SETTLEMENT ANALYSIS OF SINGLE, STATIC AXIALLY LOADED STRUCTURAL PILE IN CLAY SOIL ON A NATURAL AND LIME TREATED CONDITION

MSc. THESIS

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ADVISORS APPROVAL SHEET

This is to certify that the thesis entitled “Settlement Analysis of Single, Static Axially Loaded Structural Pile in Clay Soil on A Natural and Lime Treated Condition” submitted in partial fulfillment of the requirements for the degree of Master's with specialization in **Geotechnical Engineering**, the Graduate Program of the School of **Civil Engineering**, and has been carried out by **Pawlos Fikru,** Id No **PGGeo/031/09**, under our supervision. Therefore, we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis proposal to the department.

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ABBREVIATIONS/ ACRONOMY

ASTM	American Society for Testing and Materials
CL	Lean Clay
C	Cohesion of Soil
Cu	Undrained Cohesion of Soil
D60	Diameter on the Cumulative Size Distribution Curve where 60 Percent of Particles are Fines Plasticity Index
Ft	Foot
In	Inch
Kpa	Kilopascal
KN/rad	Kilo Newton per Rad
Lb	Pound
LI	Liquidity Index
LL	Liquid Limit
L/FA	Lime or Fly Ash
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plasticity Limit
USCS	Unified Soil Classification System
UCS	Unconfined Compressive Strength
Yd	Yard
ω	Moisture Content
ϕ	Internal Friction Angle

ABSTRACT

Piles are deep foundations, necessary when the bearing capacity of shallow foundations is not enough to ensure the support of the superstructure. When the underground soils are very weak the structure undergoes high settlement and that leads to high repairing cost. Therefore the purpose of this thesis is to analyze the settlement of single piles under axially loading of clayey soil with a consideration of lime treated and untreated conditions. Comparison of the two outcomes is as well the objective of this thesis. The proposed activity in order to intend the purposes includes a literary review on settlement analysis, soil treatment method and the analysis part computed by axisymmetric elastic modelling so called Plaxis are used. The finite element-based program of Linear Elastic (LE) model used for the pile since it is used to model stiff objects based on Hookes's Law of isotropic elasticity. The analyzed piles are cylindrical and isolated and the loading is performed axially. These conditions allow for the use of symmetry; thus, the single piles behavior is modelled in 2D. Different researches show us that stabilizing a soil will reduce its settlement and the same is true for this research also, as the depth of treatment increased from 4m to 16m for the applied 1300KN load the settlement is reduced from -2.74cm to -1.17cm and while in the scenario of increasing the diameter from 0.6 to 0.8m maintaining the length of pile 16m the settlement obtained for the applied load is -7.14cm and -1.42cm but it's observed that there will be a significant change in reduction of magnitude of settlement by increasing the piles length maintaining the diameter minimum than both treating the soil and increasing the piles diameter. Therefore the conclusion here can be given that instead of treating the soil and increasing the piles diameter, elongating the length of the pile is recommended for the better reduction in settlement of the pile .

Key Words: Axially loaded Single Pile, Plaxis, Stabilization, Settlement

1. INTRODUCTION

1.1 General

Foundations provide support to the structure, transfers the loads from the structure to the soil. But the layer at which the foundation transfers the load shall have an adequate bearing capacity and suitable settlement characteristics. One type of a deep foundation is called pile foundation. According to Wulandari (2013) Piles are deep foundations, necessary when the bearing capacity of shallow foundations is not enough to ensure the support of the superstructure. This superstructure results in vertical forces, due to its weight as well as additional loads, which are axially transferred to the pile and, through its shaft and base, to the soil, possibly reaching a stiffer layer.

The analysis of the load transfer mechanism in single pile under axial loading is therefore an important basis for deep foundation design. It is very important that the physical interaction between pile and soil is carefully studied. The settlement analysis is also fundamental, for the maximum allowable settlement of a foundation is often the most significant criterion in its design. Thus, it should be estimated accurately.

The behavior of single piles under axial loading, as far as load distribution and settlement along the pile are concerned, have been analyzed through numerous methods. They can be divided in to three main categories, according to (Poulos & Davis, 1980):

1) Load-transfer methods, which involve a comparison between the pile resistance and the pile movement in several points along its length;
2) Elastic theory-based methods, which employ the equations described in(Mindlin, 1936) for surface loading within a semi-infinite mass (such as the Poulos and Davis method), or other analytical formulations that impose compatibility between the displacements of the pile and of the adjacent soil for each element of the pile (such as the Randolph and Wroth method);
3) Numerical methods, such as the finite element method.

In this thesis, Numerical method results are used to compare with the results of axially loaded pile existed at different scenario, using a finite element method program, Plaxis 2D version 8. Numerical methods are powerful and very useful tools when used carefully and calibrated with the appropriate tests.

Settlement of piled foundation under axial loads is an important aspect of the problem, however, settlement of piled foundation is difficult to analyze because of the complex interaction between the piles, the raft and the soil. There are numerous solutions for the calculation of the settlement of piled foundations established on different approaches. Numerical analysis based on finite element method (FEM) is an advanced method which can take into consideration most of the factors involved (Erdal, 2003).

In many areas of our country and though out the world, some soils make the construction of foundation difficult because of their poor performance, so that a processes by which the property of soil are improved to meet the construction requirement which is called stabilization is conducted on the poor performance soil.

Typical foundations on soft clays include bedrock-socketed drilled shafts and piles. However, the main concern arises when the depth of bedrock is very deep. Bridge abutments and piers transfer significant load to the underlying foundation sub grade and, in most instances, are socketed in bedrock. Attaining bedrock-embedded deep foundations in such instances is impossible (when trying to reach on the bedrock is uneconomical and end bearing pile are not possible to install); hence this research target novel method of improving soft clay.

The effect of lime treatment on clayey soil and the corresponding strength of the pile foundation will be investigated in this study. An effort was made to investigate lime pressure injection method and associated improvements in soil properties, as well as the performance of the pile foundation under axial loading. Based on laboratory tests conducted on natural and treated clay soils, improvement in soil properties was documented and the results later utilized in finite element analysis, using Plaxis 2D.

1.2 Statement of the Problem

Foundations constructed on soft soil undergoes large deformations and associated movements during service period of the structure. This often results in high repairs cost. Working loads from structures built on compressible soil raise several concerns, including bearing capacity failures, differential settlements and instability.

Geotechnical solutions to address the mentioned concerns include excavation and soil replacement, ground improvement, physical stabilization and use of deep foundations, and other remediation's. These alternative engineering techniques have been practiced for more than two decades (Han and Collin, 2005). Improvement of in situ soil strength at the site can be achieved by different stabilization techniques, amongst which chemical stabilization technique is one of them.

Many research are already done on stabilizing a soil. Among these researches (Abdusemed, 2014)) analyses the settlement of drilled shaft foundation using lime slurry pressure injection, and he found a great settlement difference between the treated and untreated soil.

Almer (2001) prepared a study aiming in increasing the bearing capacity of a pile foundation by three types of grouting (permeation, compaction, and jet), he used field piles made of steel and timber, and he found that grouting is an effective way for foundation renovation and increasing existing pile capacity.

Currently our country Ethiopia is developing at high growth rate, following this development different kind of construction works are being carried out, so to have a stable and safe construction, good geotechnical investigation are required and remediation's should be taken for poor kind of soils, in related to this different remediation technology and new innovations should be practiced. So loss of time, money and human power resources can be reduced by using new technology.

The development of our country is being upward and high raise buildings which have different kind of structural pile foundation and shoring are being constructed, here the main thing we should have to know is, soil stabilization using grouting technology is done only for shoring pile to prevent lateral displacement. But this paper deals with comparison of magnitude of settlement of a pile in a lime treated soil at different treatment depth with untreated soil by differing the length and diameter of the pile.

While pile design is repeatedly done without explicit settlement checks, analyses that can accurately calculate settlement for a given load will offer opportunities for more cost-effective design in the future (Hoyoung, 2008). In addition this paper will examine how the settlement of piled foundation reduced when the soil is stabilized and how the stabilization of the ground conducted through grouting.

1.3 Research Objectives

1.3.1 General Objective

The aim of this research is to compare the magnitude of settlement of pile installed in untreated soil having larger length and diameter with a pile installed in stabilized soil having smaller length and diameter where the bed rock is unachievable.

1.3.2 Specific Objectives

- To compare the changes in settlement of pile on lime treated and untreated clay soil.
- To evaluate the effect of stabilization on settlement of the pile by differing the depth of stabilized soil.
- Evaluating three different methods (treating the soil, increasing the diameter of the pile and elongating the length of the pile) used to increasing the bearing capacity and decrease settlement to recommend the better one.

1.4 Research Questions

- Is stabilization effective in reducing the settlement for pile rather than increasing the length and diameter of the pile?
- How to determine settlement analysis of single axially loaded pile?
- Why do we need a chemical test pH for soil?

1.5 Significant of the Study

The development of our country in the field of construction is appreciable, different high-rise buildings, bridges, dams and tunnels are being constructed and will continue but there exist different kind of challenges may occur during construction, this difficulties are being solved by different mechanisms, and this research also deals with one of problem solving mechanism happened on structural piles installed in compressible soil, there are different ways of treating compressible soil like Removal and replacement, Vibrocompaction and chemical stabilization (C.C. Swan, 2008). So this research prefers to use chemical stabilization, based on this we can either recommend the soil stabilization or increasing the dimensions (length and diameter) of the pile on untreated soil.

In addition this thesis will be used as a reference for the upcoming researches on this area and it also gives some awareness to geotechnical world to see the stabilization of soil in settlement considerations.

1.6 Scope and Limitations of the Study

This study has been supported by a series of laboratory tests and FEM using Plaxis 2D. However, the findings of the research are limited to only one soil sample considered in the research which is assumed to be compressible clay with no bed rock available, (It is said to be compressible if a building, or other applied load, causes the water in the pore space to be squeezed out, causing the ground to decrease in thickness (compress). This deformation of the ground is usually a one-way process that occurs during or soon after construction. Compressible materials, such as peat, undergo both primary and secondary settlement. Primary settlement takes place in days and occurs due to water expulsion or loading; secondary settlement may last years and is due to the restructuring of the material. (British Geological Hazard, 2014)).

- Focus on the concrete pile type, and the load caring capacity is only by friction.
- Focus will be on settlement analysis pile in clay soil only and not a multi layered soil.
- Focus on piles only loading by superstructure loads will be taken into account and not loading by soil movements (lateral loads).
- Soil stabilization extended all the way to the boundary of the PLAXIS model for the treated soil calculation

2. LITERATURE REVIEW

2.1 Foundation

Foundation is integral part a structure which holds and transmits superstructure and other loads to a firm strata of soil at acceptable settlement. They are two basic types of foundation: shallow foundation - footings/raft and deep foundation - piles, piers or caissons foundation. According to (Hyeong Joo Kim, et al (2007)), shallow foundations width are often greater than their depth, while deep are those which ratio of depth to breath is greater than or equal to one. The type to use depends on load to be transmitted, soil bearing capacity, slope of the soil, geotechnical conditions etc. However, to perform satisfactorily, its designed should meet two principal requirements; ultimate limit states and serviceability limit states (Hyeong Joo Kim, et alm 2007).

2.2 Building Foundations on Soft Cohesive Soil

The main purpose of a foundation is to transmit loads to the underlying soil. This results in a soil-structure interaction. The foundation method which is most suitable depends on the properties of the soil and the functional requirements of the structure. Since structural parts of a building often have greater stiffness and strength than underlying soil, support is generally done by the use of shallow foundations (J. Paul Juyer, P.E., R.A. 200). An example of this is enlarged ground plates (or slabs) which distributes the loads over a larger area. However, if the soil stratum close to the surface is not capable to give sufficient support, deep foundations as piles or caissons may be used to transfer the loads to greater depths, where the soil often has higher strength and stiffness (Craig and Knappet, 2012).

2.2.1 Piled Foundations

In some situations, use of shallow foundation is uneconomical and unsafe. When design loads are large, near surface soils have low stiffness, soil layers are inclined, settlement sensitive structured are to be build, in marine environments where tidal, wave or flow actions are expected. In such situation(s) deep foundation becomes necessary to have stable and safe substructure (Knappett & Craig, 2012).

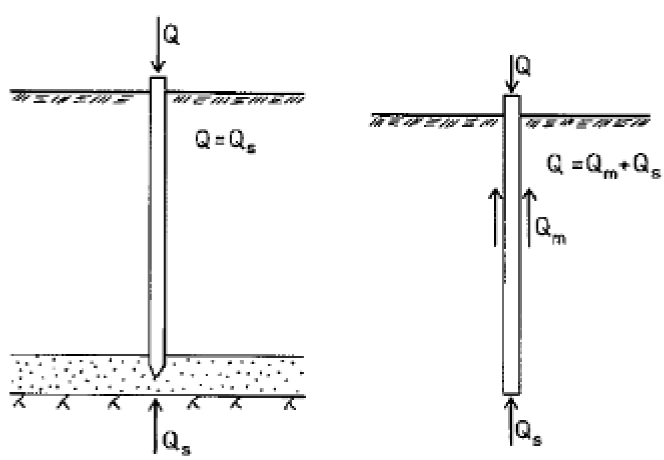
When designing foundations with piles, the two main aspects to take into consideration are bearing capacity and settlements. For a foundation on clay, settlements are almost exclusively the limiting aspect (Jendeby, 1986). The main reason for using piles in a foundation design is to transfer applied loads to a greater depth of the soil due to their stress history, normally have higher strength and stiffness compared to more shallow layers and therefore would have greater resistance to settlement. When a pile is subjected to a vertical force at the top of the pile, the pile head, shear stresses are mobilized in the ground that surrounds the pile. If the created shear stress exceeds the shear strength of the soil, ground failure will follow. Two different parameters decide the capacity:

1. Shear stress that is developed in the soil around the pile toe

2. Shear stress that is developed at the interface between the shaft and the surrounding soil.

This leads to two types of pile classification; end bearing piles and shaft bearing piles, see Figure2.1

However, this classification describes special cases. In the normal case, the pile resistance be contingent on both end and shaft resistance (Craig and Knappet, 2012).



(a) End bearing pile (b) Shaft bearing pile

Figure 2.1: Pile classification

Source: (Craig and Knappet, 2012).

The usual long and slender dimension of a pile makes axial loading the most beneficial way to use them. The failure load of a pile is defined as the load acting on a pile when the soil no longer can carry the transmitted load. The creep load of a pile is defined as the biggest load that can be applied to the pile, without attaining a substantial increase of settlements (Holm and Olsson, 1993).

2.2.2 A Friction Pile

Utilizes the shaft bearing principle, according to Figure 2.1(b), a foundation which includes friction piles can act differently depending on the duration of the load. Thus, the bearing capacity should be controlled with regards to both short-term and long-term loads. For the settlements calculation, only the long-term load is considered in a normal case (Eriksson et al., 2004).

2.2.3 Settlement Analysis of Axially Loaded Pile Foundation

Major research efforts have been dedicated towards the investigation of the capacity of axially loaded piles and shafts. The empirical results of some of these studies have aided as the fundamental formulations in the development of computer programs and software that calculate the ultimate capacities of piles and shafts in homogeneous and layered soils.

In the research conducted by (Linas Gabrielaitis, et al. 2013) they describes the estimating settlements of bored piles foundation on the site of the Elektrenai power plant, Lithuania. The bored piles foundation supports equipment of the power plant consisting of the gas turbine, the steam turbine and the generator. The piling solution was accepted for the following reasons:

i) The insufficient capacity of the soil to support excessive stresses over it;
ii) High requirements of slab settlements and bearing capacity with regard to the main equipment in power plant.

For settlement calculation the researchers use five different methods employed, such as Bowles, Schmertmann methods, the method described in EN 1997-2, NEN 6743 and finite element method applied in Plaxis 3D Foundation package.

Soil properties were estimated from site investigation of the Elektrenai power plant and soil exploration program according to Lithuanian standards. For such structure, foundation settlement should not exceed 16 mm. Because the Elektrenai power plant has high reliability requirements, piles diameter of 880 mm and 29 m long were finally carried out to endure overall loads.

Table 2.1: Comparison of settlement analysis result.

Method	Pile length, m	Immediate Settlement, mm
Bowles	29	15.9
Schmertmann	29	7.2
EN 1997-2	29	6.5
NEN 6743 Limit state 1B	29	5.3
NEN 6743 Limit state 2	29	5.4
FEM (Plaxis 3D Foundation)	29	5.9

The pile settlement analysis was performed employing most widely used standards and approaches, namely EN 1997-2, NEN 6743, Schmertmann and Bowles, and finite element method. The comparative analysis of five methods indicates that the settlement values are similar for all considered methods. The largest value of bored pile settlement was obtained employing analytical Bowles method. The reliable results of pile settlements were obtained from finite element method of Plaxis, Foundation package and employing method using Pile CPT Package from Geo5 software. (Linas Gabrielaitis et. Al, 2013)

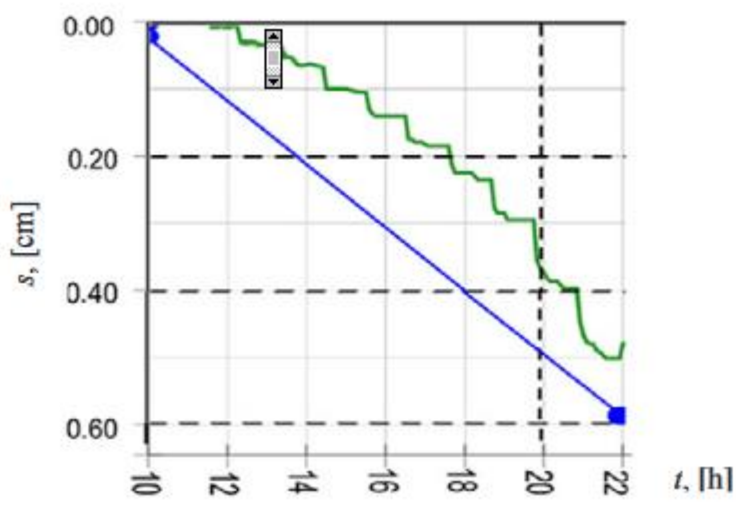


Figure 2.2: Time – settlement curve of static load pile test
Source: (Linas Gabrielaitis et. Al, 2013)

Fig. 2.2 shows a shape of the time-settlement curve for the static load pile test. The green curve represent experimental results of static load pile test, the blue curve – the calculations results obtained by employing finite element method of Plaxis Foundation package.

Finally from this research we can detect that the finite element software Plaxis is the most reliable software for conducting settlement analysis of pile foundation, as discussed above the reliable results of pile settlements were obtained from finite element method of Plaxis 3D.

In the research conducted by Abdusemed (2014), settlement analysis of a drilled shaft foundation both on lime treated and untreated compressible soil, he used both analytical and numerical analysis method, assuming that the entire strata was a homogeneous clay layer and the maximum settlement of the drilled shaft foundation was calculated under a static load of 1000KN. The diameter of the drilled shaft was kept 1m. with 5m, 10m, 15m and 20m embedment length from the surface.
During the analytical settlement analysis method he used the following formula

$$S = S1 + S2 + S3$$

Where, S = total pile settlement, S1 = settlement of drilled shaft, S2 = settlement of drilled shaft caused by the load at the pile point, S3 = settlement of drilled shaft caused by the load transmitted along the shaft

If the drilled shaft material is expected to be elastic, the deformation of the drilled shaft can be evaluated using the fundamental principles of mechanics of materials:

$$S_1 = (Q_{WP} + \xi Q_{WS}) / A_P E_P$$

Q_{WP} = load carried at the pile point under working load condition;

Q_{WS} = load carried by frictional (skin) resistance under working load condition;

A_P= area of pile cross section;

L = length of pile; E_P = modulus of elasticity of the pile material;

The magnitude of ξ will depend on the nature of unit friction (skin) resistance distribution of f is uniform or parabolic, $\xi = 0.5$. However, for triangular distribution of f, the magnitude of ξ is about 0.67 (Vesic, 1977).

$S2 = q_{wp}D\,(I - \mu s_2)\,I_{wp} / E_S$

Where D = width or diameter of pile

q_{wp}= point load per unit area at the pile point $=Q_{wp}/A_p$;

E_s= modulus of elasticity of soil at or below the pile point;

μ_s = Poisson's ratio of soil; I_{WP}= influence factor.

$S_3 = Q_{WS}/PL)\,D\,(1 - \mu S_2)\,I_{WS}\,E_S$

Where p = perimeter of the pile

L = embedded length of pile, I_{WS} = influence factor

And for the numerical analysis case he used Plaxis 2D software.

Finally the researcher obtained the result shown below in the table

Table 2.2. Settlement analysis numerical results.

Scenarios		Settlement (cm)
Diameter= 1m	L =5m	26
	L =10m	21
	L =15m	20
	L =20m	19
Diameter= 1m	5m Treated	11
	10m Treated	10.5
	15m Treated	10.1
	20m Treated	10

The final out puts from Abdulsemed's research is summarized in table 2.2

2.3 Soil Strength and Constitutive Model

Some researchers have characterized soil as an elasto-plastic material and made efforts at defining constitutive models to describe the stress-strain response (Chen and Saleeb, 1983). There has been no firm agreement on which constitutive model is the best to use. A widespread constitutive model used to represent sand response under loads is the linear elastic model during loading, with either Mohr-Coulomb or Drucker-Prager failure cap models to define the failure state (Wang and Sitar, 2004; Potts and Zdravkovic, 1999 and 2001).

This research used a finite element computer package Plaxis 2D to model the response for an axially loaded pile. The numerical pile analysis is shown in Chapters 4. The results from Plaxis 2D are dependent on the choice of constitutive model to idealize the material behaviour of clay. The following sections will discuss the various constitutive models most commonly used, and the conceivable correlations to determine the constitutive parameters. A limited parametric analysis to examine the reliability of the available parameters is also presented.

2.3.1 Constitutive Models

The increase in technology of recent years has made it possible for more complex soil problems to be solved using numerical methods, such as the finite element method. This has resulted in a higher demand for researchers to develop more comprehensive constitutive models for soil behaviour.

2.3.2 Types of Constitutive Models

The constitutive models available for describing the stress-strain behaviour of sand fall into one of two categories (Chen and Saleeb, 1983). The first category of constitutive models was derived under the assumption that the soil material behaves in an isotropic manner, in which the mechanical behaviour of the material is identical in all directions.

The second category of soil constitutive models falls under anisotropic behaviour, in which the material behaviour in at least two directions is different. (Kate Johnson, 2005)

The common constitutive models available for soil in each category are listed below:

1) Isotropic constitutive models

   ⇒elastic

   - linear elastic (i.e. Hooke's law)
   - non-linear elastic

   ⇒ elasto-plastic

   - Mohr-Coulomb
   - Drucker Prager

2) Anisotropic constitutive models

All of the above constitutive models have been placed in order of complexity. The most complex models are anistropic, which require a vast amount of input data from laboratory testing. The increase in accuracy using an anisotropic constitutive model could be lost due to crude soil testing and laboratory procedures. A rigorous amount of soil tests can lead to a substantial increase of costs for geotechnical engineers in the field. Therefore, the anistropic constitutive models are difficult to employ resulting in them rarely being used. (Kate Johnson, 2005)

Many researchers have explored constitutive models and found the use of isotropic models, such as linear elastic with Mohr-Coulomb or Drucker-Prager is sufficiently accurate (Chen and Saleeb, 1983; Hibbitt et al., 2001). In the past, linear elastic constitutive models without the use of a cap model have been commonly used in developing pile design methods (Vesic, 1977). The differences between pure linear elastic and non-linear elastic constitutive models are discussed below.

2.3.2.1 Linear Elastic Constitutive Model

Various researchers have explored the stress-strain behavior of soil and found it undergoes elastic strain under small stresses. As the stress level increases the soil will develop plastic strains until ultimate shearing capacity is attained. The elastic stress-strain behaviour may be represented by one of two approaches, linear-elastic as discussed in this section or non-linear elastic as presented in Section 2.3.2.2.

In the engineering world, the linear elastic constitutive model (Hooke's law) is probably the most common model used to approximate the stress-strain relationship of a material. Hooke's law relates the stresses as a linear function of strains in three-dimensional space through two constants, Young's modulus (E) and Poisson's ratio (ν), in the following manner:

$$\sigma x = \left(\frac{E}{(1+\upsilon)(1-2\upsilon)}\right)\left(\varepsilon x(1-\upsilon)+\upsilon(\varepsilon y+\ \varepsilon z)\right)$$

$$\sigma y = \left(\frac{E}{(1+\upsilon)(1-2\upsilon)}\right)\left(\varepsilon y(1-\upsilon)+\upsilon(\varepsilon x+\ \varepsilon z)\right)$$

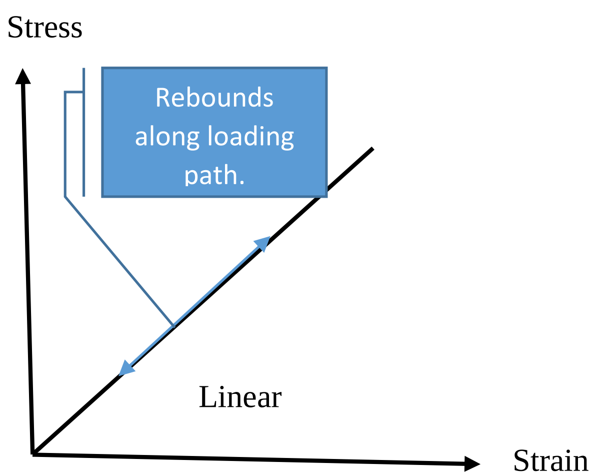
$$\sigma z = \left(\frac{E}{(1+\upsilon)(1-2\upsilon)}\right)\left(\varepsilon z(1-\upsilon)+\upsilon(\varepsilon x+\ \varepsilon y)\right)$$

where: σx, σy, σz = normal stress in x, y and z directions respectively, and

εx, εy, εz = normal strain in x, y and z directions respectively

2.3.2.2 Non-Linear Elastic Constitutive Model

Non-linear elastic models are more complex to define than their linear elastic counterparts. This is because the relationship between the stress and strain at various stress levels is not constantly proportional. Some typical linear elastic and non-linear elastic stress-strain plots are shown in Figure 2.2.



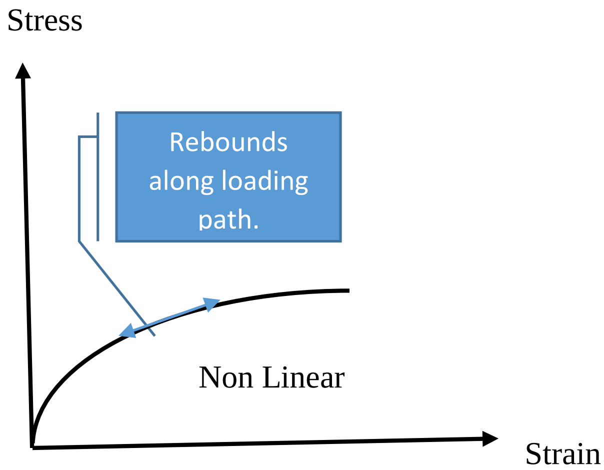


Figure 2.3: Typical linear elastic and non-linear elastic soil response

As seen in the diagram above, if a material is purely elastic it will undergo elasticstrains only, and will rebound along the same stress-strain path once unloaded. (Kate Johnson, 2005)

2.4 Soil Improvement

Improving in situ soils by using additives is usually referred to as stabilization. The soil at a construction site may not always be totally suitable for supporting structures, in many plastic clays swell considerably when water is further added to them and then shrink with the loss of water.

Certain additives such as lime, bitumen, fly ash and cement etc. are added onto the soil at site to increase its characteristics due to this soil properties such as strength, compressibility, hydraulic conductivity, workability, swelling potential, and volume change tendencies may be altered by various soil stabilization methods.

Soft saturated clay layers are often encountered at shallow depths beneath foundations. Depending on the structural load and the depth of the layers, large consolidation settlement may occur. Hence soil-improvement techniques may be required to decrease settlement. (Mohammed Bilal et al, 2016).

There are numerous techniques for soil stabilization. These methods mainly depend on the nature of strata and the purpose of improvement. Techniques for soil stabilization can be broadly classified as

1. Soil improvement using additives
2. Soil improvement using mechanical methods
3. Soil improvement without using admixtures
4. Soil improvement using thermal methods. Most of these improvement techniques tend to increase shear strength, reduce permeability and reduce compressibility. (Mohammed Bilal, 2016).

2.4.1 Lime

For the case of this research "lime" refers quicklime, hydrated lime, or hydrated lime slurry.
Quicklime (calcium oxide– CaO) is manufactured by chemically converting calcium carbonate (limestone – CaCO3) into calcium oxide.
Hydrated lime (calcium hydroxide – $Ca[OH]_2$) is formed when quicklime chemically reacts with water. It is hydrated lime that reacts with clay particles and permanently transforms them into a strong cementitious matrix, (Bulletin, 2004). Since this research is focusing on piles that resist load by skin friction the cementation property that is created due to the adding of lime will increase its shear strength.
Whenever the majority of the soils on a project have the following test results, consider using lime stabilization:

Plasticity Index> 15-18

Volume Change> 20-30%

Clay Content> 25-30%

When dealing with these kind of soils, always recommend lime stabilization to the outside edges of the shoulders.

2.4.1.1 Lime Stabilization

For the case of this research "lime" refers quicklime, hydrated lime, or hydrated lime slurry. Quicklime (calcium oxide– CaO) is manufactured by chemically converting calcium carbonate (limestone – CaCO3) into calcium oxide.

Hydrated lime (calcium hydroxide – $Ca[OH]_2$) is formed when quicklime chemically reacts with water. It is hydrated lime that reacts with clay particles and permanently transforms them into a strong cementitious matrix, (Bulletin, 2004).

The transformation of soil properties by adding lime, often alters the physical and chemical properties of the soil including the cementation of the soil particles. There are the two primary mechanisms by which lime alter the soil.

1. Rise in particle size by cementation, internal friction among the agglomerates, greater shear strength, reduction in the plasticity index, and reduced shrink/swell potential.
2. Absorption and chemical binding of moisture that will facilitate compaction.
3. Use of lime as a stabilizer enhances the long-term permanent strength, stability and stiffness particularly with respect to the action of water and frost especially in fine grained soils and sometimes in fine grained fractions of granular soils too. (Mohammed Bilal et al, 2016)

2.4.1.2 PH of Lime-Water Solutions

The pH of solutions at 25°C (77°F) rise sharply with the addition of very small concentrations of $Ca\ (OH)_2$. A concentration of only approximately 0.064 g/1 of hydrated lime will increase the pH of distilled water from 7 (neutrality) to beyond 11. From this point the pH rise with increased hydrated lime concentration is gradual. The pH of the solution peaks at approximately 12.454 at 25°C (77°F). Temperature is an important factor since rises in temperature reduce solubility of $Ca(OH)_2$ and, therefore, decrease the pH slightly. (Little; 1995). PH is measured because it's one of major soil properties that influence the soils ability to react with lime to produce cementitious material.

2.4.1.3 Effect of Lime Treatment on the Geotechnical Properties of Soil

The drying of wet soil and the increase in soil workability are credited to the immediate treatment, whereas the increase in the strength, durability and compressibility of the soil are related with the long-term treatment (Wild *et al*., 1996; Geiman, 2005).

3. MATERIAL AND METHOD

3.1 Description of the Study Area

Before selecting sampling areas, visual site investigation and information from construction firms were collected to consider the different soil types so soil sample for investigation was obtained from Addis Ababa around Casanchis. The Exact position of the sample from total station reading is $9^0$0’27” Northing and $38^0$42’50” Easting,

The locations for taking sample soils for test have chosen by considering two things which are the depth of the excavation was up to six meters from normal ground level and able to take clay sample immediately after excavation was done in order to know the exact moisture content of the soil.

Figure 3.1: Location of soil sample.

3.2 Sampling

In order to meet the desire objective of the study (which is to analyze the settlement of a pile foundation on both treated and untreated clay soil where the bed rock is unachievable), sample are taken and preserved carefully not to lose their moisture content.

Figure 3.2: Block sampling

After sampling and careful preservation soil different laboratory tests are conducted for two cases this are for untreated or control soil and lime-treated soil.

The amount or dosage of lime required for treating soil is obtained based on Eads and Grim PH test. "Standard Test Method for using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization" This test identifies the lime content required to satisfy immediate lime-soil reactions at high pH (about 12.4 at 25^0C), the long term effect of lime were briefly discussed on literature review part. After obtaining demand of lime, all soil tests done for the untreated sample will be conducted also for the treated soils for comparing the settlement of pile on both soil. .

3.3 Laboratory Investigations

In this sub title basic soil properties like specific gravity, sieve analysis, Atterberg limits, PH, standard proctor, Direct shear test, Consolidation and unconfined compression test, which are used as an input for the Plaxis 2D and comparison of settlement analysis will be discussed. Laboratory tests for determining the properties of the soil were carried out according to procedures of ASTM manual for soil testing, both for natural and lime-treated sample soils.

3.3.1 Natural Moisture Content

Moisture content is the ratio expressed as percentage of the mass of “pore” or free water in a given mass of soil. Determining the in-situ water content of the soil is important for the determination of other soil parameters which are important for the design of any civil engineering structures (Punimia et al, 2006).

The results of the natural moisture content of the sample soils under investigation are presented in the table below.

Table 3.1: Water content data

Depth(cm)	6m	
Container No	182	189
Wt of Wet Sample + Container(gm)	102.6	106.10
Wt of Dry Sample + Container(gm)	81.7	84.30
Wt. Of Water(gm)	20.9	21.8
Wt. Of Container (gm)	33.3	33.5
Wt of Dry Sample (gm)	48.4	50.8
Water Content (%)	43.18	42.91
Average Water Content (%)	43.047602004295	

3.3.2 Grain Size Distribution

The grain size distribution tests were conducted from a sample of soil located at a depth of 6m, since determining the soil gradation of sample is important for soil classification. From the fact that predominantly the samples have fine grain particles sieve tests were used for particle retained on sieve size 0.075mm to get the adequate degree of separation between grains. For particles which were finer than 0.075mm hydrometer tests were conducted. The test results are presented in tabulated form on the graphs below and for details graph and table's see Appendix-A.

Table 3.2: percentage of Grain size

Gravel (%)	0.00
Sand (%)	29.36
Fines (%)	70.64

From the table above it's already known that the soil is very fine and it can be silt or clay which is discussed later on soil classification part.

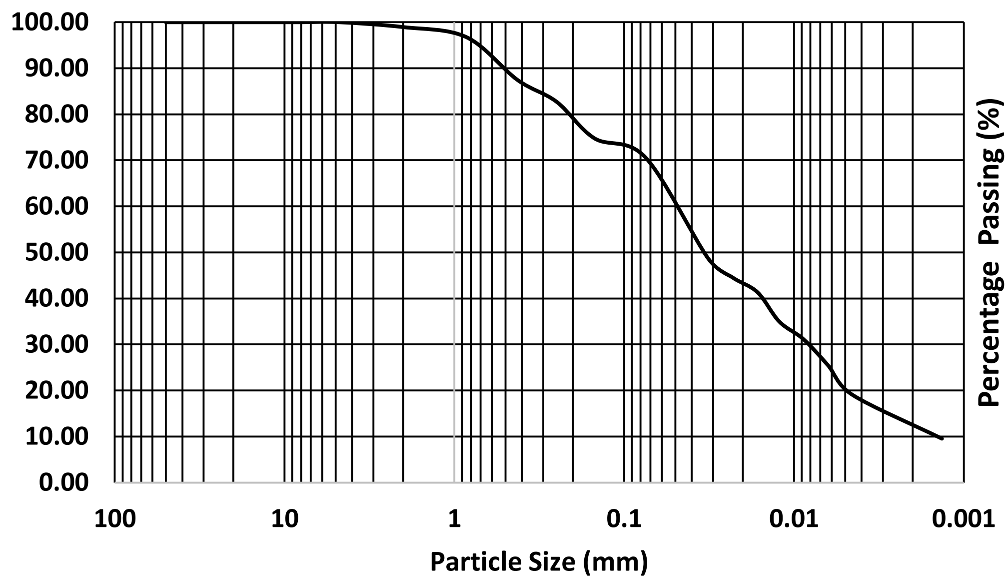


Figure 3.3: Grain size data

3.3.3 Specific Gravity

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The test is conducted according to Standard Test procedure of ASTM D 854 for Specific Gravity of Soil Solids by Water Pycnometer. Summary of the test results are presented in Table 4.3 and analysis of the results and equations are stated in Appendix-A

Table 3.3: Specific gravity data

Depth	6m	6m
Weigth of dry,clean pycnometer	54.7	42.7
Weight of pycnometer + water, wpw (g)	154.4	142.4
Observed temperature of water, Ti (°c)	21.6°c	21.6°c
Weight of pycnometer + soil + water, Wpws(g)	169.5	157.6
Temperature, Tx(°c)	22.6	22.5
Weight of pycnometer + water at Tx,Wpw(atTx)(g)	154.39	142.39
Specific gravity of soil at 20°c	2.52	2.55
Average specific gravity of soil	2.54	

3.3.4 Atterberg Limit Test

Atterberg Limits are arbitrary boundaries through which a soil passes from liquid to plastic, semi-solid and solid states. These boundaries are defined by moisture contents. It is the primary form of classification for cohesive soils. Besides its objective, it is useful to obtain basic index information about the soil used to estimate strength and settlement characteristics, while doing this research Atterberg Limits test was conducted both for lime treated and untreated soil.

In this study the Atterberg limit tests were carried out according to the procedure of ASTM D 4318 standard test methods for liquid limit, plastic limits and plastic indexes. The results are presented in Table 3.4.

Table 3.4: Atterberg limit values

Liquid Limit, LL (%)	45
Plastic Limit, PL (%)	25
Plasticity Index, PI (%)	20

It is well known that the liquid limit and the plasticity index together constitute a measure of the plasticity of a soil. Soils which possess large values of liquid limit and plastic limit are said to be highly plastic where as those with low values are said to be low plastic. As one can see from the table above, soil samples taken from 6m meters depth have high plastic index, that is one of the indication of soil suitable for the lime stabilization.

3.3.5 Soil Classification

A soil classification is a systematic method of categorizing soils into various groups and subgroups according to their probable engineering behavior without detailed descriptions. There are different classification systems available. For this research work the method used to classify soils of the study area was USCS.

3.3.6 Unified Soil Classification System (USCS)

This system requires liquid limit and plasticity index values of Atterberg limit test. According to the Atterberg limit test the soils classification are presented in the form of plasticity chart and in table below. The sample soil was fine grained soil which is more than 50% of the sample passes number 200 sieve therefore we can conclude that the sample is either clay or silt and when we sees the liquid limit it was about 45%, the sample is inorganic with plasticity index of 20 this mean that plot above "A" line, following all this steps the last phase is to classify the soil that its "CL" or lean clay.

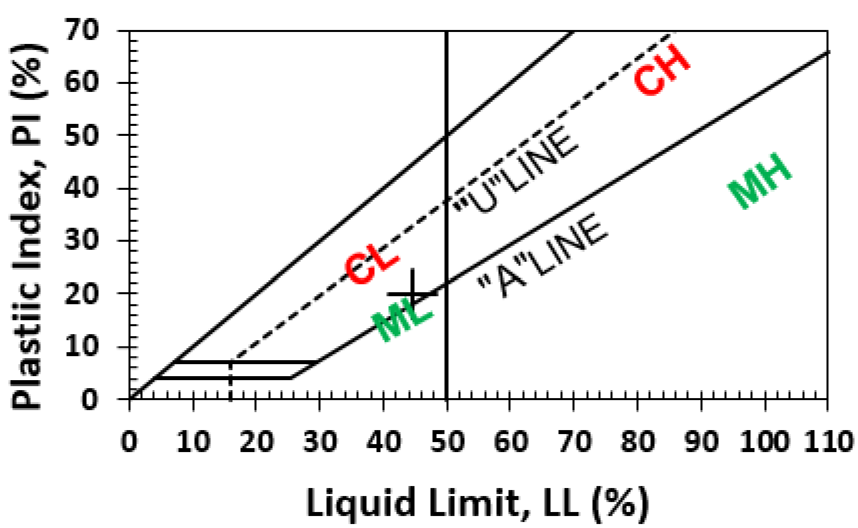


Figure 3.4: "A" line chart to show the specimens soil type

3.3.7 Compaction Test

The standard proctor compaction test was carried out to according to ASTM (D 698).

Standard Proctor compaction tests were performed to establish the moisture content–dry density compaction relationships of the soils as per ASTM D698. For the case of this research this test is conducted both for the untreated and lime treated soil. The standard compaction curve obtained for this soil is shown in Figure. This clay with no lime attained a maximum dry density of (1.15 g/cm^3) at 35.1 % moisture content and stabilized clay attain 0.968g/cm^3 at 32.5%.

Table 3.5: MDD and OMC of untreated soil

Optimum Moisture Content, OMC (%)	35.1
Maximum Dry Density, $\rho_{d, max}$ (g/cm^3)	1.15

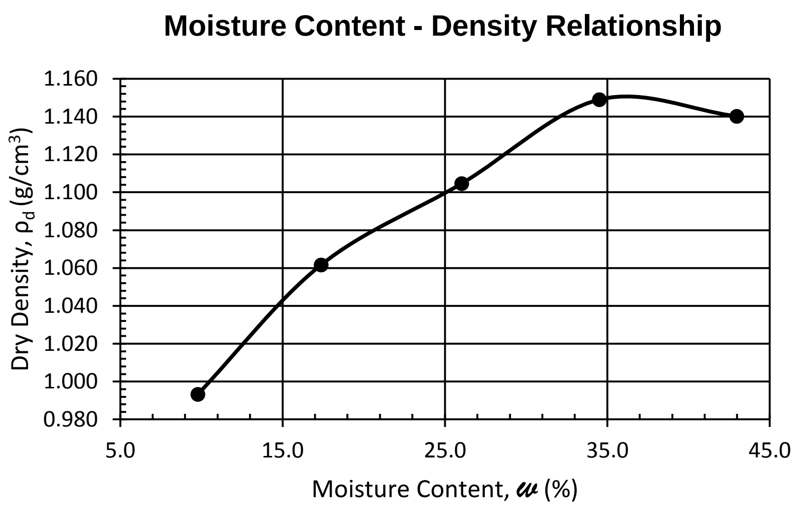


Figure 3.5: Graphical values of MDD and OMC of untreated soil.

3.3.8 pH Test

Standard Test Method for using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization which is ASTM D 6276, where introduced to obtain lime dosage. This test identified the lime content required to satisfy immediate lime-soil reaction and a high pH (about 12.4 at 24°C) will observed.

(a). Samples during PH test (b). PH meter.

Figure 3.6: Soil samples during PH test and PH meter

The procedure, specimen preparation and all the way it is conducted is described briefly in Appendix D.

3.3.9 Unconfined Compression Test

The unconfined compression test is one of the most commonly used laboratory shear strength tests for cohesive soils and reasonably simple and rapid to perform. The test is conducted to determine the unconfined compressive strength (qu) which is then used to calculate the unconsolidated undrained shear strength (Cu) of the cohesive soil under unconfined conditions. The undrained shear strength (Cu) of a cohesive soil is equal to half of the unconfined compressive strength (qu) when the soil is under the φ= 0 condition. Moreover, the most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the undrained cohesion (Cu).

In this research UCS test was performed for two case scenarios which are untreated and lime treated conditions and during this the obtained unconfined compressive strength (qu) of the lime treated one is greater than the others that we have obtained, the results are 83.62 and 200.92Kpa for unteated and 6% Lime treated soil specimens respectively. This shows that the lime treated unconfined compressive strength (qu) is 117.3Kpa greater than the untreated.

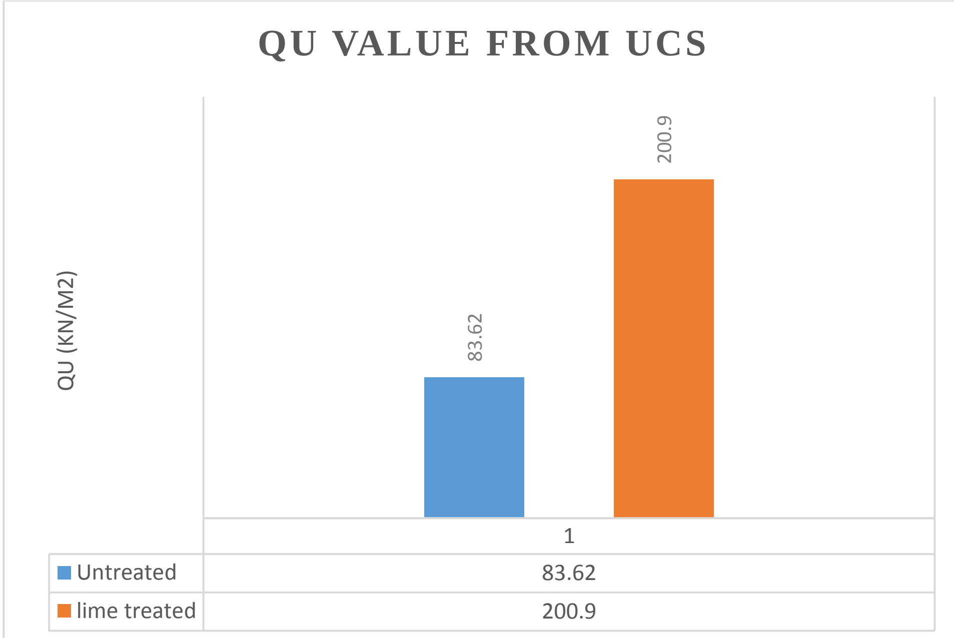


Figure 3.7: Result of unconfined compressive strength (qu), for untreated and lime treated soil.

3.3.10 One-Dimensional Consolidation Test

The main objective of the one-dimension consolidation test is to determine the compressibility of saturated fine-grained soils, which is considered a time-dependent phenomenon. In this study, the tests were conducted in accordance with ASTM D-2435-96 standard procedure on the soil specimens prepared at OMC soil by using an automated consolidometer test setup, Porous stones were placed on both the top and bottom of the specimen to facilitate water dissipation from the soil. After that, the specimens with porous stones were placed in a consolidation ring and transferred into a consolidometer. Water was added into the consolidometer to keep the soil saturated. During the saturation process, normally 24 hours, the specimen was under a seating load of 7Kpa in order to be certain that the specimens became saturated, with no swelling occurring prior to the loading. At the end of the test, the specimen was carefully removed from the ring, and the weight of the specimen was recorded immediately. The weight of the specimen after oven-drying was also measured in order to calculate the moisture content of the saturated specimen. Finally, void ratios were calculated using the height-of-solids method and plotted with vertical stress to obtain the compression indexes of the specimens.

The full data which is obtained from consolidometer dial guage reading, results of calculation and graphs are listed in appendix C.

3.3.11 Direct Shear Test

Direct shear test is a simple method used to measure the friction angle, cohesion, and shear strength of soils. The testing method follows closely the ASTM D-3080-98 procedure for standard direct shear test. The soil specimen size of 6*6cm. height and 2cm height were prepared for all samples at the OMC condition of the soil and at natural in-situ at 5% lime-treated condition for the soil. The soil specimen was placed in a shear box and installed in the direct shear testing machine. The specimen was then pre-consolidated under a water bath, with a load increment from the minimum applied at normal stress of 50Kpa and increase to 100Kpa and 200Kpa. During the consolidation stage, the upper and lower shear box halves were held in contact with each other with alignment screws.

The full data which is obtained from direct shear test dial gauge reading and graphs are presented in appendix A.5.

3.4 Finite Element Method (FEM)

One of the subtopics of this chapter in addition to the soil test and its result is about the finite element software, Geotechnical applications require advanced constitutive models for the simulation of the non-linear, time-dependent and anisotropic behavior of soils and/or rock, therefore the finite element method program Plaxis 2D version 8.6 issued to model single piles' behavior under axial loading.

3.4.1 The Model

Numerical simulation of settlement behavior of axially loaded single pile and group pile were investigated using PLAXIS 2D and PLAXIS 3D finite element packages respectively. In both investigations, three primary finite element material models were incorporated and they are (i) Linear Elastic (LE) model, (ii) Mohr Coulomb (MC) model and (iii) Hardening Soil (HS) model.

Linear Elastic (LE) model is based on Hookes's Law of isotropic elasticity. It involves two basic elastic parameters i.e. Elastic Modulus (E) and Poisson's ratio (υ). Although the linear elastic model is not suitable to model the soil, it may be used to model the stiff volume of soil or stiff formulations in the soil. In this study, the piles were modelled using LE material model. (Seo H and Prezzi M, 2007].

The elastic-plastic Mohr-Coulomb model which involves five input parameters, which are E and ν for soil elasticity; φ and c for soil plasticity and ψ as an angle of dilatancy were considered. Mohr-Coulomb model represents a 'first-order' approximation of soil or rock behaviour. It is recommended to use this model for a first analysis of the problem considered. For each layer one estimates a constant average stiffness. Due to this constant stiffness, computations tend to be relatively fast and one obtains a first impression of deformations. (PLAXIS Version 8 Material Models Manual).

Drained behavior: Using this setting no excess pore pressures are generated. This is clearly the case for dry soils and also for full drainage due to a high permeability (sands) and/or a low rate of loading. This option may also be used to simulate long-term soil behavior without the need to model the precise history of undrained loading and consolidation. (PLAXIS Version 8 Reference Manual)

The analyzed piles are cylindrical and isolated and the loading is performed axially. These conditions allow for the use of symmetry; thus, the single piles behavior is modelled in 2D.

3.4.2 Geometry

The generation of a finite element model begins with the creation of a geometry model, which is a representation of the problem of interest. A geometry model consists of points, lines and clusters. Points and lines are entered by the user, whereas clusters are generated by the program. In addition to these basic components, structural objects or special conditions can be assigned to the geometry model to simulate tunnel linings, walls, plates, soil-structure interaction or loadings. It is recommended to start the creation of a geometry model by drawing the full geometry contour. In addition, the user may specify material layers, structural objects, lines used for construction phases, loads and boundary conditions. The geometry model should not only include the initial situation, but also situations that occur in the various calculation phases. After the geometry components of the geometry model have been created, the user should compose data sets of material parameters and assign the data sets to the corresponding geometry components. When the full geometry model has been defined and all geometry components have their initial properties, the finite element mesh can be generated. (PLAXIS Version 8 Reference Manual).

The model consists of two different materials are used in this analysis: reinforced concrete (referred to as "pile") and soil. The simulation is performed under drained conditions. This is an elastic analysis, and therefore it is considered that the materials do not yield. The soil-pile interface (parameter Rinter = 0.85), i.e. there is a small slip or gap between the two materials when the load is applied. The value of the Poisson's ratio, ν, is commonly used. The value of the Young's modulus of the pile, Ep = $30*10^6$ Kn/m^2 used for reinforced concrete which is obtained from (Joana Gonçalves Sumares Betencourt Ribeiro's, 2013) research, and the one of the soil, E,is 8.78Mpa from UCS test result.

For the boundaries, standard fixities are used: the bottom boundary's vertical and horizontal displacements are null (representing the rigid layer), the left and right boundaries' horizontal displacements are null and the upper boundary is free, as represented in Figure 3.2

The soil and pile material properties which are used as an input of the software are listed as shown in table 3.6 and 3.7

Table 3.6: Soil and pile material property

Property		Soil	Pile
c (KN/m^2)	Treated	18	N/A
	Untreated	12	
ϕ	Treated	21.3^0	N/A
	Untreated	18.3^0	
γ_{sat}	Treated	18.66	24
	Untreated	15.26	
v_s	0.3		N/A
E_s (Kpa)	8787		N/A
v_p	N/A		0.15
E_p (Mpa)	N/A		30000

Table 3.7: Material properties used during analysis.

	Material Model	Material type	R_{inter}
Pile	Linear Elastic	Drained	0.85
Soil	Mohr columb	Drained	0.85

The soil parameters c and φ are obtained from direct shear test and unit weight of the soil is obtained results of compaction test which are MDD and OMC, from a correlation shown below

$\rho_d * g = \gamma_d$ equation 3.1

$1.16 * 9.81 = 11.37 KN/m^3$ dry unit weight for untreated

$1.35 * 9.81 = 13.14\ KN/m^3$ unit weight for lime treated

After obtaining dry unit weight we can calculate bulk unit weight by the following relation

$\gamma_{d\ (1\ +\ \omega)} = \gamma_b$ equation 3.2

$11.37(1+ 0.35) = 15.36\ KN/m^3$

$13.14(1+ 0.402) = 18.6\ KN/m^3$

Where: ρ_d : Maximum Dry density (MDD)

g: Gravity

γ_d : Dry unit weight

$\gamma_{b:}$ bulk unit weight

ω: Optimum moisture content (OMC)

3.5 Method

In general this research is conducted for comparing the magnitude of settlement of single axially loaded pile at different scenarios which are shown in chart 3.1, this comparison is required to identify that weather the stabilizer is effective or not as compared with increasing the length and diameter of the pile.

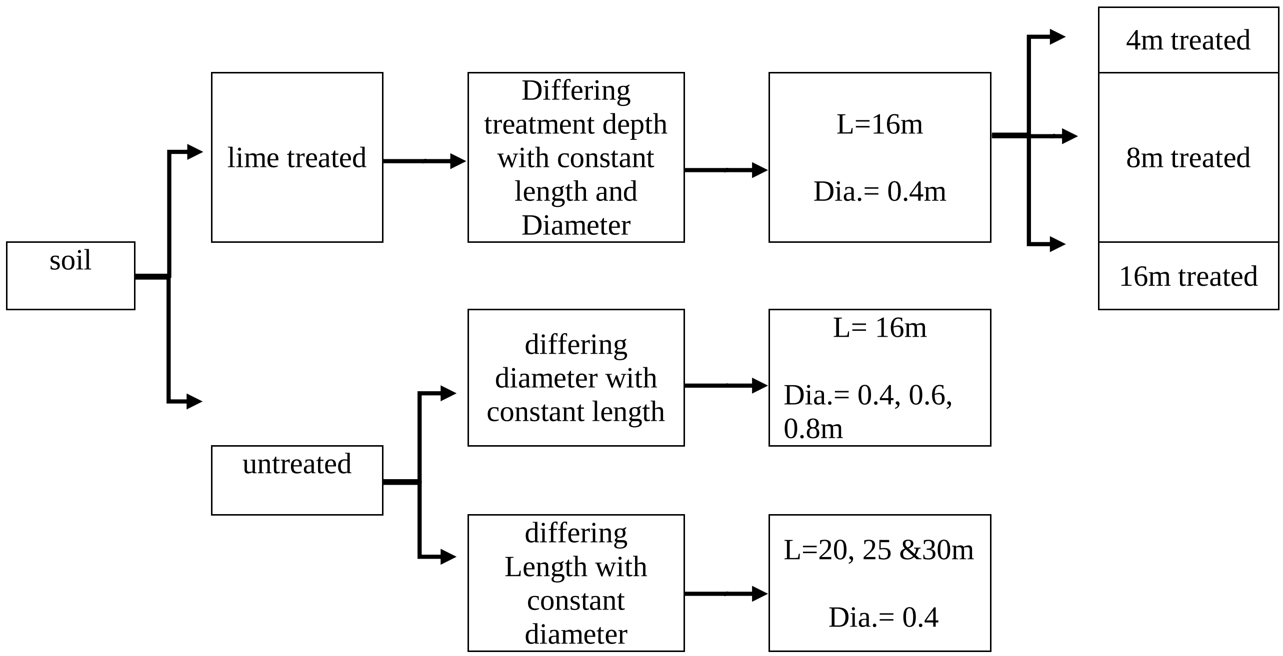


Figure 3.8: Different scenarios of the pile and the soil assumed for the analysis.

For the case of this research the soil was found in different conditions that it is treated and untreated soil, for the case of treated soil its assumed that the length of the pile L= 16m with diameter is equals to 0.4m for 4m, 8m, and 16m treated soil condition and for the case of untreated soil the pile is exposed for two conditions which are constant length L=16m by differing the diameter to 0.4m, 0.6m and 0.8m, and constant diameter which is equals to 0.4m by differing the length of the pile L= 16m, 20m, 25m.

The three scenarios observed in the chart are used to compare the magnitude of settlement of the pile and to select the appropriate condition for installing the pile.

Bored kind of pile installation will be used for the numerical analysis purpose. Here it should be known that the dimensions of the pile listed above are an assumption in order to load it and knowing the effect of stabilization settlement.

In order to understand the performance of piles in both lime-treated and untreated clay; the following specific tasks will be conducted.

- Selection of clay soil and corresponding basic soil characterization studies.

- Selection of optimum lime dosages (by applying PH test as per ASTM 6276) and corresponding characterization.
- Laboratory determination of index and engineering properties of both control and lime treated soil.
- Finite element studies, using Plaxis 2D, for analyzing the settlement of pile both untreated and treated ground
- Based on FEM analysis result, treatment recommendations.

The overall procedure of the research is presented as follows,
First searching a site having clay soil, then after obtaining the site block sampling were taken and preserve it till its transported to lab, test will be conducted.

The next phase is know the dosage of lime requirement to mix with the soil and this is known by pH test as per ASTM D 6276 (Standard Test Method for "Using pH to Estimate the Soil-Lime Proportion requirement for Soil Stabilization"). pH is measured because it's one of major soil properties that influence the soils ability to react with lime to produce cementitious material.

Then laboratory test on index and engineering properties will be conducted both on the untreated and lime soil mixture for numerical analysis data input.

The effect of stabilization on the magnitude of pile settlement will be discussed, this is done by changing the depth of injection for the treated case and changing the length and diameter of the pile for the untreated case as described in chart 3.1 using numerical analysis software Plaxis 2D.

The settlement analysis is held in three different scenarios for treated soil:

First by using the lime treated soil parameters from the top of the pile tip up to one fourth (1/4) of the total depth of a pile length which is 4m.

Second using the lime treated soil parameters up to half length of the pile from the top of it which is 8m.

Finally effect of grouting a soil all over the piles length this means using the lime treated soil parameters on the total length of the pile which is 16m.

The settlement analysis is held in three different scenarios for untreated soil both for constant length and constant diameter piles:

For the case of constant length L=16m the diameter is varies like 0.4m, 0.6m and 0.8m and for the case of constant diameter which is equals to 0.4m the length varies like 16m, 20m and 25m.

4. RESULTS AND DISCUSSIONS

4.1 Discussions of the Laboratory Test Results

In the previous chapter it was explained about conducted laboratory test with results obtained and also the geometry and material interface which is used to model the pile.

This chapter discusses on the results obtained from laboratory test and finite element software.

1. The first experiment conducted in the laboratory is grain size analysis, the results obtained from the grain size analyses indicate that the dominant proportion of soil particle in the research area is fine soil, this indicates us the sample may be is silt or clay kind of soil and we can judge that its either clay or silt using USCS soil classification system. Figure 4.1 indicates the total grain size distribution of the sample both from sieve and hydrometer analysis.

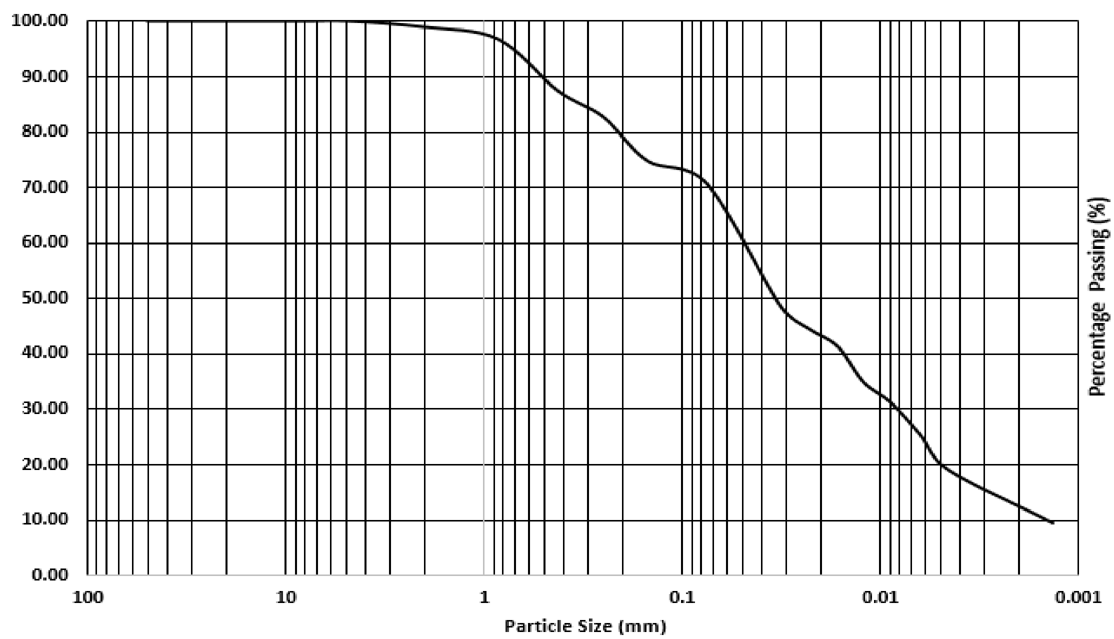


Figure 4.1: Total grain size distribution curve.

Table 4.1: Particle size in percentage

Gravel (%)	0.00
Sand (%)	29.36
Fines (%)	70.64

Generally the figure and table above indicates us it's a very fine kind of soil.

2. The other test is Atterberg Limit test which were conducted as per ASTM D 4318-98. results obtained from the test are liquid limit is 45%, plastic limit 25% and plastic index for this soil is 20 for the untreated soil and PI of the treated soil was 7 which means that the addition of six percent hydrated lime decreases its PI by 35%. Here the main aim of stabilizing clay soil using hydrated lime is to decrease its PI is achieved successfully. It was also above "A" line shown as a sign of cross and found in "CL" type of clay soil as shown in figure 3.4.

While comparing atterberg limit tests of the research done by abdulsemed Kemal, he have obtained liquid limit is 46%, plastic limit is 28% and plastic index for this soil is 18%.

3. According to Unified Soil Classification System, the soil under investigation lies above the A-line. It contains 70.64% fines as discussed above and this value is greater than 50% and its PI is 20 which is greater than 7. Therefore the sample is said to be lean clay having a group symbol CL. Here also the type of the soil which is discussed in Abdusemed kemal's research was lean clay from USCS soil classification.

4. The results obtained from compaction test which are the maximum density and optimum moisture are needed for determining the bulk unit weight using equation 3.1 and 3.2 and both bulk and dry unit weight are used as an input for Plaxis analysis. The sampled clay for this research attained a maximum dry density of 1.158g/cm^3 (11.3kn/m^3) at 35% moisture content for the controlled or no lime soil sample, and this result is found in acceptable range 1.16–1.35g/cm^3 (TxDOT Designation: Tex-142-E, 1999) and maximum dry density of 0.968g/cm^3 (9.5kn/m^3) at 32.5% moisture content for the lime treated soil.

   From the previous research of Abdulsemed Kemal's the MDD of the untreated soil were 17.35kn/m^3 at 13.4% moisture content.

5. The optimum dosage of the lime treatment of the soil for this research was determined using pH tests. From the pH versus lime content plot figure 4.3, the optimum dosage of the lime content was determined to be 6% of the soil. Researches done on stabilizing a

soil by lime using pH method as per ASTM D 6276 found their percent of lime mostly less than 8%, like Abdulsemed Kemal, 2014 obtains 5% of lime and Aref al-Swaidani et al., 2016 obtained 8% of lime were enough for stabilizing the soil. More detail explanation and procedure on how to find a PH of lime treated soil is discussed in Appendix D of the last chapter.

6. As Direct shear tests were conducted on three samples of untreated soil, three samples of the undisturbed and three samples of lime-treated soil. From the results, it was observed that the friction angle of this soil sample was 18.3^0 with cohesion $9KN/m^2$ for untreated sample, friction angle 17.7^o with cohesion 17 KN/m^2 for undisturbed and friction angle 21.3^0 with cohesion 18 KN/m^2 lime-treated soil sample. Likewise (Abulsemed Kemal 2014) also obtained the value of untreated soil 22^o and the treated one was 33^o.

   From the result of direct shear test the value of cohesion lime treated soil increased by 50% from the untreated soil sample thus with the increase in the value of cohesion the soil pile interaction and friction also increases so the lime treated soil can resist more load by friction than the untreated.

7. The unconfined compression strength obtained from unconfined compression tests which were conducted in accordance with the ASTM D 2166 standard for both conditions which are untreated and lime-treated samples. The UCS of lime treated sample was done after a 7-day curing period and obtained results are 83.62Kpa and 200.92Kpa for untreated and lime treated samples respectively, this shows that a 58.38% increase in unconfined strength of lime treated soil from untreated sample.

8. The consolidation test was conducted for all the conditions as done for the previous soil tests which are for undisturbed, untreated and 6% lime-treated soil for determination of the preconsolidation pressure, compressibility index, and void ratio. And the results are shown in the table below.

Table 4.2: Result of laboratory test.

Test			Test results for the sampled soil
Specific gravity			2.54
Atterberg limit test	LL	Treated	19%
		Untreated	45%
	PL	Treated	12%
		Untreated	25%
	PI	Treated	7
		untreated	20
USCS			CL
pH			12.3 for 6% lime
		Untreated. E (Mpa)	8.78
		Lime Treated E (Mpa)	21.1
		Untreated	Cc= 0.048, e= 0.99
		lime treated	Cc= 0.016, e= 0.95
		Untreated	$\varphi = 18.3^0$, c = 9
		Lime treated	$\varphi = 21.3^0$, c = 18

In general it's observed that the engineering parameter of the soil has increased due to the addition of 6% hydrated lime, since stabilization of clay soil using hydrated lime is effective the next step is to use this laboratory results for finite element analysis software Plaxis.

4.2 Discussion on Stabilization

As the term soil stabilization refers it is the procedure in which a special soil, a cementing material, or other chemical or nonchemical materials are added to a natural soil or a technique use on a natural soil to improve one or more of its properties. One may achieve stabilization by physically mixing the natural soil and stabilizing materials together so as to achieve a homogeneous mixture or by adding stabilizing material to an undisturbed soil deposits and obtaining interaction by letting it permeate through soil voids, Abood et al., (2007), and for the case of this research the stabilization is held on by Grouting method which is obtaining interaction by letting it permeate through the soil void.

The main reason for the discussion on the stabilization is because this research uses pH test to know the dosage for stabilizing that specific kind of soil, which is different from most of the researches done on stabilization, in many soil stabilization researches the dosage of stabilizer is obtained by trial and error which is by increasing the stabilizer percentage and finding the increase in the required parameter but while conducting pH test it’s not that much difficult to obtain the required amount of stabilizer. Every steps and equipment’s to conduct pH test is presented on appendix D.

In order to begin stabilization the soil sample should full fill the requirements to be stabilized by hydrated lime, which are Plasticity Index > 15-18 and Clay Content > 25-30%. (Indianapolis, 2018).

As shown above on the table 4.1 the fines are 70.64% which is greater than 30% and the PI of the soil is 20 greater than 18. Therefore the soil satisfies the requirements to be stabilized by hydrated lime.

The next thing to conduct is determining the amount of hydrated lime required for stabilization, the concentration or dosage of stabilizer required is determined by chemical test pH test, as discussed on chapter two a concentration of only approximately 0.064 g/1 of hydrated lime will increase the pH of distilled water from 7 (neutrality) to above 11. Laboratory test for pH was conducted as per ASTM D6276 and the result found indicates that 6% of hydrated lime is required for stabilizing a soil.

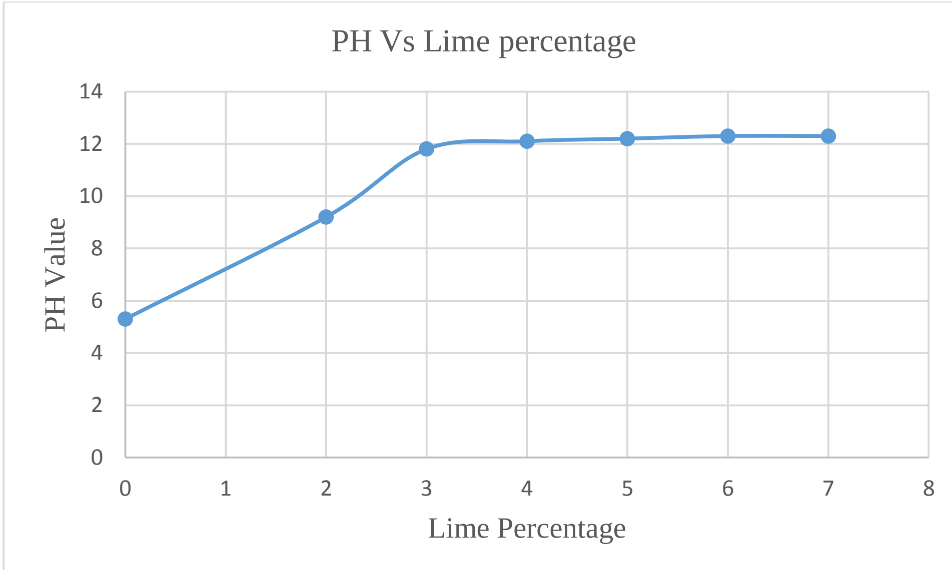


Figure 4.2: pH value versus percentage of lime

The stabilization process greatly influences the engineering characteristics of the soil, for instance let us see the stabilization effect on UCS by comparing the case scenarios which are Disturbed untreated and 6% Lime treated soil specimens,

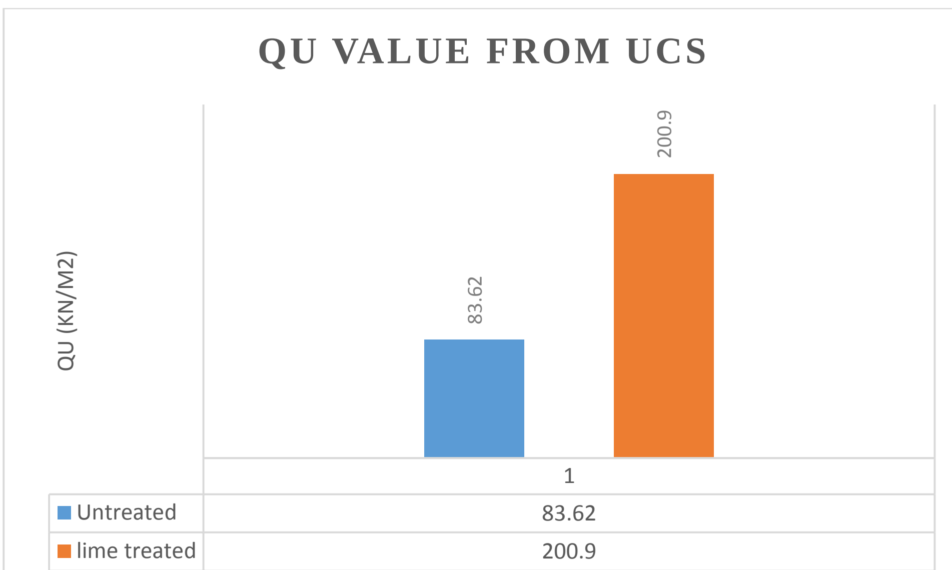


Figure 4.3: Value of qu for untreated and treated soil.

Here lime treatment increases the unconfined compressive strength of the soil even from the soils undisturbed condition. Therefore this mean that the research conducted on stabilizing a soil for pile installation is successful on its stabilization part and we are going to check the settlement on treated and untreated soil on the next section.

4.3 Discussion on the FEM and Settlement Analysis of a Pile

In this section results provided by the FEM software Plaxis discussed, the analyzed pile is cylindrical and isolated, subject to axial loading. Therefore, it is was analyzed by a two-dimensional axisymmetric simulation with 15-node triangular elements are used.

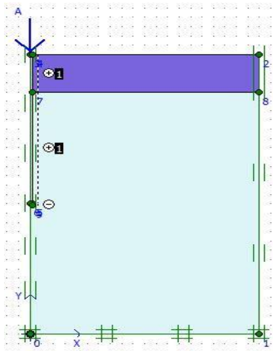
As the tutorial manual of Plaxis describes on selecting either 15-node or 6-node element, the generation of the mesh, clusters are divided into triangular elements. A choice can be made between 15-node elements and 6-node elements. The powerful 15-node element provides an accurate calculation of stresses and failure loads. In addition, 6-node triangles are available for a quick calculation of serviceability states. Considering the same element distribution (for example a default coarse mesh generation) the user should be aware that meshes composed of 15-node elements are actually much finer and much more flexible than meshes composed of 6-node elements, but calculations are also more time consuming.

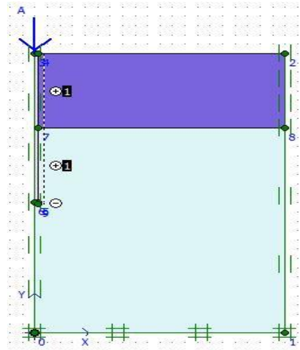
The pile is modelled as linear elastic, and the soil as elasto-plastic using the Mohr-Coulomb criterion, the parameters of the soil like c, φ, E and other were determined in from laboratory test,

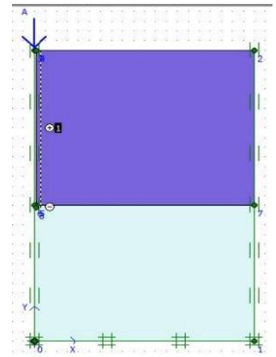
The settlement analysis is conducted for three different kind of cases, one is by differing the piles diameters as 0.4m, 0.6m and 0.8m by making the length of the pile 16m and differing the depth of pile 20m, 25m and 30m with constant diameter 0.4m for the untreated soil case and the other is by differing the depth of stabilized soil 4m, 8m and 16 m for a constant diameter of pile 0.4m.

Pile load carrying capacity depends on various factors, including: (1) pile characteristics such as pile length, cross section, and shape; (2) soil configuration and short and long-term soil properties; and (3) pile installation method. (Bogumił wrana, 2015). One of the objective of this research is determining the load bearing capacity of a pile by differentiating its cross section.

The geometry of the system with its exact position of node is shown for a 0.4m diameter pile and other conditions.

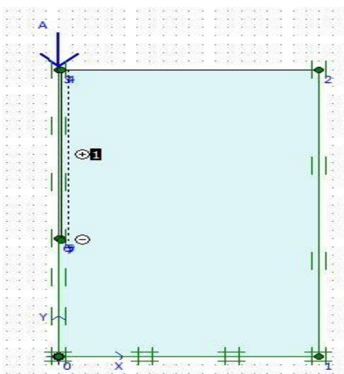


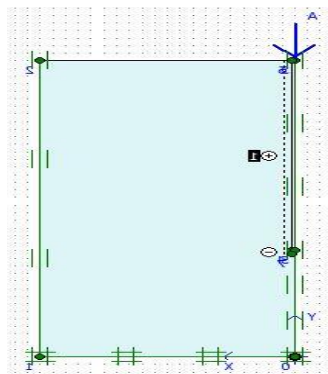


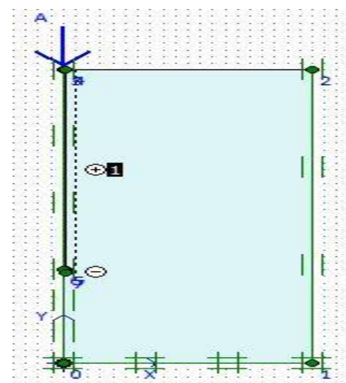


a) 4m depth treated soil geometry b) 8m depth treated soil geometry c) 16m depth treated soil geometry

Figure 4.4: Plot of geometry model with significant nodes for 0.4m diameter by differing the depth of treatment

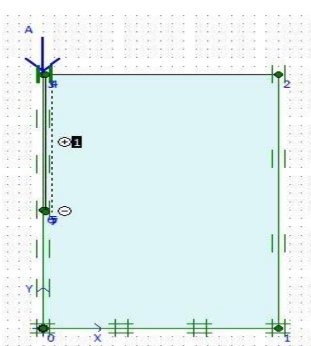


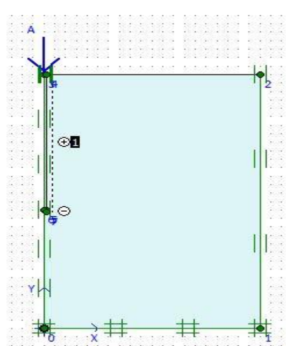


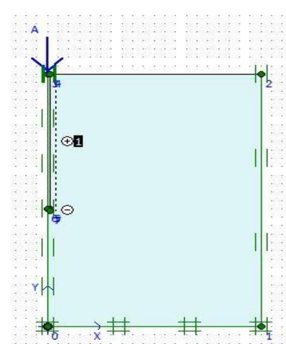


d) 20m pile geometry e) 25m pile geometry f) 30m pile geometry

Figure 4.5: Plot of geometry model with significant nodes for 0.4m diameter by differing length of the pile







g) 0.4m diameter pile geometry h) 0.6m diameter pile geometry i) 0.8m diameter pile geometry

Figure 4.6: Plot of geometry model with significant nodes for 16m length by differing diameter of the pile.

The analysis is done by giving a concentrated load of 1300KN which is the software gives us the magnitude of settlement for that specific soil condition by the assigned kind of soil and pile property. As shown in figure 4.4, 4.5 and 4.6. So we can compare the observed minimum magnitude of settlement from the scenarios considered.

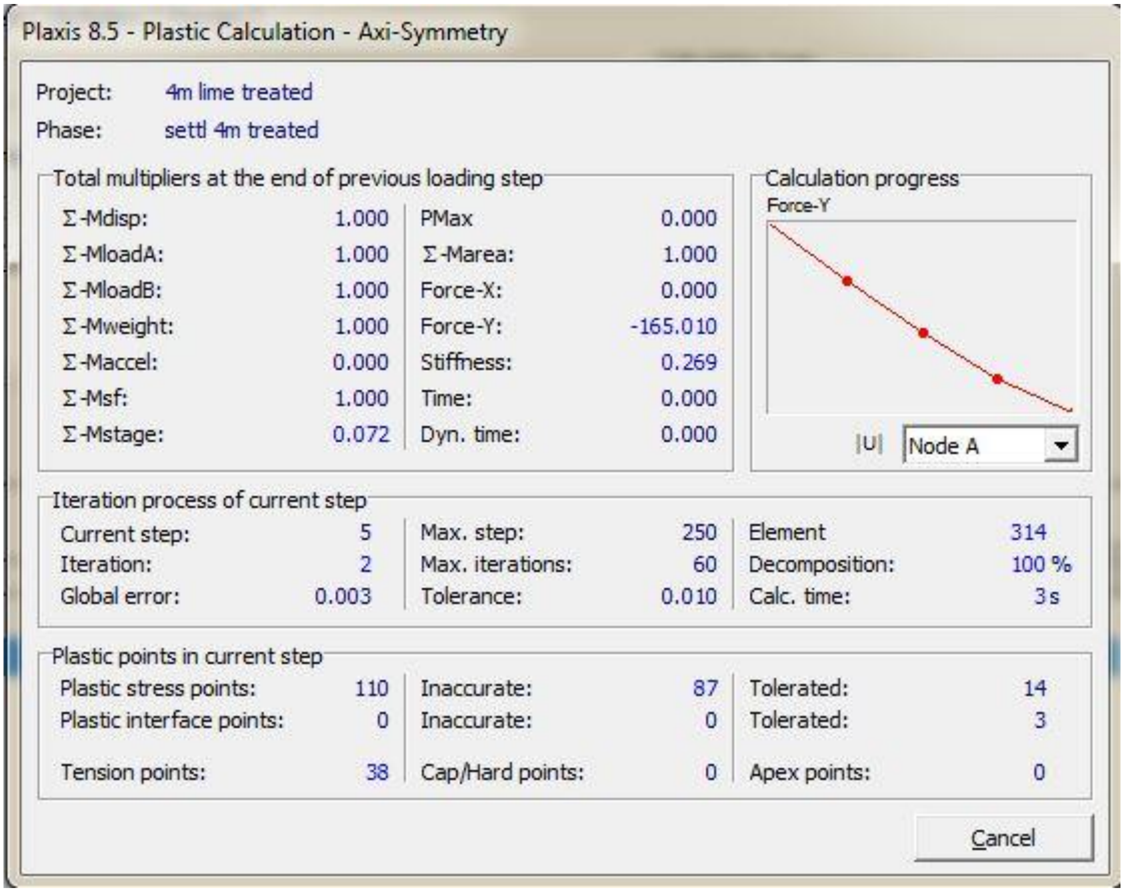


Figure 4.7: Calculation progress observed during conducting the analysis.

The first case in the analysis is observing the results by differing the treatment depth, as the depth of treatment increase the magnitude of settlement decreases as shown in table 4.3, the settlement obtained from the applied 1300KN load for 4m treatment is -2.74cm and for 8m treatment is -1.86cm, and finally for 16m treatment -1.19cm.

Table 4.3: Load settlement data from Plaxis 2D for Lime treated soil

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0036	307.44
-0.0072	600.21
-0.0108	844.70
-0.0144	1036.27
-0.0180	1158.80
-0.0216	1228.84
-0.0274	1300.00

(a) 4m Lime Treated condition

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0064	645.45
-0.0097	875.91
-0.0129	1069.74
-0.0161	1224.68
-0.0186	1300.00

(b) 8m Lime Treated condition

|U| [m]	Fy [kN]
0.00000	0.00
0.00000	0.00
-0.00361	474.66
-0.00723	917.42
-0.01084	1278.78
-0.01117	1300.00

(c) 16m Lime Treated condition

Abdusemed, 2014. In his research, the result found by improvement in the skin friction resistance due to treated soil was not significant. However, as the depth of the treated soil reached a distance equal to the depth of the drilled shaft, the maximum settlement dramatically reduces. This result shows that the there was a significant increase in the end bearing due to the lime treatment. And here also there is a better bearing capacity as the depth of the treated soil reached a distance equal to the depth of the pile.

The second case in the analysis is observing the results by differing the diameter of the pile with constant length 16m without any treatment, as the diameter of the pile increase the load required to settle is increased and the magnitude of settlement decreased for the increase in the diameter of the pile is more satisfactory. However the magnitude of the load resistance of the treated soil is greater than the second condition which is increasing the diameter, the increasing rate of load resistance for the second condition is better. As shown below the settlement obtained from 0.4m diameter is -7.14cm and for 0.6m diameter -1.9 and finally for 0.8m diameter -1.42cm.

Table 4.4: Load settlement data from Plaxis 2D for untreated soil, 16m length with different diameter

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0111	746.48
-0.0167	953.25
-0.0223	1065.66
-0.0278	1124.49
-0.0334	1160.79
-0.0445	1216.31
-0.0556	1256.29
-0.0668	1289.41
-0.0714	1300.00

(a) 0.4m Diameter pile

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0141	1123.09
-0.0190	1300.00

(b) 0.6m Diameter pile

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0141	1296.62
-0.0212	1662.18
-0.0142	1300.00

(c) 0.8m Diameter pile

The third case in the analysis is observing the results by differing the length of the pile with constant and minimum diameter of 0.4m without any treatment, as the length of the pile increase the applied constant load shows a decrease in settlement as shown on the table 4.5 and the magnitude of settlement decreased for the increase in the length of the pile is more satisfactory than the previous two condition. As shown below the 1300KN applied load required to determine settlement for 20m pile is gives us -1.83cm and for 25m pile is 1.65cm and finally for 30m pile 1.42cm. The third condition which is increasing the length of the pile even with small diameter of the pile is better in resisting the load applied axially from super structures and at the same time the is no great variation between the magnitude of settlement for different observed length of pile, but during observing the treated soil there is a great variation between the 4m treated and 16m treated which indicates us that the soil needs a great depth of treatement for being safe which is uneconomical time taking activity.

Table 4.5: Load settlement data from Plaxis 2D for untreated soil 0.4m Diameter with different length

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0083	666.72
-0.0125	961.12
-0.0167	1220.46
-0.0183	1300.00

(a) 20m length pile

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0067	603.78
-0.0134	1150.51
-0.0165	1300.00

(b) 25m length pile

|U| [m]	Fy [kN]
0.0000	0.00
0.0000	0.00
-0.0082	781.83
-0.0142	1300.00

(c) 30m length pile

As per Abdusemed, 2014. The depth of the drilled shaft varied from 5 to 20 m, with an addition of 5 m in between. The settlements calculated in the 5% lime treated soil were considerably lower than those of the natural soil.

The third scenario of this research is comparing the treated soil with a pile having greater length and obtained a better bearing capacity for longer pile with untreated soil than the treated but shorter length which is different from the previous researches due to the settlement magnitude comparison method difference.

Table 4.6: Summary of the output from Plaxis.

Scenarios		Load (KN)	Settlement (cm)
Length = 16m	Dia. =0.4m	1300	-7.14
	Dia. =0.6m	1300	-1.9
	Dia. =0.8m	1300	-1.42
Diameter= 0.4m	L =20m	1300	-1.83
	L =25m	1300	-1.65
	L =30m	1300	-1.42
Diameter= 0.4m And Length = 16m	4m Treated	1300	-2.74
	8m Treated	1300	-1.86
	16m Treated	1300	-1.19

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This project has been to investigate the foundation principle of an axially loaded single pile and how well this can be modelled with numerical analysis, by general option of axisymmetry model using the finite element software PLAXIS 2D.

During conducting the research, different lab experiments and numerical studies were conducted to investigate the effect of lime treatment on a clayey soil collected from Addis Ababa around casanchis. After basic soil characterization studies, pH tests were conducted to obtain the optimum dosage of lime treatment for the soil. After the determination of the optimum lime content for the soil, direct shear, unconfined compression strength, atterberg limit and consolidation tests were conducted both for the untreated and lime-treated soils. During testing the lime treated soil the soil-lime mixture was cured for 7 days as described in appendix B.

Then the numerical study was conducted in PLAXIS to investigate the load resisting capacity of lime treated soil in comparison with the untreated soil. The pile in the untreated soil is analyzed in two conditions which are by increasing its diameter and length.

Finally the load carrying capacity of the soil was better for the pile in the increasing the length than increasing the diameter and treating the soil using lime, however the magnitude settlement for 16m treated soil is smaller than all the observed case it should be compared with others based on cost and time required to stabilize the soil. Therefore it can be concluded that stabilizing this kind of soil using hydrated lime do not have that much efficiency in decreasing the settlement as compared with increasing the diameter and length due to that it takes time and needs great cost to stabilize it.

5.2 Recommendations

Based on the findings of this research, the following recommendations are forwarded

Most of the time hydrated lime is used on road constructions and the consumption of hydrated lime for stabilizing a soil for building a structure not adapted specially in our country, so I recommend to use hydrated lime for stabilization for structures since it have a long term stabilization advantage.

There are no soil stabilization equipment's for deeper depth are found in our country, therefore I recommend for deep investigation about the equipment and injection application technique, and comparison of cost of injection with other load carrying capacity factors like increasing cross sectional area and length of the pile can also be done, so I recommend to go deep through this.

Though the finite element analysis provides efficient results, the programs need many input parameters and may be complicated to use. Analysis should be performed very carefully, as the parameters are very sensitive to affect the output by change the input little bit. This concept enables new users to work with the package after only a few hours of training. Therefore I recommend to practice the software and simplifying complexities occurred in real life.

Finally I recommend to increases the depth of a pile with a minimum diameter than increasing the diameter with smaller length and treating the soil in order to get a better bearing capacity.

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APPENDICES

Appendix A: Laboratory tests

Appendix A 1. Particle Size Distribution.

Table A 1.1: percentage passing during sieve

sieve size (mm)	sieve size (ASTM)	Mass Retained (g)	Cumulative mass retained (g)	Mass passing (g)	Percentage passing (%)
75	3-in	0.0	0	1744.20	100.00
50	2-in	0.0	0	1744.20	100.00
19	1 1/2-in	0.0	0	1744.20	100.00
16	1-in	0.0	0	1744.20	100.00
11.3	7/6-in	0.0	0	1744.20	100.00
8	5/16-in	0.0	0	1744.20	100.00
4.75	No. 4	0.0	0	1744.20	100.00
2	No. 10	18.5	18.5	1725.70	98.94

Table A 1.2: Hydrometer reading data

Time interval (min)	Hydrometer reading	Temprature (^{0}C)	Calibration cylinder reading	Composite correction	Corrected Hydrometer reading	Percentage passing (%)	K factor	Particle size, D (mm)
2	1.0183	22.0	1.0030	-0.003	1.0153	48.62419	0.01332	0.0319402
4	1.0170	22.0	1.0030	-0.003	1.014	44.49272	0.01332	0.0228779
8	1.0160	22.0	1.0030	-0.003	1.013	41.31467	0.01332	0.0163814
15	1.0140	22.0	1.0030	-0.003	1.011	34.95857	0.01332	0.012208
30	1.0128	22.0	1.0030	-0.003	1.0098	31.14491	0.01332	0.008802
60	1.0110	22.0	1.0030	-0.003	1.008	25.42441	0.01332	0.0062948
120	1.0090	22.0	1.0030	-0.003	1.006	19.06831	0.01332	0.0045334
1440	1.0060	22.0	1.0030	-0.003	1.003	9.534155	0.01332	0.0013458

Table A 1.3: Result of sieve after hydrometer reading

sieve size (mm)	sieve size (ASTM)	ass Retained (g)	Cumulative mass retained (g)	Mass passing (g)	Percentage passing (%)
2.000	No. 10	0.5360	0.53601437	50.00	98.94
0.850	No. 20	1.1	1.63601437	48.90	96.76
0.425	No. 40	4.7	6.33601437	44.20	87.46
0.250	No. 60	2.4	8.73601437	41.80	82.71
0.150	No. 100	4.0	12.7360144	37.80	74.80
0.075	No. 200	2.1	14.8360144	35.70	70.64

Table A 1.4: Overall particle size distribution

Sieve size (Particle size) (mm)	Sieve size (Particle size) (ASTM)	Percentage Passing (%)
75	3-in	100.00
50	2-in	100.00
19	1 1/2-in	100.00
16	1-in	100.00
11.3	7/6-in	100.00
8	5/16-in	100.00
4.75	No. 4	100.00
2	No. 10	98.94
0.850	No. 20	96.76
0.425	No. 40	87.46
0.250	No. 60	82.71
0.150	No. 100	74.80
0.075	No. 200	70.64
0.032	-	48.62
0.023	-	44.49
0.016	-	41.31
0.012	-	34.96
0.009	-	31.14
0.006	-	25.42
0.005	-	19.07
0.001	-	9.53

Table A 1.5: Coefficient of uniformity and coefficient of curvature

Cu	36.08735574
Cc	0.846718817

Appendix A 2. Atterberg Limits Determination Untreated soil

Table: A 2.1: Data for water content determination for Liquid limit for untreated condition

Type of Test	Liquid Limit				Plastic Limit	
Number of Blows	35	28	22	15		
Container Number	155	152	128	104	102	165
Mass of Empty Container, M_c	33.2	33.0	33.4	33.6	33.3	33.2
Mass of Container + Wet Soil, M_{csw} (g)	49.5	52.5	51.7	60.9	39.4	40.1
Mass of Container + Dry Soil, M_{cs} (g)	44.6	46.5	46.0	52.2	38.2	38.7
Mass of Dry Soil, M_s (g)	11.40	13.50	12.60	18.60	4.90	5.50
Mass of Water, M_w (g)	4.90	6.00	5.70	8.70	1.20	1.40
Water Content, *w* (%)	42.98	44.44	45.24	46.77	24.49	25.45

Figure A 2.1:. Graph of water content versus number of blow for untreated soil sample.

Table: A 2.2: Atterberg limit value for untreated condition

Liquid Limit, LL or w_L (%)	45
Plastic Limit, PL or w_p (%)	25
Plasticity Index, PI (%)	20

Lime treated soil sample

Table: A 2.3: Data for water content determination for Liquid limit for lime treated condition

Type of Test	Liquid Limit				Plastic Limit	
Number of Blows	37	28	21	12		
Container Number	187	78	16	28	11	17
Mass of Empty Container, M_c	33.3	33.7	33.1	33.4	33.4	33.0
Mass of Container + Wet Soil, M_{csw} (g)	70.0	81.0	85.1	79.9	45.1	48.2
Mass of Container + Dry Soil, M_{cs} (g)	67.5	75.6	76.0	67.5	43.9	46.5
Mass of Dry Soil, M_s (g)	34.20	41.90	42.90	34.10	10.50	13.50
Mass of Water, M_w (g)	2.50	5.40	9.10	12.40	1.20	1.70
Water Content, w (%)	7.31	12.89	21.21	36.36	11.43	12.59

Figure A 2.2: Graph of water content versus number of blow for lime treated soil sample.

Table: A 2.4: Atterberg limit value for lime treated condition

Liquid Limit, LL or w_L (%)	19
Plastic Limit, PL or w_p (%)	12
Plasticity Index, PI (%)	7

Appendix A.3 Standard Proctor Compaction Test

Table: A 3.1: General Information about the compaction equipment's

Testing Standard	Sample Preparation Method	Testing Method	
ASTM D698	Air Dry	Method C	
Diameter of Mold (cm)	10.2	Mass of Mold (g)	4557.0
Height of Mold (cm)	11.6	No. of Layers	3
Volume of Mold (cm^3)	940.449	No. of Blows Per Layer	25
Drop Height of Hammer (cm)	30.5	Mass of Hammer (Kg)	2.5

Table: A 3.2: Moisture content and density determination sheet for untreated soil sample.

Trial No.	1	2	3	4	5
Molding Moisture Content Determination					
Container Number	91	98	120	171	179
Mass of Empty Container, M_c	32.7	33.6	33.1	33.3	32.9
Mass of Container + Wet Soil, M_{csw} (g)	81.0	97.8	102.8	105.0	107.1
Mass of Container + Dry Soil, M_{cs} (g)	76.7	88.3	89.0	86.6	84.8
Mass of Dry Soil, M_s (g)	44.0	54.7	55.9	53.3	51.9
Mass of Water, M_w (g)	4.3	9.5	13.8	18.4	22.3
Water Content, w (%)	9.8	17.4	24.7	34.5	43.0
Density Determination					
Weight of soil + Mould, M_{MS} (g)	5582.4	5728.8	5866.4	6010.6	6090.0
Weight of soil, M_S (g)	1025.4	1171.8	1309.4	1453.6	1533.0
Bulk Density, ρ (g/cm^3)	1.095	1.251	1.398	1.552	1.637
Dry Density, ρ_d (g/cm^3)	0.997	1.066	1.121	1.154	1.145

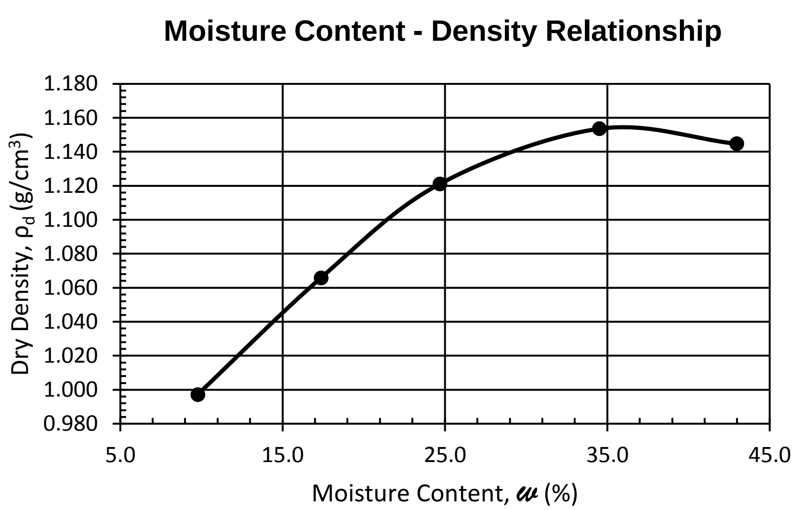


Figure: A 3.1: MDD versus OMC curve for untreated soil

Table: A 3.3: Moisture content and density determination sheet for lime treated soil sample.

Trial No.	1	2	3	4	5
Molding Moisture Content Determination					
Container Number	128	132	142	166	182
Mass of Empty Container, M_c	33.4	33.8	33.8	33.7	33.4
Mass of Container + Wet Soil, M_{csw} (g)	70.3	73.8	77.8	74.7	85.4
Mass of Container + Dry Soil, M_{cs} (g)	65.3	66.4	67.7	63.1	68.6
Mass of Dry Soil, M_s (g)	31.9	32.6	33.9	29.4	35.2
Mass of Water, M_w (g)	5.0	7.4	10.1	11.6	16.8
Water Content, w (%)	15.7	22.7	29.8	39.5	47.7
Density Determination					
Weight of soil + Mould, M_{MS} (g)	5150.0	5630.9	5950.0	6300.0	6250.0
Weight of soil, M_S (g)	589.0	1069.9	1389.0	1739.0	1689.0
Bulk Density, ρ (g/cm^3)	0.629	1.142	1.483	1.856	1.803
Dry Density, ρ_d (g/cm^3)	0.544	0.931	1.142	1.331	1.221

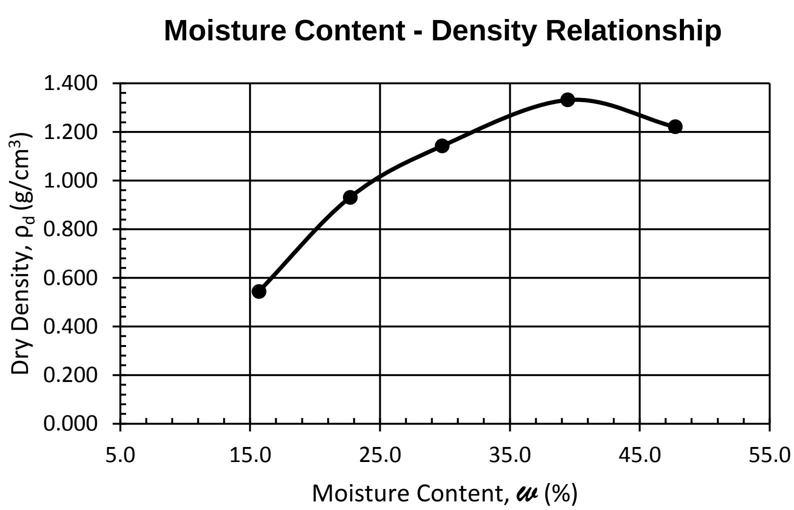


Figure: A 3.2: MDD versus OMC curve for untreated soil

Appendix A 4 Unconfined Compressive Strength Test

Unconfined Compressive Strength (qu),kPa = 120

Undrained Shear Strength (cu), kPa = 60

Table A 4.1: Information about the UCS machine and mass of the lime treated sample.

Diameter(cm)=	38.00
Length(cm)=	76.00
Rate (mm/min)	0.80
Ring Factor (N/div)=	0.746
Mass of Soil(g)	146.50
Date of Test :-	17/10/2014

Table A 4.2: Water content determination sheet for lime treated sample.

BH No	1	
Depth(cm)	_	
Container No	89	102
Wt of Wet Sample + Container (gm)	91.3	87.90
Wt of Dry Sample + Container (gm)	65.5	73.30
Wt. Of Water (gm)	25.8	14.6
Wt. Of Container (gm)	33.3	33.2
Wt of Dry Sample (gm)	32.2	40.1
Water Content (%)	80.12	36.41
Average Water Content (%)	58.266600579297	

Table A 4.3: Area, volume and unit weight determination sheet for lime treated soil.

Area,A_0 (mm^2)=		1133.54
Volume, (mm^3)=		86149.04
Bulk Unit Weight(g/cc)=		1.70
Dry Unit Weight(g/cc)=		1.07

Table A 4.4: Information about the UCS machine and mass of the sample lime treated

Diameter (cm)=	38.00
Length (cm)=	76.00
Rate (mm/min)	0.80
Ring Factor (N/div)=	1.3400
Mass of Soil (g)	146.50

Table A 4.5: Reading from deformation dial gauge and Load dial gauge for lime treated sample.

Deformation Dial Reading	Load Dial Reading	Sample Deformation,ΔL(mm)	Strain(ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.00	1133.540	0.000	0.00
10	14.0	0.10	0.00	0.13	1135.032	0.019	16.53
20	37.0	0.20	0.003	0.263	1136.523	0.050	43.62
30	43.0	0.30	0.004	0.395	1138.015	0.058	50.63
40	50.0	0.40	0.005	0.526	1139.506	0.067	58.80
50	56.0	0.50	0.007	0.658	1140.998	0.075	65.77
60	63.5	0.60	0.008	0.789	1142.489	0.085	74.48
70	69.0	0.70	0.009	0.921	1143.981	0.092	80.82
80	75.0	0.80	0.011	1.053	1145.472	0.101	87.74
90	81.0	0.90	0.012	1.184	1146.964	0.109	94.63
100	86.0	1.00	0.013	1.316	1148.455	0.115	100.34
110	90.0	1.10	0.014	1.447	1149.947	0.121	104.87
120	91.0	1.20	0.016	1.579	1151.438	0.122	105.90
130	98.0	1.30	0.017	1.711	1152.930	0.131	113.90
140	106.0	1.40	0.018	1.842	1154.421	0.142	123.04
150	113.0	1.50	0.020	1.974	1155.913	0.151	131.00
160	121.0	1.60	0.021	2.105	1157.404	0.162	140.09
170	124.0	1.70	0.022	2.237	1158.896	0.166	143.38
180	134.0	1.80	0.024	2.368	1160.387	0.180	154.74
190	140.0	1.90	0.025	2.500	1161.879	0.188	161.46
200	145.0	2.00	0.026	2.632	1163.370	0.194	167.01
210	153.0	2.10	0.028	2.763	1164.862	0.205	176.00
220	158.0	2.20	0.029	2.895	1166.353	0.212	181.52
230	164.0	2.30	0.030	3.026	1167.845	0.220	188.18
240	169.0	2.40	0.032	3.158	1169.336	0.226	193.67
250	173.0	2.50	0.033	3.289	1170.828	0.232	198.00
260	175.0	2.60	0.034	3.421	1172.319	0.235	200.03
270	176.0	2.70	0.036	3.553	1173.811	0.236	200.92
280	175.5	2.80	0.037	3.684	1175.302	0.235	200.09
290	174.4	2.90	0.038	3.816	1176.794	0.234	198.59
300	171	3.00	0.039	3.947	1178.285	0.229	194.47
310	165	3.10	0.041	4.079	1179.777	0.221	187.41

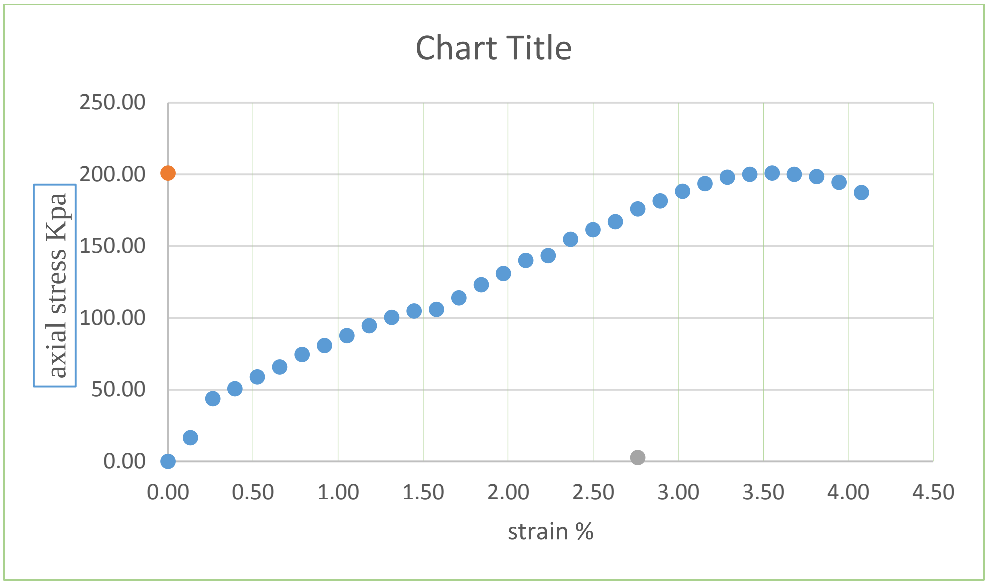


Figure: A 4.1: Axial stress versus percent strain curve for lime treated soil

Unconfined Compressive Strength (qu),kPa = 200.92

Undrained Shear Strength (cu), kPa = 100.45

Table A 4.6: Area, volume and unit weight determination sheet for untreated soil.

Area,A_0 (mm^2)=	1133.54
Volume, (mm^3)=	86149.04
Bulk Unit Weight(g/cc)=	1.76
Dry Unit Weight(g/cc)=	1.10

Table A 4.7: Information about the UCS machine and mass of the untreated sample

Dia(cm)=	38.00
Length(cm)=	76.00
Rate (mm/min)	0.80
Ring Factor (N/div)=	1.3400
Mass of Soil(g)	151.20

Table A 4.8: Reading from deformation dial gauge and Load dial gauge for untreated sample.

Deformation Dial Reading	Load Dial Reading	Sample Deformation,ΔL(mm)	Strain(ε)	% Strain	Corrected Area A'	Load (kN)	Stress (kPa)
0	0.0	0.00	0.00	0.00	1133.540	0.000	0.00
10	2.0	0.10	0.00	0.13	1135.032	0.003	2.36
20	3.0	0.20	0.003	0.263	1136.523	0.004	3.54
30	9.0	0.30	0.004	0.395	1138.015	0.012	10.60
40	10.0	0.40	0.005	0.526	1139.506	0.013	11.76
50	11.0	0.50	0.007	0.658	1140.998	0.015	12.92
60	12.0	0.60	0.008	0.789	1142.489	0.016	14.07
70	13.0	0.70	0.009	0.921	1143.981	0.017	15.23
80	14.0	0.80	0.011	1.053	1145.472	0.019	16.38
90	14.0	0.90	0.012	1.184	1146.964	0.019	16.36
100	14.5	1.00	0.013	1.316	1148.455	0.019	16.92
110	15.3	1.10	0.014	1.447	1149.947	0.021	17.83
120	15.3	1.20	0.016	1.579	1151.438	0.021	17.81
130	16.0	1.30	0.017	1.711	1152.930	0.021	18.60
140	17.0	1.40	0.018	1.842	1154.421	0.023	19.73
150	17.5	1.50	0.020	1.974	1155.913	0.023	20.29
160	18.0	1.60	0.021	2.105	1157.404	0.024	20.84
170	19.0	1.70	0.022	2.237	1158.896	0.025	21.97
180	19.7	1.80	0.024	2.368	1160.387	0.026	22.75
190	20.5	1.90	0.025	2.500	1161.879	0.027	23.64
200	21.5	2.00	0.026	2.632	1163.370	0.029	24.76
210	22.0	2.10	0.028	2.763	1164.862	0.029	25.31
220	23.0	2.20	0.029	2.895	1166.353	0.031	26.42
230	23.5	2.30	0.030	3.026	1167.845	0.031	26.96
240	24.5	2.40	0.032	3.158	1169.336	0.033	28.08
250	25.0	2.50	0.033	3.289	1170.828	0.034	28.61
260	25.5	2.60	0.034	3.421	1172.319	0.034	29.15
270	26.0	2.70	0.036	3.553	1173.811	0.035	29.68
280	27.0	2.80	0.037	3.684	1175.302	0.036	30.78
290	28.0	2.90	0.038	3.816	1176.794	0.038	31.88
300	29	3.00	0.039	3.947	1178.285	0.038	32.41
340	33	3.40	0.045	4.474	1184.251	0.044	37.34
380	37	3.80	0.050	5.000	1190.217	0.050	41.66
420	41	4.20	0.055	5.526	1196.183	0.054	45.37
460	46	4.60	0.061	6.053	1202.149	0.062	51.27
500	50	5.00	0.066	6.579	1208.115	0.067	55.46

540	55	5.40	0.071	7.105	1214.081	0.074	60.70
580	59	5.80	0.076	7.632	1220.047	0.079	64.80
620	64	6.20	0.082	8.158	1226.013	0.086	69.95
660	68	6.60	0.087	8.684	1231.979	0.090	73.42
700	72	7.00	0.092	9.211	1237.945	0.096	77.94
740	76	7.40	0.097	9.737	1243.911	0.102	81.87
780	78	7.80	0.103	10.263	1249.877	0.105	83.62
790	78	7.90	0.104	10.395	1251.369	0.105	83.52
800	78	8.00	0.105	10.526	1252.860	0.105	83.43
810	78	8.10	0.107	10.658	1254.352	0.105	83.33
820	78	8.20	0.108	10.789	1255.843	0.105	83.23
830	77	8.30	0.109	10.921	1257.335	0.103	82.06
840	76	8.40	0.111	11.053	1258.826	0.102	80.90
850	75	8.50	0.112	11.184	1260.318	0.101	79.74

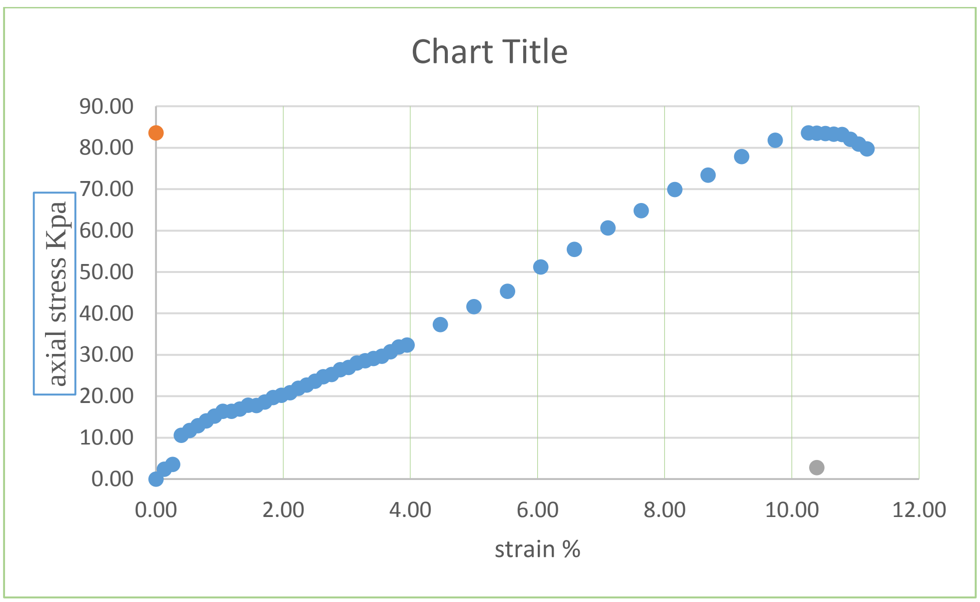


Figure: A 4.2: Axial stress versus percent strain curve for untreated soil

Unconfined Compressive Strength (qu),kPa = 83.62

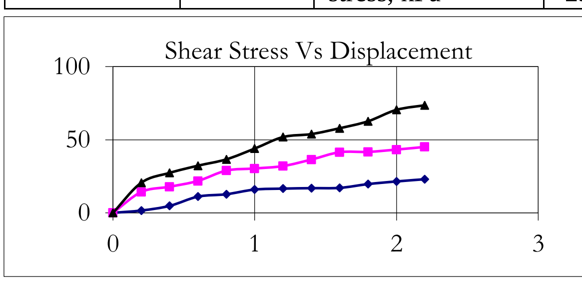
Undrained Shear Strength (cu), kPa = 41.81

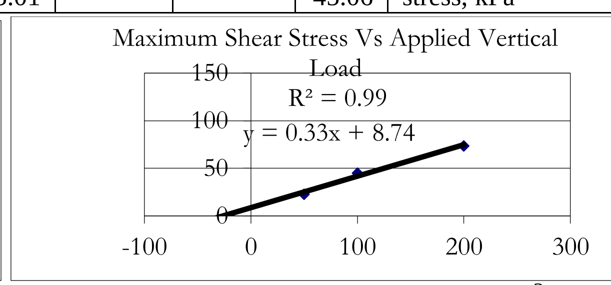
Appendix A.5: Direct Shear Test

Untreated

Table: A 5.1: Reading from deformation deal gauge load deal gauge, and calibration factor of the machine for untreated sample.

Sample No.		TP1		Sample Source:		Test Pit		Depth, m :		-6.00
Thickness of sample:		20 mm		Ring Calibration Factor:		1.428 N/div		Wet unit weight, kN/m^3:		19.08
Length of sample :		60 mm		Rate of strain :		1.6 mm/min		Dry Unit Weight,kN/m^3		14.56
Width of sample:		60 mm		Moisture content, %		31.0		Sample Condition:		untreated
			Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress	
			50 kpa			100 kpa			200 kpa	
Horizontal Displacement [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0.0	3600	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.2	3600	4.00	5.71	1.59	36.00	51.41	14.28	52.00	74.26	20.63
0.4	3600	12.00	17.14	4.76	45.00	64.26	17.85	69.00	98.53	27.37
0.6	3600	28.00	39.98	11.11	55.00	78.54	21.82	81.40	116.24	32.29
0.8	3600	32.10	45.84	12.73	72.80	103.96	28.88	92.30	131.80	36.61
1.0	3600	40.20	57.41	15.95	76.45	109.17	30.33	110.90	158.37	43.99
1.2	3600	41.80	59.69	16.58	80.60	115.10	31.97	130.80	186.78	51.88
1.4	3600	42.60	60.83	16.90	91.80	131.09	36.41	135.90	194.07	53.91
1.6	3600	43.10	61.55	17.10	104.50	149.23	41.45	146.00	208.49	57.91
1.8	3600	49.60	70.83	19.67	105.00	149.94	41.65	158.00	225.62	62.67
2.0	3600	54.20	77.40	21.50	109.00	155.65	43.24	177.70	253.76	70.49
2.2	3600	58.00	82.82	23.01	113.60	162.22	45.06	185.55	264.97	73.60
		Maximum shear stress, kPa		23.01			45.06	Maximum shear stress, kPa		73.60



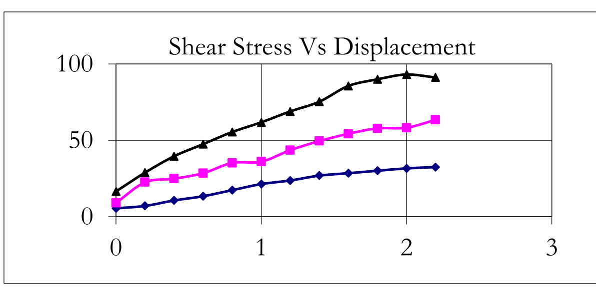


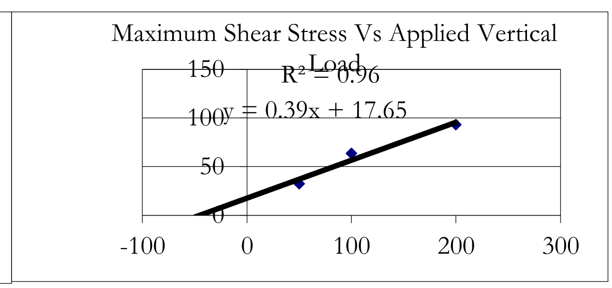
Angle of internal friction, 18.9 Cohesion , (kN/m^2) = 12

Figure: A 5.1: Diagram to determine friction angle and cohesion untreated sample *Lime treated soil*

Table: A 5.2: Reading from deformation deal gauge load deal guage, and calibration factor of the machine for lime treated sample.

Sample No.		TP1		Sample Source:		Test Pit		Depth, m :		-6.00
Thickness of sample:		20 mm		Ring Calibration Factor:		0.70 N/div		Wet unit weight, kN/m^3:		21.26
Length of sample :		60 mm		Rate of strain :		1.6 mm/min		Dry Unit Weight,kN/m^3		17.46
Width of sample:		60 mm		Moisture content, %		21.7		Sample Condition:		Disturbed
			Applied Vertical Stress			Applied Vertical Stress			Applied Vertical Stress	
			100 kpa			200 kpa			300 kpa	
Horizontal Displace [mm]	Corrected Area [mm2]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]	Proving Ring Reading	Shear Load [N]	Shear Stress [kPa]
0.0	3600	14.00	19.99	5.55	23.00	32.84	9.12	42.00	59.98	16.66
0.2	3600	18.00	25.70	7.14	57.00	81.40	22.61	73.00	104.24	28.96
0.4	3600	27.00	38.56	10.71	63.00	89.96	24.99	100.00	142.80	39.67
0.6	3600	34.00	48.55	13.49	72.00	102.82	28.56	120.00	171.36	47.60
0.8	3600	44.00	62.83	17.45	89.00	127.09	35.30	140.00	199.92	55.53
1.0	3600	54.00	77.11	21.42	91.00	129.95	36.10	156.00	222.77	61.88
1.2	3600	60.00	85.68	23.80	110.00	157.08	43.63	174.00	248.47	69.02
1.4	3600	68.00	97.10	26.97	125.00	178.50	49.58	190.00	271.32	75.37
1.6	3600	72.00	102.82	28.56	137.00	195.64	54.34	216.00	308.45	85.68
1.8	3600	76.00	108.53	30.15	146.00	208.49	57.91	227.00	324.16	90.04
2.0	3600	80.00	114.24	31.73	147.00	209.92	58.31	235.00	335.58	93.22
2.2	3600	82.00	117.10	32.53	160.00	228.48	63.47	230.00	328.44	91.23
		Maximum shear stress, kPa		32.53			63.47	Maximum shear stress, kPa		93.22





Angle of internal friction, 21.3°

Cohesion, c (kN/m^2) = 18

Figure: A 5.2: Diagram to determine friction angle and cohesion for lime treated sample

Appendix B: Steps for Mixture Design and Testing for Lime Stabilized Soil

National lime association has put certain steps on how to proceed the laboratory test for lime treated soil sample which is described here:

Step 1 – Initial Soil Evaluation

Purpose: Evaluate key soil characteristics as an initial step to determine if it is suitable for lime stabilization.

Procedure: Use ASTM C136 [10] procedures to determine the amount of soil passing the 75 micron (75-µm) screen and ASTM D 4318 to determine the soil plasticity index (PI).

Criteria: Generally, soil with at least 25% passing a 75 micron screen and having a PI of 10 or greater are candidates for lime stabilization. Some soils with lower PI can be successfully stabilized with lime, provided the pH and strength criteria described in this document can be satisfied.

Additional Considerations: Soil with organics content above 1-2% by weight as determined by ASTM D 2974 may be incapable of achieving the desired unconfined compressive strength for lime stabilized soil (Step 6). Soils containing soluble sulfates greater than 0.3% can be successfully stabilized with lime, but may require special precautions (see NLA's "Technical Memorandum – Guidelines for Stabilization of Soils Containing Sulfates"

Step 2 – Determine the Approximate Lime Demand

Purpose: Determine the minimum amount of lime required for stabilization.

Procedure: Use ASTM D 6276 procedures. This is also known as the "Eades-Grim" test.

Criteria: The lowest percentage of lime in soil that produces a laboratory pH of 12.4 [flat section of the pH vs. lime percentage curve produced by the test] is the minimum lime percentage for stabilizing the soil.

Additional Considerations: ASTM D 6276 has additional provisions for cases in which the measured laboratory pH is 12.3 or less. Note that lime can react with moisture and carbon dioxide. Careful storage is required to maintain lime's integrity and produce reliable results.

Step 3 – Determine Optimum Moisture Content and Maximum Dry Density of the Lime-Treated Soil

Purpose: Determine optimum moisture content (OMC) and maximum dry density (MDD) of the soil after lime has been added. This is necessary because adding lime will change the soil's OMC and MDD.

Procedure: Make a mixture of soil, lime, and water at the minimum percentage of lime as determined from Step 2 (Eades-Grim test), using a water content of OMC + 2-3%. Seal the mixture in an airtight, moisture proof bag stored at room temperature for 1-24 hours. Determine the OMC and MDD of the mixture using ASTM D 698 procedures (standard compaction effort).

Criteria: Determine the OMC and MDD for Step 4.

Additional Considerations: When using quicklime, the mixture should be stored for 20-24 hours to ensure hydration.

Step 4 – Fabricate Unconfined Compressive Strength (UCS) Specimens

Purpose: Fabricate test specimens for UCS testing (Step 6).

Procedure: Using ASTM D 5102 procedure B, fabricate a minimum of two test specimens of lime, soil and water using the amount (percentage) of lime determined from Step 2 at the OMC (± 1%) as determined from Step 3. The soil-lime-water mixture should be stored in an airtight, waterproof bag for 1-24 hours prior to fabricating the test specimens.

Desired Result: A minimum of two specimens for UCS testing

Additional Considerations: When using quicklime, the mixture should be stored for 20-24 hours to ensure hydration. Additional specimens may be fabricated if additional testing is desired. In some cases it may be advisable to make test specimens at higher lime content(s) than that determined from ASTM D 6276 testing (Step 2). These additional specimens can be used to determine the UCS of lime-soil-water mixtures at higher lime contents. For instance, if ASTM D 6276 testing (Step 2) indicates that 4% lime is needed, additional UCS testing could be done at 5% and 6% lime to ensure that the UCS criteria (Step 6) is also achieved.

Step 5 – Cure and Condition the Unconfined Compressive Strength (UCS) Specimens

Purpose: Approximate, in an accelerated manner, field curing and moisture conditions.

Procedure: Immediately following the fabrication of the test specimens, wrap the specimens in plastic wrap and seal in an airtight, moisture proof bag. Cure the specimens for 7 days at 40°C. Subject the specimens to a 24 hour capillary soak prior to testing. The capillary soaking process should be done by removing the specimens from the airtight bag, then removing the plastic wrapping. The specimens are wrapped with wet absorptive fabric and placed on a porous stone. The water level should reach the top of the stone and be in contact with the fabric wrap throughout the capillary soak process, but the soil specimen should not come directly into contact with the water.

Desired Result: A minimum of two cured and moisture conditioned specimens for UCS testing.

Step 6 – Determine the Unconfined Compressive Strength (UCS) of the Cured and Moisture Conditioned Specimens

Purpose: To determine the UCS of the lime-stabilized soil to ensure adequate field performance in a cyclic freezing and thawing and an extended soaking environment.

Procedure: Use ASTM D 5102 procedure B to determine the UCS of the cured and moisture conditioned specimens. The UCS is the average of the test results for a least two specimens.

Criteria: The minimum desired UCS depends on the intended use of the soil, the amount of cover material over the stabilized soil, exposure to soaking conditions, and the expected number of freezing and thawing cycles during the first winter of exposure.

Appendix C: Calculations of consolidation test results

Table C 1 : Soil data for lime treated soil before and after the test.

Before Test					
			Diameter,D	50.00	mm
Weight of sample+Ring		119.40	Area, A	1963.50	mm^2
Weiht of Ring		60.00	Thickness, H_o	20.00	mm
Weight of sample		59.40	Volume	39.27	cm^3
Weiht of dry sample		48.00	Density	1.51	g/cc
Initial moisture content Mo		23.75	Dry Density	1.22	g/cc
Initial Void ratio	$e_{o=}$	1.08			
Initial saturation	$s_{0=}$	55.958165			
Volume change Faector F=		0.1039019			
After Test					
Weight of sample+ring		119.80	Overall settlement	0.4860	mm
Weight of dry sample+ring		108.00	Volume change	0.9543	cm^3
Weight of Ring		60.00	Final volume	38.3157	cm^3
Weight of wet sample		59.8	Final density	1.5607	g/cc
Weight of dry sample		48	Final Dry density	1.2527	g/cc
Weight of moisture		11.8	Final void ratio	1.03	
Final moisture content		24.58%			
Final saturation Sf=		60.77			

Table C 2: Dial gauge reading of consolidometer for lime treated soil.

Time	Sqrt Time	0.5kg	1xE-03	1kg	1xE-03	2kg	1xE-03	4kg	1xE-03	8kg
	0	0.000	0.000	0.261	261.000	0.279	279.000	0.321	321.000	0.409
0.25	0.50	0.240	240.000	0.265	265.000	0.300	300.000	0.367	367.000	0.410
1.00	1.00	0.242	242.000	0.269	269.000	0.372	372.000	0.372	372.000	0.411
2.25	1.50	0.243	243.000	0.269	269.000	0.304	304.000	0.375	375.000	0.415
4.00	2.00	0.245	245.000	0.269	269.000	0.308	308.000	0.378	378.000	0.419
6.25	2.50	0.245	245.000	0.269	269.000	0.308	308.000	0.379	379.000	0.419
9.00	3.00	0.246	246.000	0.270	270.000	0.309	309.000	0.380	380.000	0.412
12.50	3.54	0.247	247.000	0.289	289.000	0.310	310.000	0.380	380.000	0.413

16.00	4.00	0.248	248.000	0.270	270.000	0.310	310.000	0.381	381.000	0.418
25.00	5.00	0.248	248.000	0.270	270.000	0.310	310.000	0.383	383.000	0.421
36.00	6.00	0.249	249.000	0.270	270.000	0.311	311.000	0.384	384.000	0.422
49.00	7.00	0.250	250.000	0.271	271.000	0.313	313.000	0.384	384.000	0.450
64.00	8.00	0.250	250.000	0.275	275.000	0.315	315.000	0.385	385.000	0.480
81.00	9.00	0.250	250.000	0.275	275.000	0.320	320.000	0.385	385.000	0.490
100.00	10.00	0.250	250.000	0.276	276.000	0.320	320.000	0.386	386.000	0.510
121.00	11.00	0.251	251.000	0.276	276.000	0.320	320.000	0.387	387.000	0.520
225.00	15.00	0.257	257.000	0.278	278.000	0.321	321.000	0.389	389.000	0.527
400.00	20.00	26.000	26000.000	0.279	279.000	0.321	321.000	0.401	401.000	0.530
1440.00	37.95	0.261	261.000	0.279	279.000	0.321	321.000	0.409	409.000	0.538
Final		0.261		0.279		0.321		0.409		0.538
Change		0.261		0.018		0.042		0.088		0.129
Correction		0.018		0.024		0.031		0.040		0.052
Settlement, Del H		0.243		0.255		0.290		0.369		0.486

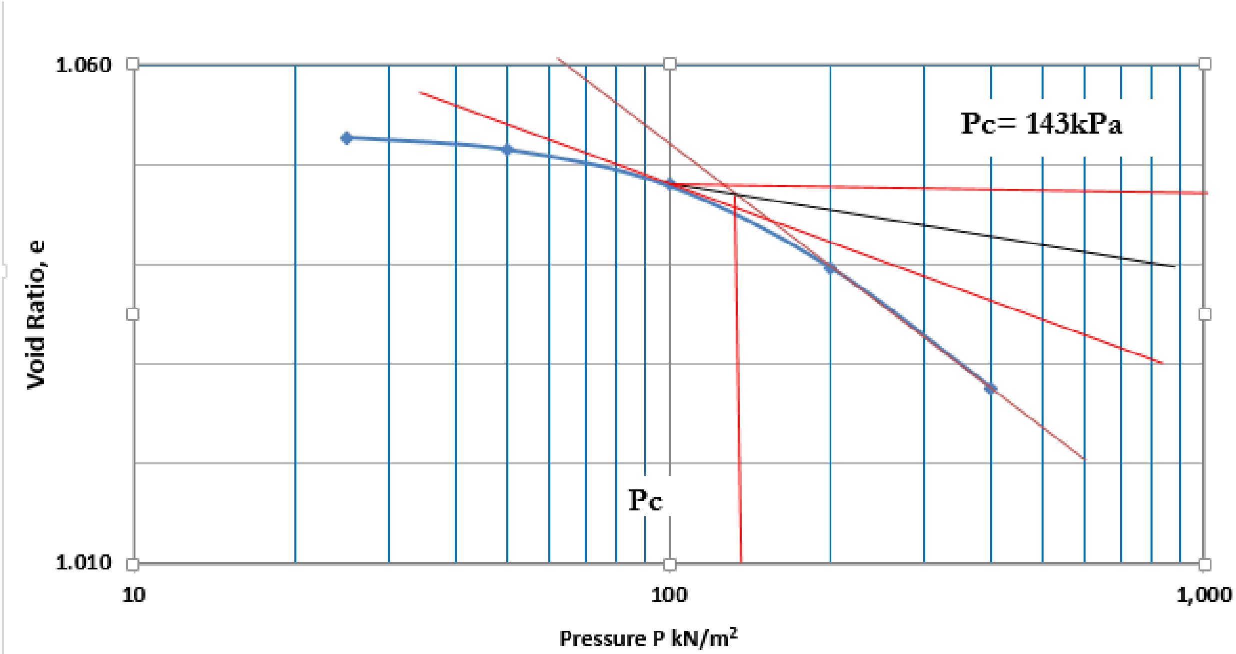


Figure C 1: void ratio verses pressure curve for the determination of preconsolidated pressure, casagrande method. For the case of lime treated soil.

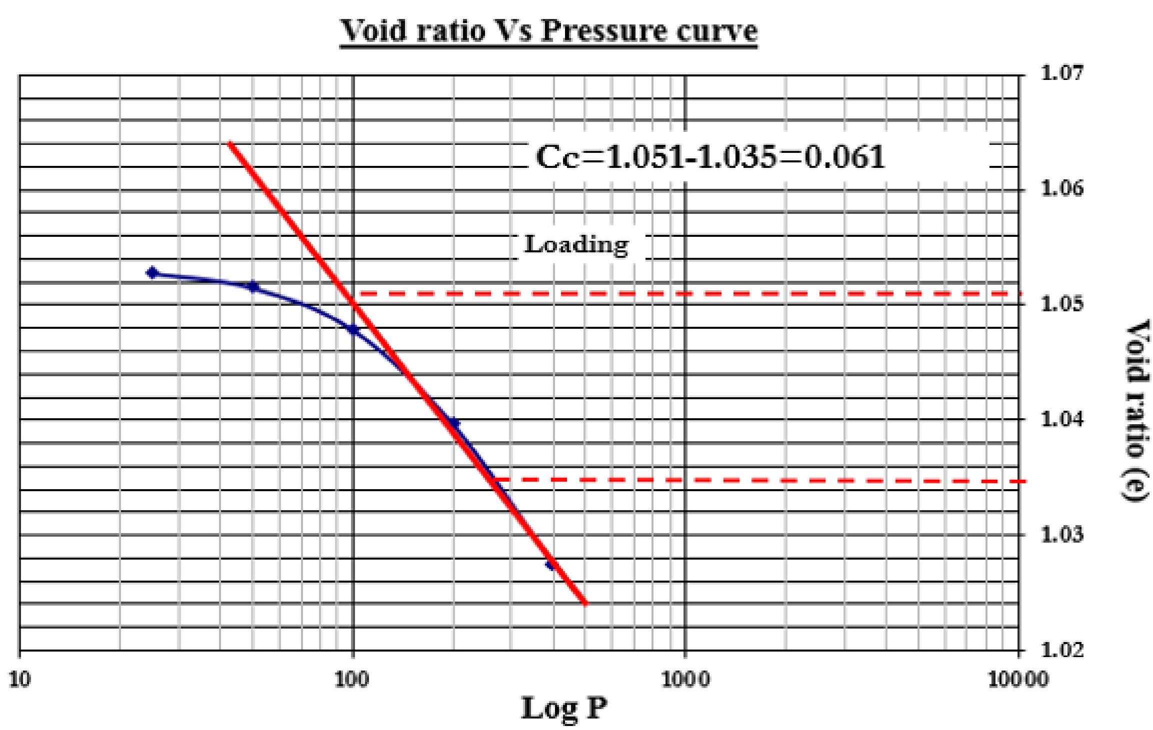


Figure C 2: void ratio versus pressure curve for lime treated soil.

Untreated soil

Table C 3: Dial gauge reading of consolidometer for untreated sample.

Pressure intensity	BH No :	TP-1	Depth	6	m							
time(min)	Sqrt Time	0.14kg	1xE-03	0.25kg	1xE-03	0.5kg	1xE-03	1kg	1xE-03	2kg	1xE-03	4kg
0	0	0.250	250	0.430	430	0.530	530	0.710	710.000	1.000	1000.000	1.295
0.25	0.50	-		0.403	403	0.590	590	0.860	860.000	1.100	1100.000	1.280
0.50	0.71	-		0.410	410	0.610	610	0.870	870.000	1.120	1120.000	1.290
1.00	1.00	-		0.413	413	0.660	660	0.870	870.000	1.125	1125.000	1.290
2.00	1.41	-		0.419	419	0.685	685	0.885	885.000	1.128	1128.000	1.200
4.00	2.00	-		0.420	420	0.690	690	0.950	950.000	1.130	1130.000	1.152
8.00	2.83	-		0.423	423	0.710	710	0.975	975.000	1.130	1130.000	1.170
15.00	3.87	-		0.429	429	0.736	736	0.981	981.000	1.160	1160.000	1.186
30.00	5.48	-		0.430	430	0.750	750	0.999	999.000	1.165	1165.000	1.205
60.00	7.75	-		0.435	435	0.760	760	1.050	1050.000	1.170	1170.000	1.222
120.00	10.95	-		0.441	441	0.764	764	1.082	1082.000	1.176	1176.000	1.243
240.00	15.49	-		0.443	443	0.770	770	1.091	1091.000	1.180	1180.000	1.260
480.00	21.91	-		0.490	490	0.792	792	1.093	1093.000	1.189	1189.000	1.280
1440.00	37.95	0.430	430	0.530	530	0.710	710	1.000	1000.000	1.295	1295.000	1.659
Final		0.430		0.530		0.710		1.000		1.295		1.659
Change		0.180		0.100		0.180		0.290		0.295		0.364
Correction		0.018		0.024		0.031		0.040		0.052		0.053
Settlement,Del H		0.412		0.506		0.679		0.960		1.243		1.606

Table C 4: void ratio and coefficient of consolidation data for untreated sample.

Inc.no	Void Ratio				Volume Compressiblity				Coefficient of Consolidation					Compression Index, Cc
	Pressure P kN/m^2	Settlement (dH) mm	δe= F*dH	$e=e_0-\delta e$	Incremental Changes δe	δp kN/m^2	1+e	Mv= $\delta e/\delta p$*1000 / 1+e (m^2/kN)	t_{90} (min)	$H=H_0$-dH (mm)	$H_{ave.}$= (H1+H2) / 2 mm	$(H_{ave.})^2$ mm^2	Cv= 0.848*$(H_{ave.})^2$ / t_{90} (mm^2/min)	
	0	0	0.104	1.078	0	0				20				
1	7	0.412	0.043	1.035	0.0428	7	2.035	3.005	4.84	19.588	19.794	391.802	68.646	
2	12.5	0.506	0.053	1.025	0.0098	5.5	2.025	0.877	25.00	19.494	19.541	381.851	12.952	
3	25	0.679	0.071	1.007	0.0180	12.5	2.007	0.716	12.25	19.321	19.408	376.651	26.073	0.061
4	50	0.960	0.100	0.978	0.0292	25	1.978	0.590	11.56	19.04	19.181	367.892	26.987	
5	100	1.243	0.129	0.949	0.0294	50	1.949	0.302	10.20	18.757	18.899	357.153	29.693	
6	200	1.606	0.167	0.911		100	1.911							
7	100	0.000	0.000	0.000		-100	1.000							

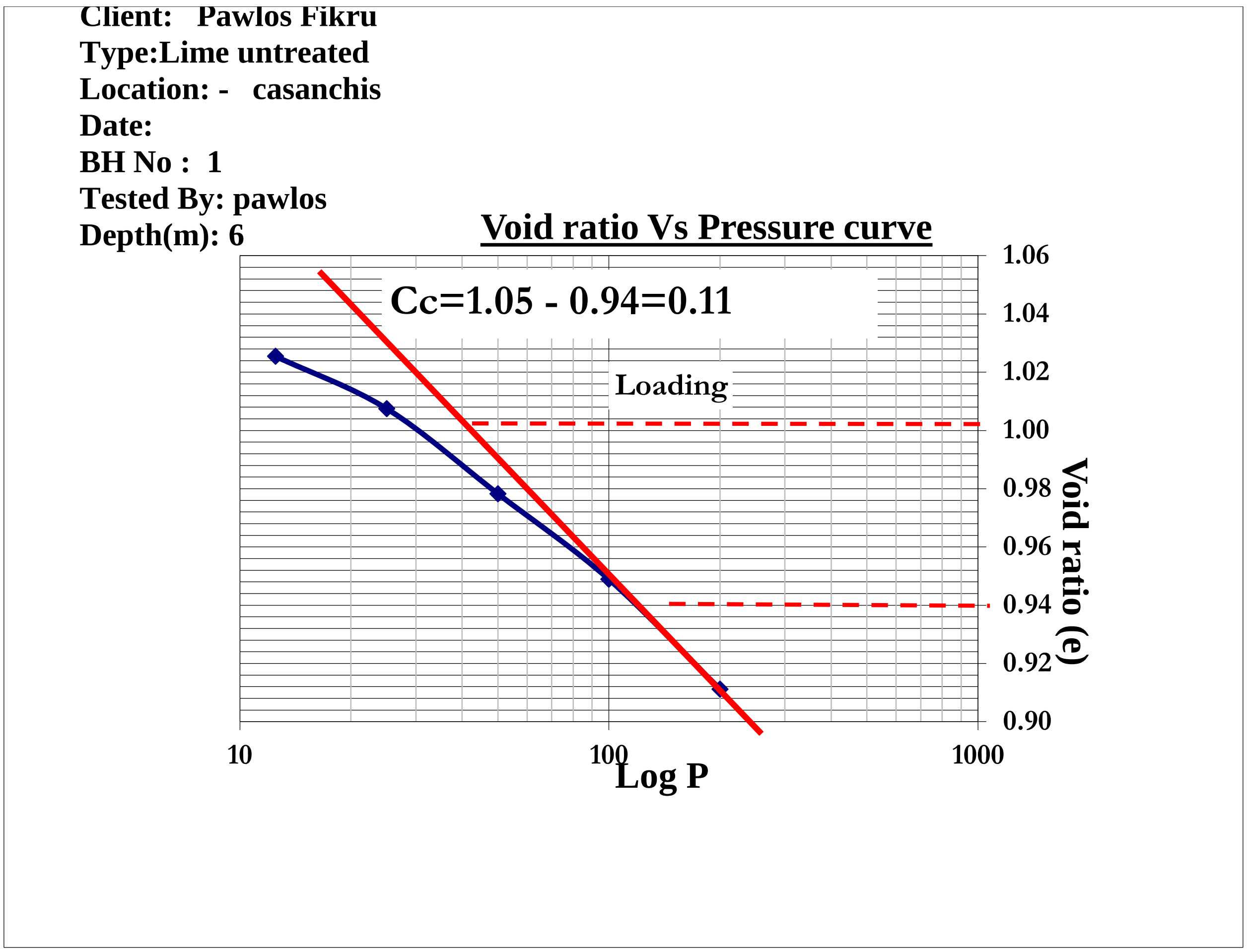


Figure C 3: void ratio versus pressure curve for untreated soil.

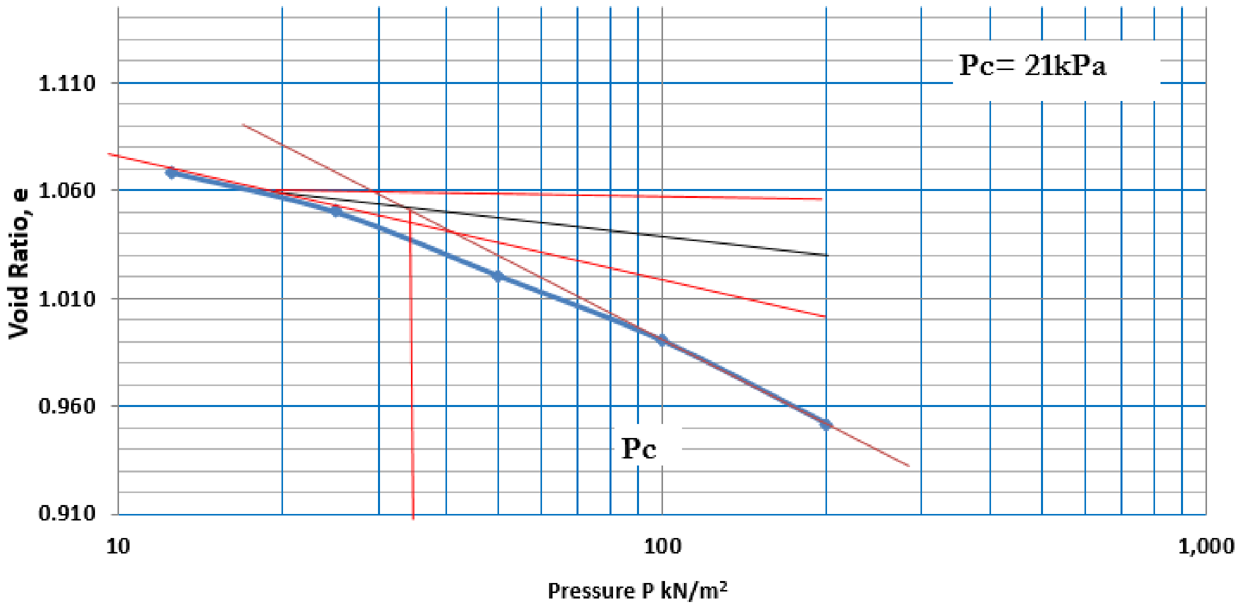


Figure C 4: void ratio verses pressure curve for the determination of preconsolidated pressure, casagrande method. For the case of untreated soil.

Appendix D: PH test procedure

This test method provides a means for estimating the soil-lime proportion requirement for stabilization of a soil. This test method is performed on soil passing the −425-µm (No. 40) sieve. The optimum soil-lime proportion for soil stabilization is determined by tests of specific characteristics of stabilized soil such as unconfined compressive strength or plasticity index.

1. Specimens

1.1 Prepare a representative sample of air-dried soil in accordance with Practice D 421. Soil may be oven-dried at atemperature#60°C.

1.2 Pass 350 g of material through the 425-µm (No. 40) sieve.

1.3 Thoroughly mix the material passing the 425-µm (No.40) sieve.

1.4 Determine the water content, in accordance with Test Method D 2216, of a representative specimen of the material obtained in 1.3. Place the remaining material obtained in 1.3 in an airtight container to preserve the moisture content until the procedure described in Section 2 is performed.

2. Calibration and Standardization

2.1 Calibrate the pH meter in accordance with the manufacturer's instructions using a pH 12 buffer solution at 25±1°C.

2. Procedure

2.1 Specimen Preparation:

2.1.1 using the air-dried sample in accordance with Section 1, obtain five specimens, each equivalent to 25.0 g of oven-dried soil. Splitting or other appropriate means should be used to obtain each of the five specimens.

2.1.2 Determine the mass of each air-dried soil specimen equivalent to 25.0 g of oven-dry soil as follows:

$Ma = 25 * (1 + W/100)$

where: Ma = mass of air-dried soil specimen, and

W = water content, %, of air-dried sample determined in 1.4.

2.1.3 Place each specimen into dry plastic bottles and cap tightly.

2.1.4 Obtain six representative specimens of lime meeting the requirements of Specification C 977. Five specimens are representative of 2, 3, 4, 5, and 6% of the equivalent 25-goven-dried soil mass. The sixth specimen of 2.0 g of lime represents a saturated lime solution. Place the 2.0 g of lime into a dry plastic bottle and cap tightly.

2.1.5 Add one of the first five lime specimens to one of the soil specimens in plastic bottles, cap tightly, mark the percentage on the bottle, and mix thoroughly by shaking. Repeat this procedure for the remaining four lime and soil specimens.

2.1.6 Add 100 mL of water to each of the soil-lime mixtures and to the bottle containing 2.0 g lime.

2.1.7 Cap the bottles and shake each of the soil-lime-water and lime-water mixtures for a minimum of 30 s or until the specimens are thoroughly mixed. Continue to shake the specimens for 30 s every 10 min for 1 h.

2.2 If necessary, heat or cool the specimen as needed to bring the temperature of the specimen to 25 ± 1°C.

2.3 Within 15 min of the end of the 1-h shaking period, determine the pH of each soil-lime-water and the lime-water mixture of 0.01 pH units. Maintain the temperature of the mixture at 25 ± 1°C when determining pH.

2.4 Record the pH value for each soil-lime-water mixture and for the lime-water mixture. (ASTM D 6276 - 99a, pp 2 to 3)