297.49	428.934	0.0158
4	5	2	36	84	450	17	85	68	93.702	48562.31	36074.86	0.36	161.7	58.213	92.7094	0.0158
7.5	10	5	47	109.7	450	17	170	127.5	26.653	53774.18	39946.53	0.44	154.15	67.827	96.6026	0.0146
11.25	12.5	2.5	50	116.7	517	17	212.5	191.3	11.846	78078.95	58001.51	0.44	203.1	89.362	127.273	0.0146
14.25	16	3.5	44	102.7	450	17	272	242.3	7.3831	71063.64	52790.13	0.46	249.63	114.83	159.765	0.0114
18	20	4	52	121.3	450	17	340	306	4.6273	79566.06	59106.22	0.43	310.63	133.57	192.589	0.012

layer 1		layer 2		layer 3		layer 4		layer 5		layer 6	
e^p_v	p	e^p_v	p	e^p_v	p	e^p_v	p	e^p_v	p	e^p_v	p
0	25.5	0	68	0	10	0	213	0	242.2	0	578
0.01596	70	0.0259	350	0.0414	170	0.0073	350	0.014	800	0.00596	950
0.02691	140	0.0327	540	0.063	750	0.0174	700	0.016	1000	0.00658	1000
0.04138	350	0.0383	770	0.0672	1000	0.0226	1000	0.018	1200	0.00877	1200
0.05234	700	0.0425	1000	0.0774	2000	0.0327	2000			0.0149	2000
0.06329	1400			0.0934	6000	0.0487	6000			0.02808	6000
0.06892	2000			0.1009	10000	0.0562	10000			0.03421	10000

GIDABO PROJECT

$\varnothing$	Depth to center	Depth (Z)	Layer thickness	N70	N30	ν	γ	vertical stress	P_o	5858	E_s	E	k_0	σ_1	σ_3	p	x
29.697	4.5	9	9	12	28	162	17.8	160.2	80.1	138.12	19923.35	14800.2	0.62	218.22	135.3	162.9	0.1223
30.875	34.5	60	51	14	32.67	450	16	976.2	568.2	2.3499	108126.1	80322.27	0.37	570.55	211.1	330.9	0.0718

layer 1	
e^p_v	p
0	80.1
0.0846	160.2
0.2106	450
0.2729	750
0.308	1000
0.349	1400
0.3926	2000
0.442	3000

layer 2	
e^p_v	p
0	568.2
0.038858	976.2
0.053678	1200
0.0697	1500
0.090355	2000
0.119468	3000
0.140123	4000

k.k. project

vertical stress	γ	depth	x	Depth to center	lnp$_o$	P$_o$	K$_o$	σ_1	σ_3	5500	p	x	Es		E	eo = 1.21
153	17	9	0.017	4.5	4.337	76.5	0.5	206.1818	103.1	129.68	137.4545	0.017	74599	4.5	55417	
272	17	16	0.017	12.5	5.666	289	0.43	305.8068	131.5	16.807	189.6002	0.017	80323	12.5	59669	
321.2	16.4	19	0.055	17.5	6.059	428	0.56	436.3749	244.4	8.5749	308.3716	0.055	1E+05	17.5	81846	
433.2	16	26	0.017	22.5	6.271	529	0.38	534.3873	203.1	5.1873	313.5072	0.017	1E+05	22.5	78004	
552.2	17	33	0.136	29.5	6.57	714	0.49	716.7176	351.2	3.0176	473.0336	0.136	1E+05	29.5	109721	
671.2	17	40	0.136	36.5	6.794	892	0.54	894.1711	482.9	1.9711	619.9587	0.136	2E+05	36.5	125220	

layer 1	
e	p
0	76.5
0.0103	140
0.0187	230
0.0259	350
0.0332	540
0.0393	770
0.0444	1040
0.0555	2000
0.0624	3000
0.0673	4000
0.0711	5000
0.0742	6000
0.0828	10000

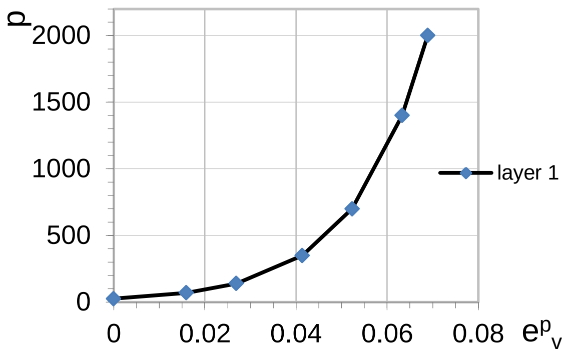
for layer 2	
e	p
0	289
0.003256	350
0.010627	540
0.016659	770
0.021769	1040
0.026204	1350
0.030023	1690
0.032886	2000
0.04467	4000
0.051562	6000
0.060247	10000

layer 3	
e	p
0	427
0.01	500
0.03	770
0.05	1040
0.06	1350
0.08	1690
0.08	2000
0.12	4000
0.15	6000
0.17	10000

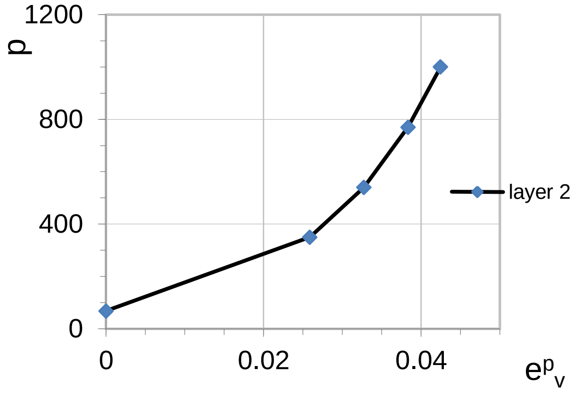
layer 4	
e	p
0	529
0.002	600
0.007	800
0.011	1040
0.016	1350
0.023	2000
0.034	4000
0.041	6000
0.05	10000

layer 5	
e	p
0	713
0.015658	800
0.046005	1000
0.070801	1200
0.140273	2000
0.289685	6000
0.359157	10000

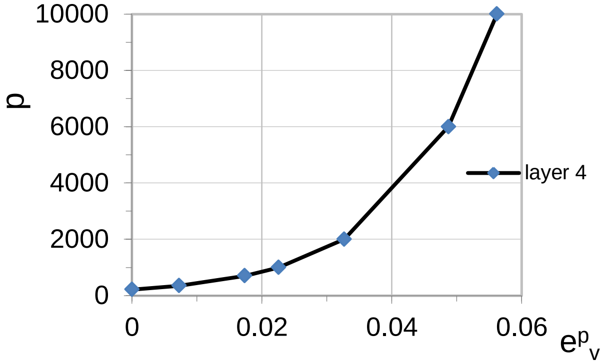
layer 6	
e	p
0	893
0.0084	950
0.0154	1000
0.0402	1200
0.1097	2000
0.2039	4000



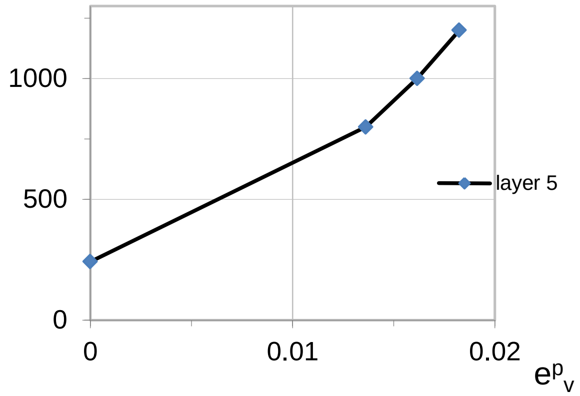
Cap hardening curve for soil layer 1 Dire Dawa



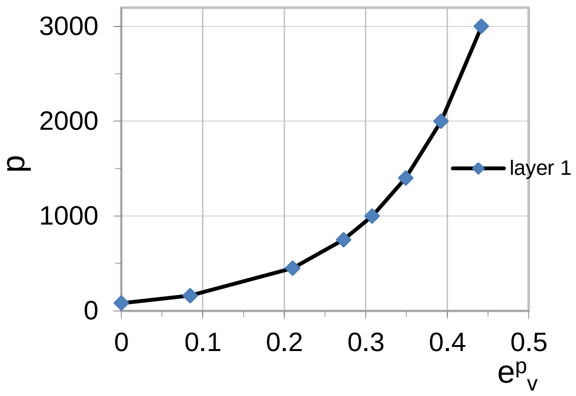
Cap hardening curve for soil layer 2 Dire Dawa



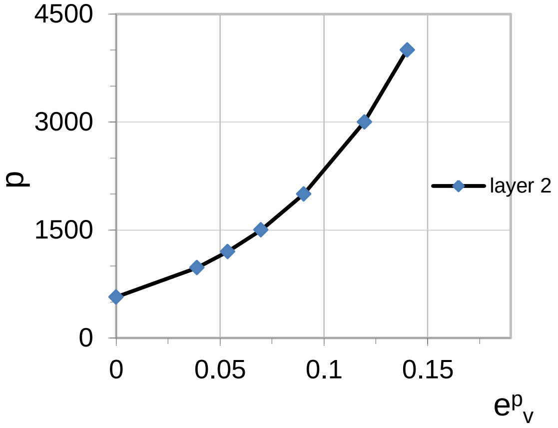
Cap hardening curve for soil layer 4 Dire Dawa



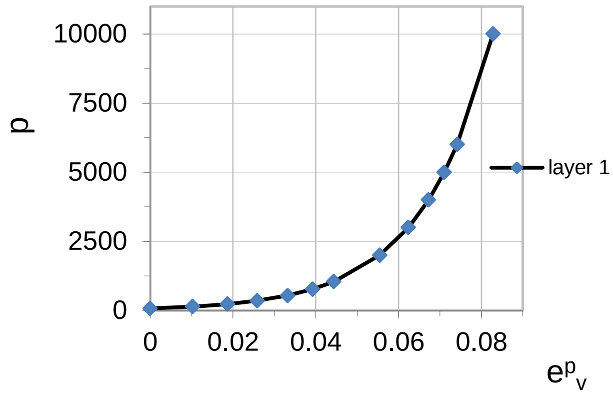
Cap hardening curve for soil layer 5 Dire Dawa



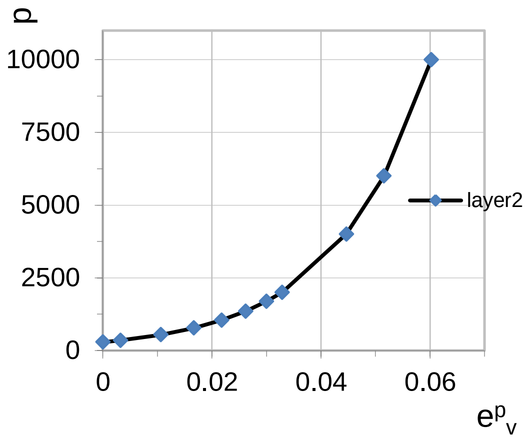
Cap hardening curve for soil layer 1 Gidabo



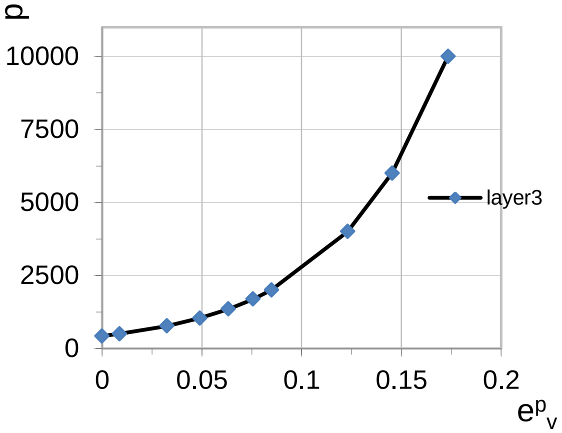
Cap hardening curve for soil layer 2 K.K.



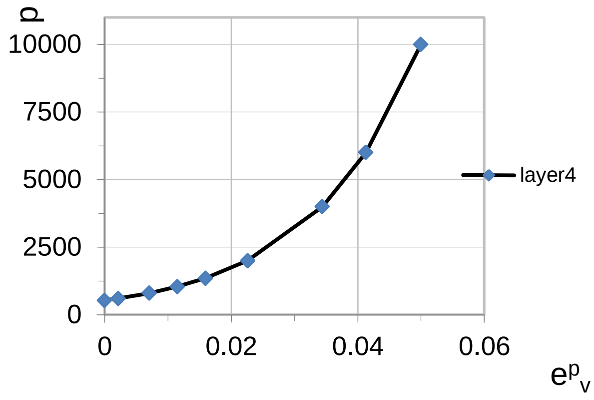
Cap hardening curve for soil layer 1 K.K.



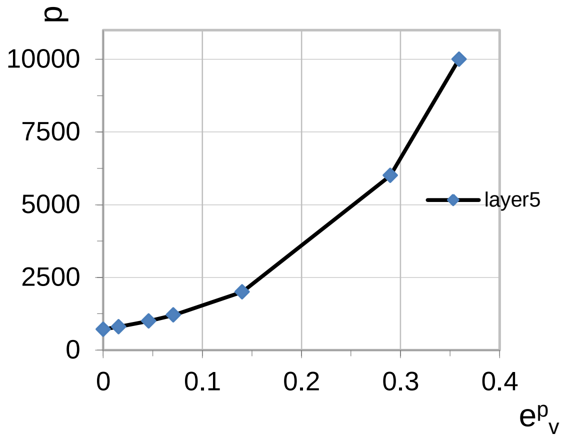
Cap hardening curve for soil layer 2 K.K.



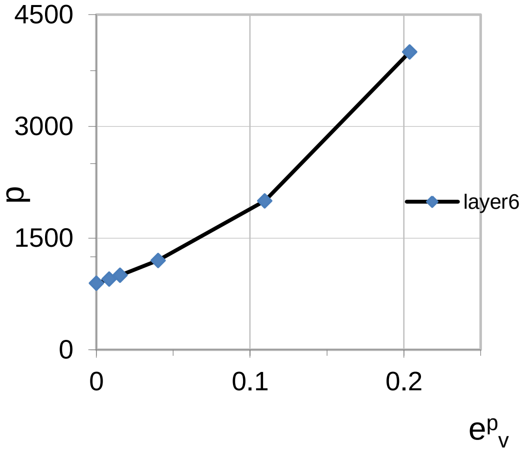
Cap hardening curve for soil layer 3 K.K.



Cap hardening curve for soil layer 4 K.K.



Cap hardening curve for soil layer 5 K.K.



Cap hardening curve for soil layer 6 K.K.