Jimma University

Jimma Institute of Technology

School of Graduate Studies

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Stream

Deformation Characteristics of Cement Modified Soft Clay Soil for Shallow Foundation Using Finite Element Method

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

By:

Meaza Koirita

June 2021

Jimma, Ethiopia

Jimma University

Jimma Institute of Technology

School of Graduate Studies

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Stream

Deformation Characteristics of Cement Modified Soft Clay Soil for Shallow Foundation Using Finite Element Method

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

By:

Meaza Koirita

Main Advisor: Damtew Tsige (Ph.D.)

Co-Advisor: Adamu. Beyene (MSc)

June 2021

Jimma, Ethiopia

DECLARATION

I, the undersigned, declare that the thesis entitled **"Deformation Characteristics of Cement Modified Soft Clay Soil for Shallow Foundation Using Finite Element Method"** is my original work and has not been presented by any other person for an award of a degree in this or any other university, and all sources of material used for this thesis have to be duly acknowledged.

Meaza Koirita	________________	_____________
[Researcher]	[Signature]	[Date]

This research paper has been submitted for examination with my approval as university supervisor.

Damtew Tsige (Ph.D.)	________________	_____________
[Advisor]	[Signature]	[Date]
Adamu Beyene (MSc)	________________	_____________
[Co-advisor]	[Signature]	[Date]

APPROVAL

The thesis entitled **"Deformation Characteristics of Cement Modified Soft Clay Soil for Shallow Foundation Using Finite Element Method"** by Meaza Koirita, is approved for the degree of Master of Science in Geotechnical Engineering.

Name	Signature	Date
Main adivisor: Damtew Tsige	____________________	__________________
Co-adivisor: Adamu Beyene	____________________	__________________

Approved by Board Examiners

1. Dr. Endalu Tadele	____________________	__________________
[External Examiner]	[Signature]	[Date]
2. Bereket Mamo	____________________	__________________
[Internal Examiner]	[Signature]	[Date]
3. Alemneh Sorsa	____________________	__________________
[Chair Person]	[Signature]	[Date]

ACKNOWLEDGMENTS

Before all things, I would like to thank almighty God for his very existence in every moment of my life and giving me health and strength to reach this stage.

Secondly, I would like to take this opportunity to express my gratitude to a number of peoples.

- My deep sense of gratitude and profound thanks to my supervisor Damtew Tsige (Ph.D.) and Adamu Beyene (MSc) for his continuous support, enthusiasm, patience, encouragement and professional guidance throughout the research.
- I acknowledge Jimma University, Jimma Institute of Technology, Faculty of civil and environmental Engineering and Ethiopian Road Authority (ERA) for providing me the opportunity to study MSc. Degree in Geotechnical Engineering.
- I would also like to thank the Geotechnical laboratory technicians who have offered their skills throughout the labor-intensive test procedures.
- It is my pleasure to thank my postgraduate classmates who would create a joyful atmosphere and never lay down helping hand when it was needed.
- Finally, my special thanks remain to my parents (Mum and Dad) and my husband (Mr. Amanuel Tesfaye lecture at Jimma university department of mechanical Engineering). I can’t able to describe the value of your presence, love and kind support during each and every stage of my works. You have always realized the importance of my work and for that reason I had motivated to concentrate on my work.

ABSTRACT

The larger area of the world is covered with soft clay; the soil type known for its high susceptibility to a shear failure, unexpected differential settlement, and experiencing low bearing capacity which makes it be categorized as unsuitable soil for variety of engineering use. Yet, it remained cumbersome to suitably use soft clay soil because of its high vulnerability to the subsequent foundation failures; which is a headache to construction stakeholders especially in the contemporary engineering world. Similarly, majority of Jimma town's land cover is predominantly known for its deposit of soft clay soil which is conventionally excavated up to a certain depth and discarded ahead of actual commencement of construction projects due to its stability concern. The usage of soft clay soil as foundation of different engineering structures is excavated unless its intrinsic properties are enhanced through application of appropriate improvement mechanisms among which the cement stabilization approach is one. Hence, the study aimed at studying deformation analysis of cement modified soft clay by using finite element method. The soil samples were collected from three different pits located in Jimma town and laboratory tests were conducted to determine index properties, unconfined compressive strength, one-dimensional consolidation characteristics and shear strength parameters of the soil samples. To improve these engineering properties of the clay soil a varying percentage of cement in the mix by 9 %, 12 %, and 15 % were applied. Besides, the deformation analysis of the soil was assessed by using Plaxis 2D software which operates with the principle of Finite Element Method. Therefore from finite element analysis the vertical displacement encountered in untreated soft clay was 124 mm and the displacement of 15% of cement treated of soil is 26.77 mm. It is observed that the displacement is improved when soft clay soil is modified with 15% of cement. There was a high difference in the magnitude of untreated and treated soft soils this is a result of soil cement reaction and effect of curing period of the specimen.

Key words: soft clay, cement, finite element method, Hardening soil model

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Official
ASTM	American Society of Testing Materials
c	Cohesion
Cc	Compression index
Cs	Swelling index
Cu	Shear strength
CSC	Cement Stabilized Clay
e	Void ratio
E_{50}^{ref}	Secant stiffness to describe plastic straining due to primary diavotric loading
E_{oed}^{ref}	Secant stiffness to describe plastic straining due to primary compression
E_{ur}^{ref}	Stiffness to describe elastic unloading/reloading behavior
FEM	Finite element method
LL	Liquid limit
OPC	Ordinary Portland cement
Pc	Pre-consolidation pressure
PL	Plastic limit
PI	Plasticity index
UCS	Unconfined compressive strength
ϕ	Internal frictional angle
ψ	Dilatancy Angle
σ	Stress
ε	Strain

CHAPTER ONE

INTRODUCTION

1.1. Background of the study

In Civil Engineering, all projects are built into the ground which needs a foundation with sufficient bearing capacity of the soil. In addition, the bearing capacity of the soil for civil engineering structures not needed to vary conditionally. Infrastructures and constructions which are founded on soils in which their strength changes conditionally are most susceptible to failure which may range to a different extent (Nelson et al., 2015). Soft to very soft clays are mostly associated with substantial difficulties. Since these type soils are sensitive to deformations and have very small shear strength, they may result in structural damage during the execution as well as throughout the life of the projects especially in the urban area (Gebreselassie & Kempfert, 2006).

Such type of soil needs either replacement with other suitable soil or treatment with suitable mechanism to acquire enough bearing capacity and strength to support the load imposed upon it. Soil strength generally refers to the soil ability to support the load imposed by building or structures perfectly without failure. Treated soft soil behaves differently with different loads resulting in varying degrees of initial strength gain and final strength development to support foundation for building purposes (Kalantari, 2012).

Jimma soils are problematic soils that need treatment (Muhammed & Teferra, 2014). Therefore, the research was done to evaluate the deformation analysis by overlying square footing on Jimma soft Clay which was stabilized with cement as an admixture.

Cement is composed of calcium-aluminates and calcium-silicates, when combined with water hydrate it forms the cementing compounds of calcium aluminate-hydrate and calcium-silicate-hydrate, as well as excess calcium hydroxide. Because of cementitious material, as well as the formed calcium hydroxide (lime), cement may be successful in stabilizing both granular and fine-grained soils, aggregates and miscellaneous materials (Onyelowe et al., 2019). In this study, Portland cement used to modify the problematic soils to analyze the deformation by using finite element method in PLAXIS 2D software.

In different geotechnical applications, PLAXIS, a two-dimensional finite element based the computer program is used for simulation purposes. This sophisticated computer program is mainly used for the analysis of deformation and stability. It has an advantageous feature that enables users to choose different soil models, which is dependent on mechanical deformation behaviors of soil, for the simulation (Ismail et al., 2011). Therefore, the study aimed to evaluate the deformation analysis of cement-modified soft clay by using the finite element method in PLAXIS software.

1.2. Statement of the problem

The larger area of the world is covered with soft clay; the soil type is known for its high susceptibility to a shear failure, unexpected differential settlement, and experiencing low bearing capacity which makes it be categorized as unsuitable soil for a variety of engineering use. Yet, it remained cumbersome to suitably use soft clay soil because of its high vulnerability to the subsequent foundation failures; which is a headache to construction stakeholders especially in the contemporary engineering world. (Al-Khafaji et al.,2017). Soils are abundant material that play a main role in the design, construction, and performance of civil engineering structures. However, all soils are not appropriate for all applications due to their mechanical, physical, and chemical properties, as well as constructability. Soil are characterized in several ways, and one of which is their designation as soft. Soft clay has relatively low stiffness and a high level of deformation. When load has applied, the deformation of soft clay is high due to its relatively low stiffness. Therefore, the deformation analysis should consider soft clay as a large-deformation continuum approach (Teunissen & Zwanenburg, 2017).

Ethiopia has a large part of thick deposits of soft clay, but there is no detailed study on their properties. Above 40% of the country is covered with soft soil, which is causing construction challenges and slowing down economic development (Rezaei et al., 2012). The construction built over these deposits can result in premature distresses, such as excessive settlement, rutting in the wheel paths, cracking, and potholes (Sorsa et al., 2020).

Majority of Jimma town's land cover is predominantly known for its deposit of soft clay soil which is conventionally excavated up to a certain depth and discarded ahead of the actual commencement of construction projects due to its stability concern. The usage of soft clay soil as the foundation of different engineering structures is excavated unless its intrinsic properties are enhanced through the application of appropriate improvement mechanisms among which the cement stabilization approach is one.

Therefore, the present study focuses on the deformation analysis of cement modified soft clay using finite element method a case of shallow foundation (square footing) overlying on both natural and stabilized soft clay with different added loads.

1.3. Research questions

Questions that would be answered through the research findings are;

- Can cement suitably be used to modify the soft clay soil?
- To what extent the deformation characteristics of soft clay is altered when treated with cement?

1.4. Objectives of the study

1.4.1. General objective

The general objective of this study is to investigate deformation characteristics of cement modified soft clay soil for shallow foundation using the Finite Element Method.

1.4.2. Specific objectives

The specific objectives of this study include:

- To determine swelling and compression properties of treated and untreated soft clay soils.
- The effectiveness of cement in modifying soft clay soils and recommending the optimum percentage of cement to be used.
- To assess the extent to which the deformation potential of soft clay is altered after treated with cement.

1.5. Significance of the study

The importance of this study is to provide a comprehensive solution to the problems of soft clay soil settlement and provided safety of any structures on soft and compressible soil. It gives detailed information on the deformation analysis of natural and cement-modified Jimma soft clay. So, the study was conducted to modify soft clay with different concentrations of cement to investigate the strength of soft clay by using FEM. Therefore, this study is important to the government and contractor to minimize the time for the construction of civil works. Furthermore, the results of this study can serve as a source of information for further investigations in future studies.

1.6. Scope of the study

The study focuses on the deformation analysis of cement-modified soft clay by using the finite element method. Plaxis hardening soil constitutive model was used for both natural soft soil and treated soft soils. A Laboratory test was conducted like; index properties, unconfined compressive strength, one- dimensional consolidation test, and triaxial test. For one- dimensional consolidation test and triaxial test the sample was mix and remolded with 9, 12, and 15 percent of cement for simulation of deformation analysis. The research also aimed to determine the effect of cement on the consolidation properties of soft clay. PLAXIS 2D software was used for the analysis of deformation characteristics for both natural and blended soil. Finally, the result from analysis and discussion, conclusion, and recommendation are discussed.

1.7. Organization of the thesis

This thesis work is divided into five Chapters, each covering a specific topic of the research work. In the first Chapter background of the study, the objective of the research, the scope of the investigation, and the organization of the thesis are presented. The Literature review is presented in Chapter two. Chapter three addresses the study area, research methodology, material, and type of tests proposed to be conducted. Laboratory tests result and finite element results go to chapter four. The last Chapter, Chapter five, is devoted to conclusions and recommendations. Detailed test results are presented in the appendices.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

Soft clay as a foundation soil is met all over the world, being deposited mainly as sedimentary soils under alluvial, marine, lacustrine, or similar environments. High compressibility and low shear strength characterize weak and compressible (soft) soils. They are fine-grained soils are highly plastic with moderate to high clay fractions in nature. They give rise to problems of settlement and stability even under a small-superimposed load. Consequently, infrastructures like buildings, bridges, and underground construction in soft clay give rise to problems of design and construction (Dixit, 2016). soft soils include clay and silty clay soil which have the characteristics of high water content, low strength, high compressibility, and bad penetrability (Jiang, 2011).

When soil is exposed to a certain amount of load it tends to deform in the direction of the application of the load. The type and value of deformation differ from one soil to another. For instance, soft clay tends to deform differently when compared to stiff soil through the application of the same amount of load. The deformation property of soil is dependent on the origin of the soil, the structure of the soil particles, the bond between particles, water content, and so on (Ismail et al., 2011). Soils with characteristics of low strength and compressibility exist all over the world. One of the most significant problem arises due to characteristics of the soil is difficulties in supporting loads on a foundation and low strength in guaranteeing the stability of the structures.

Therefore, the stabilization of soil is important to be done in the construction. The importance of soil stabilization is to modify the soil, expedite construction, and improve the strength and durability of the soil. Besides that, soil stabilization can be defined as the modification or improvement of the characteristics of the soil to enhance the engineering performance of the soil. For example, improve the density of soil, mixing the soil with additives to change the chemical and physical properties such as stiffness, compressibility, permeability, workability, lower the groundwater level, and eliminate weak soil (Diyana et al., 2010).

2.2. Types of Settlement in soft soils.

Settlement is an important criterion in the design of the foundations. Foundation settlement must be estimated carefully to ensure the stability of buildings, bridges, towers, and any high-cost structures. The compressive deformation of the soil is the main reason for the occurrence settlement. According to (Towhata & Ikuo, 2009), settlement is classified as Immediate (Elastic) settlement and Consolidation Settlement.

2.2.1. Immediate (Elastic) Settlement

The settlement takes place rapidly after adding loads without any change in the moisture content and volume.

2.2.2. Consolidation Settlement

This type of soil settlement includes two phases:

i) **Primary consolidation phase:** This phase is a consolidation settlement resulted from the change of volume due to water extrusion from soil voids. It occurs in saturated cohesive soils and the change happens slowly which takes place over a long time (Al-Taie et al., 2016). Soil settlement consists of primary and secondary consolidation. With time, the excess pore water pressure dissipates as drainage occurs and the clay under goes further settlement due to volume changes as stress is from pore pressure to effective stress. The rate of change in volume and corresponding settlement is governed by how fast the water can drain out of from the clay under the induced hydraulic gradients (Chin, 2005).

ii) **Secondary consolidation phase**: This phase is a compression settlement which consider as a plastic adjustment observe in saturated cohesive soils. It is an extra deformation of soil that occurs due to adding incremental loads which follows the primary consolidation phase (Bowles, 1996).

The foundation soil settlement (immediate and consolidation) is estimated depending on the stresses in the mass of the soil related to foundation pressure. The settlements and stresses distribution are calculated with the assumption of the soil model as isotropic, homogeneous, and linearly elastic. They play an important role in the foundation's design. Immediate settlement is usually estimated according to elastic theory, which assumes the soil may behave elastically under stresses at any point in soil mass (Ramthan, 2012).

2.3. Consolidation Settlement Analysis

Soil is constituted from soil particles and pore system between developed soil particles. The Pore system can be filled by water, a small amount of water, or without water. When the soil is compressed in the cases of containing a small amount of water or without water, the first soil skeleton is deformed immediately, then the chain between soil particles is destroyed and soil particles move closely together, results in gradual reduction porosity and make the soil denser.

For the saturated soil, when soil is compressed, the behavior is almost the same as that of the above mentioned. However, the pore is filled by water, the soil particles move closer to each other and water in the pore is dissipated. The compressibility of saturated soil associated with pore water dissipation is called consolidation. The consolidation process can be divided into two stages such as Primary consolidation which is the process that water in pore dissipates, the pore becomes smaller, thus the soil is denser and the other is Secondary consolidation that is the process by which pore water has fully dissipated, however, soil particles continue moving and sliding over each other to a stable position (Pham, 2017).

To estimate the consolidation settlement of soft soils, like clay; it is important to evaluate the field conditions including the underlying soil conditions and obtain essential parameters such as compression index, pre-consolidation pressure, and present overburden stresses before the addition of foundation load. This can be achieved by using either in-situ or laboratory tests. For a given initial and final stress condition; the total consolidation settlement of a compressible layer is highly dependent on the value of the pre-consolidation stress (Fox, 2003).

Pre-consolidation stress is a demarcation between the region of small strain (stiff deformation) and large strain (soft deformation) in the void ratio-log effective vertical stress curve of the oedometer test. If the final effective vertical stress is less than pre-consolidation stress, the loaded soil will experience a small settlement in the region of small strain (recompression). On the other hand, if the final vertical effective stress is larger or equal to pre-consolidation stress, the soil will experience a large settlement in the region of large strain. Another important parameter to consider is the void ratio. Increased change in void ratios will result in increased indices and hence, large settlements in both regions (Makusa P. & Gregory, 2013).

Consequently, before estimating the settlement, it is necessary to find out the possible region of compression of soil in question. The stress history of the soil (over-consolidated or normally consolidated) should be established. This is obtained by computing the consolidation ratio (CR) and then choosing the appropriate formula for estimating the consolidation settlement. The *CR* of soil is given as a ratio of effective pre-consolidation pressure, and the effective present overburden vertical stress,

$$CR = \frac{\sigma_p'}{\sigma_{v0}'} \qquad (2.1)$$

Where;

σ_p' = pre-consolidation pressure, kPa

σ_{v0}' = initial effective vertical stress, kPa. It is required that the *CR* is computed based on past pressure but due to some reasons such as erosion or excavation this past pressure cannot be quantified and, therefore, the pre-consolidation pressure is used to compute the *CR* regardless of the cause of pre-compression (Terzaghi et al., 1996) If the *CR* is between 0.8 and 1.2 the soil is categorized as normally consolidated (NC) soils. The soil at this transition point is likely to experience a large strain than a small strain. Therefore, the analytical computation for primary consolidation settlement is done on the region of large strain.

2.3.1. Consolidation Settlement of Normally Consolidated Soil

The pre-consolidation pressure of normally consolidated soil is approximately equal to the final effective vertical stress at a point of interest in a compressible layer. Therefore, the consolidation settlement is computed on the region of large strain using the change in void ratio under compression virgin gradient. According to (Atkinson, 2007), this magnitude is given as a product of void ratio and thickness of the layer, which is inversely proportional to a specific volume, ν given by:

$$\delta_c = \frac{\Delta e_i}{v} H_{0i} \qquad (2.2)$$

Where:

δ_c = the settlement at the end of primary consolidation, m

Hoi = thickness of sub layer, m

Δei = change in void ratio and

ν = specific volume

The specific volume for saturated soils is given by

$$v = \frac{v_{total}}{v_s}$$

Where:

Vtotal = Total volume, which is the sum of the volume of water (*Vw*) plus a volume of solid (*Vs*)

2.3.2 Consolidation Settlement of Over-Consolidated Soil

The consolidation settlement analysis of over-consolidated (OC) soil lies between two categories. The first category is when the OC soil has undergone some loss of overburden stress such that the final vertical effective stress is less than the pre-consolidation pressure. This means that the soil sometime in the past has been subjected to stresses which were equal to pre-consolidation stress, and later for some reasons (e.g. erosion, excavation, or land slide) the overburden stress was reduced. However, the pre-consolidation pressure is still unaltered; thus, small deformation is expected. Consequently, the consolidation settlement can be computed only on the side of a small strain with recompression slope *Cr* in the form of

$$\delta_c = \sum \left[\frac{c_{ri}}{1+e_{0i}} H_{0i} \log\left(\frac{\sigma'_{vfi}}{\sigma'_{v0i}}\right)\right] \tag{2.3}$$

Where, H_{0i} = thickness of sub-layer (m)

σ'_{vfi}= final effective vertical stress, kPa

σ'_{v0i}= initial effective vertical stresses before foundation loads, kPa.

In case of final effective vertical overburden stress greater than pre-consolidation pressure, the total settlement can be computed from the region of small strain to the region of a large strain of change in void ratio using:

$$\delta_c = \sum \left[\frac{c_{ri}}{1+e_{0i}} H_{0i} \log\left(\frac{\sigma_p'}{\sigma_{v0i}'}\right) + \frac{c_{ci}}{1+e_{0i}} H_{0i} \log\left(\frac{\sigma_{v0i}'}{\sigma_p'}\right) \right] \quad (2.4)$$

2.3.3. Pre-Consolidation pressure (Pc)

The maximum effective vertical overburden stress that a particular soil sample has sustained in the past is said to be Pre-consolidation pressure. It's an important in geotechnical engineering, particularly for finding the expected settlement of foundations and embankments. In the other word the pre-consolidation pressure of soil is defined as the highest stress of the soil ever felt in its history. It is the pressure at which major structural changes including the breakdown of inter-particle bonds and interparticle displacement begins to occur. The practical significance of the pre-consolidation load appears in calculating the settlements of structures. soil may have been pre-consolidated during the past geologic events by the weight of ice which has melted away, structural loads which no longer exist, or/and due to other geologic overburden. For example, thick layers of overburdened soil may be eroded or excavated away or heavy structures may have been torn down. The significance of the pre-consolidation pressure in practical, to appear during the settlement calculation of structures (Jumikis A.R., 1984).

There are a few graphical methods employed to determine Pc based on consolidation laboratory test data. Among these methods, the earliest and most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log *p* curve of an undisturbed sample as shown in Fig 2.1 from *B*, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the pre-consolidation pressure Pc (Murthy,V.N.S.,1990).

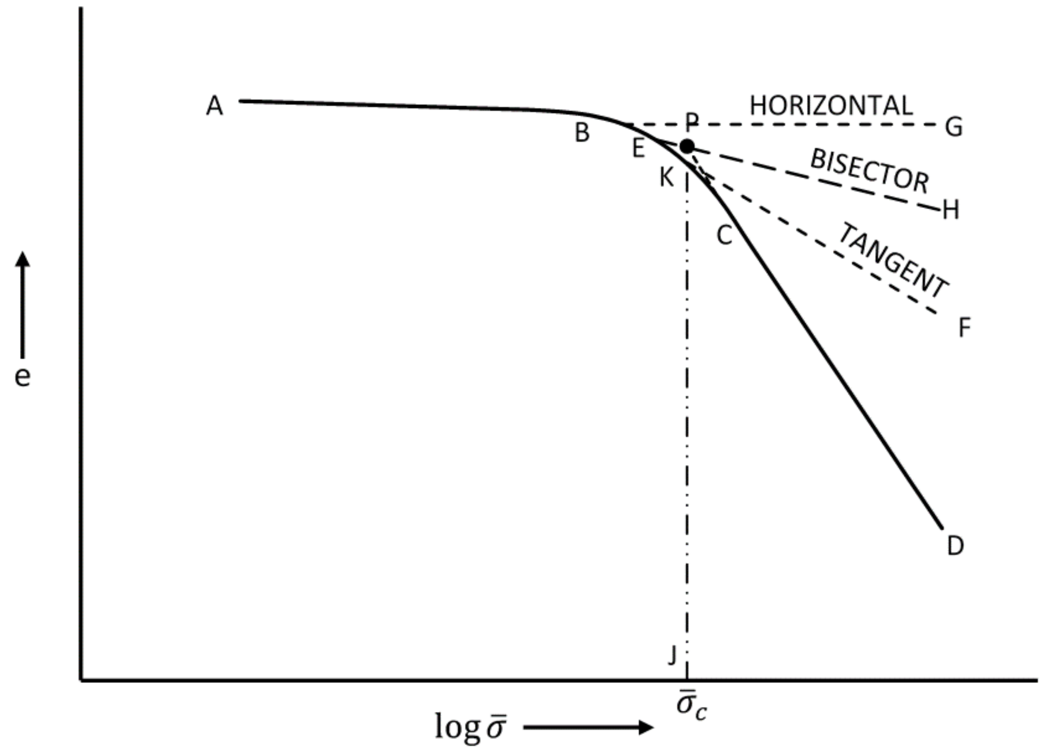


Figure 2.1: Method of determining Pc by Casagrande method (Murthy, 1990).

2.4. Construction Problems on Soft Compressible Soils

In the case of highly compressible saturated soft clay, imposition of load generates excess pore water pressure in a soft layer. This excess pore water pressure is initially borne by water molecules existing within the soft layer. Initial excess pore pressure in the water is equal to the applied external pressure on the soil. Slowly and slowly, as water oozes out of pore space depending upon the permeability of the soil, excess pore water pressure dissipates and the load gets transferred to soil grains. With time, excess pore water pressure decreases, and correspondingly, the effective stresses in the layer increase. After a very long time, the excess hydrostatic pressure becomes zero and the entire consolidation pressure becomes effective stress transmitted from grain to grain of the soil. As a result, soft soil gets compressed and the consolidation process continues over a long period of time unless measures are taken for quick dissipation of excess pore water pressure. Throughout this process, the foundation of the structure would settle until the complete consolidation takes place. By this time, irreparable damage to the structure may occur (Hawsan, 2004) and (Gebby, 2005).

Because of low permeability and poor drainage characteristics of soft soil, it becomes essential to drain out pore water before construction begins, otherwise, the added weight of the new structure will cause water to squeeze out for a long time. Moreover, due to the high compressibility of these soils, the consolidation settlements are of a very high order and from a structural safety point of view, it would be better if a major portion of this consolidation settlement takes place before or during the construction phase itself (Hinchberger, 2009).

2.5. Plaxis 2D Software

Plaxis 2D is an advanced finite element method software intended for analyzing two-dimensional problems of deformation and stability in geotechnical engineering. The development of the software began in 1987 at Delft University of Technology. The Plaxis 2D software comprises three sub programs namely the input program, the calculation program, and the output program. It performs analysis with either an assumption of plane strain or axisymmetry with 6-noded or 15-noded triangular elements (Plaxis, 2000).

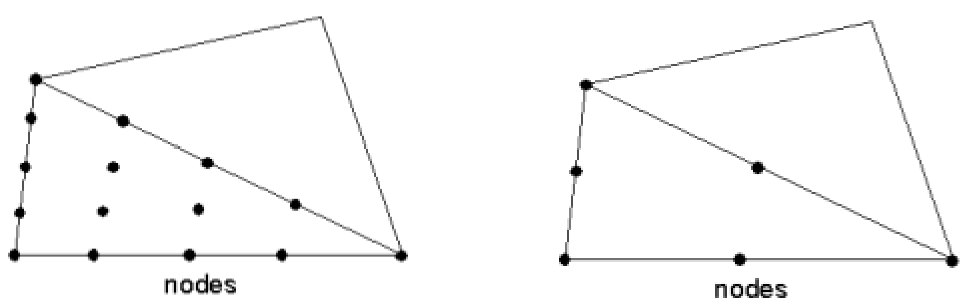


Figure 2.2: Number of nodes in triangular element (M.plaxis, 2000).

If one dimension is much larger than the others, it is possible to implement the assumption of plane strain. This implies that the principal strain in the direction of the longest dimension is constrained and assumed to be zero. To describe the deformations of soil occurring from changes in the current stress state, a mathematical framework is assigned to the soil. These govern the force-displacement relationships and are called material models (Makusa P. & Gregory, 2013). In Plaxis 2D, there are several material models available. However, within the scope of this master's thesis, only the plaxis hardening soil model was analyzed.

2.6. Constitutive Models.

Soil behavior under load is, in fact, complicated; Soils may change from idealized linear, homogenous, and isotropic behavior to non-linear, heterogeneous, and anisotropic behavior when subjected to stresses. The Selection of an appropriate soil constitutive model is essential for successful finite element analysis (FEA). Even though numerous constitutive models have been proposed to describe the various aspects of soil behavior in FEM, none of the available constitutive models can reproduce all aspects of soil behavior. (Zdravkovic, 2001).

The successful use of any numerical analysis in solving any geotechnical engineering problems depends on the constitutive model chosen to represent the actual material behavior. The constitutive model associated with a specific material has to describe the material evolution under external actions. There exist a large variety of models which have been recommended in recent years to represent the stress-strain and failure behavior of soils (Devi, 2014). Due to the complexity of real soil behavior, a single constitutive model that can describe all facets of behavior, with a reasonable number of input parameters, has not been achieved. Consequently, soil models are continuously developed and improved hence there are many soil models available today, each of which has different advantages and limitations.

2.6.1. Linear Elastic model (Hooke's law).

It is a linear elastic model which is based upon Hooke's law of linear elasticity. This model consists of two basic parameters which are modulus of elasticity (E) and poisons ratio μ. The Linear elastic soil model does not consider the shear strength of the soil mass hence over stressing may be encountered. Since the soil behavior is highly nonlinear and irreversible, this model is insufficient to capture the essential features of soil however, this model can be used to model the stiff volumes in soil like the concrete walls (Brinkgreve, 2005).

2.7.2. Mohr-Coulomb model.

Mohr-Coulomb model is an elastic-perfectly plastic model which is often used to model soil behavior in general and serves as a first-order model. In general stress state, the model's stress-strain behaves linearly in the elastic range, with two defining parameters from Hooke's law (Young's modulus, E and Poisson's ratio,ν). Two parameters define the failure criteria (the friction angle, ϕ, and cohesion, c) and also a parameter to describe the flow rule (dilatancy angle, ψ which comes from the use of non-associated flow rule which is used to model a realistic irreversible change in volume due to shearing). This model is applicable to analyze the stability of dams, slopes, embankments, and shallow foundations (Lee, 1990).

Although failure behavior is generally well captured in drained conditions, the effective stress path that is followed in undrained materials may deviate significantly from observations. It is preferable to use un-drained shear parameters in an un-drained analysis, with friction angle set equal to zero. The stiffness (hence also deformation) behavior before reaching the local shear is poorly modeled. For perfect plasticity, the model does not include strain hardening or softening effect of the soil.

2.6.3. Modified cam-clay.

The Modified Cam-clay is an elastic-plastic strain hardening model where the non-linear behavior is modeled using hardening plasticity. The model is based on Critical State theory and the basic assumption that there is a logarithmic relationship between the mean effective stress, σ' and the void ratio, e. Virgin compression and recompression lines are linear in the e-ln σ' space, which is most realistic for near-normally consolidated clays. Only linear elastic behavior is modeled before yielding and may results in unreasonable values of ν due to log-linear compression lines. This model is more suitable to describe deformation than failure, especially for normally consolidated soft soils. The above limitations were confirmed by (Gens et al., 1988).

Despite this, this model is unable to predict an important feature of behavior that is commonly observed in un-drained tests on loose sand and normally consolidated undisturbed clays, and that is a peak in the deviatoric stress before the critical state is approached. The critical state had been much less successful for modeling granular materials due to its inability to predict observed softening and dilatancy of dense sands and the un-drained response of very loose sands. The above limitations were confirmed by(Gens et al., 1988) where it is noted that the materials modeled by critical state models appeared to be mostly limited to saturated clays and silts, and stiff over-consolidated clays did not appear to be generally modeled with critical state formulations.

2.6.4. Plaxis soft soil model.

The soft soil model is based on the modified cam-clay model, but some of its drawbacks have been improved in this model. First of all, the model does not involve the over-prediction of the shear strength for over-consolidated states of stress. a Mohr-coulomb failure criterion has been added to improve the capabilities of the model at failure. The “friction constant” M, which determines the steepness of the critical state line and therewith the k_o-value can be obtained. The stiffness behavior in the soft soil model is based on the logarithmic relationship between the mean effective stress, p, and the volumetric strain, εv (rather than the void ratio). Therefore, this model slightly modified parameters λ^* and k^* insteady of the original cam-clay parameters and requires no input on the initial void ratio.

The model has no advantages over the Mohr-coulomb model in unloading problems, such as excavations or tunnel problems. And the model can be considered a second-order model, at least for near-normally consolidated clays for the ‘loading’ type of applications mentioned above. The soft soil model does not have the capabilities to model anisotropic strength and stiffness generally observed for peat. Nevertheless, it could be used for peat as long as anisotropy does not play a major role in the application (Brinkgreve, 2005).

2.6.5. Plaxis Hardening Soil Model.

The Hardening Soil model is a true second-order model for soils in general (soft soils as well as harder types of soil), for any type of application (Brinkgreve, 2005). The model involves friction hardening to model the plastic shear strain in deviatoric loading, and cap hardening to model the plastic volumetric strain in primary compression. The Distinction can be made between two main types of hardening, namely shear hardening and compression hardening. Shear hardening is used to model irreversible strains due to primary deviatoric loading. Compression hardening is used to model irreversible plastic strains due to primary compression in odometer loading and isotropic loading. Both types of hardening are contained in the present model. Failure is defined using the Mohr-Coulomb failure criterion. Because of the two types of hardening, the model is also accurate for problems involving a reduction of mean effective stress and at the same time mobilization of shear strength. Such situations occur in excavation (retaining wall problems) and tunnel construction projects. Some basic characteristics of the model are Stress dependent stiffness according to a power law (m), plastic straining due to primary deviatoric loading (E_{50}^{ref}), plastic straining due to primary compression (E_{oed}^{ref}), elastic unloading/reloading input parameters (E_{ur}^{ref}) and failure criterion according to the Mohr-Coulomb model (c and ϕ).

2.7. Material Parameters required for hardening soil model

a) Cohesion

Cohesion of soil is defined as the shear strength at zero normal pressure on the surface of failure. Soil cohesion is the result of the reaction of an extremely large and countless number of resisting elements on the failure surface, each considered by the resistance value. Based on this definition, soil cohesion is a constant parameter. Besides, Classical soil mechanics which is based on the Mohr-Coulomb failure principle indicates that cohesion is one of the fundamental properties of soil contributing to the shear strength. Soil cohesion of derived through conducting a triaxial compression test (Shahangian, 2011). Soil cohesion of soft clay and can be derived through conducting a triaxial compression test. When a pre-Consolidation /pre-Compaction surcharge acts on a soil sample, some small elastic deformations appear; soil compacts (Consolidates) under the pre-consolidation. This essential soil parameter is required as an input in the soil model to calculate the shear strength of the soil mass exposed to loading.

b) Friction Angle

Internal friction angle is an intrinsic soil parameter considered as the measure of the shear strength of soils due to friction. It is one of the important parameters considered as a typical characteristic for reconnaissance of granular soils in particular. In a numerical simulation of soil model geometry, internal friction angle is used as an input to consider the effect of soil shear strength in soil constitutive model (Shooshpasha et al., 2015).

c) Dilatancy Angle

Dilatancy is an important aspect of soil behavior. It manifests itself as a volumetric strain coupled to an applied shear strain. The angle of dilatancy, ψ, is a constant of a soil model, the elastic perfectly plastic Mohr-Coulomb (MC) model with a non-associated flow rule. It embodies the concept of dilatancy within the confines of the MC model. In real soils, dilatancy is variable and depends on soil density and stress level among other things (Maranha et al., 2000). Clays are characterized by a very low amount of dilation according to (Obrzud, 2016) for cohesive soils

$\Psi=0^0$ for normally-and lightly –over consolidated soils. (2.11)

Ψ=$\phi/6$ for over consolidated soils. (2.12)

Ψ= $\phi/3$ for heavily over consolidated soils. (2.13)

d) Stiffness parameters

The modulus in the hardening soil model is described more accurately by adding three moduli; Eur, which refers to reloading/unloading modulus and $E_{50}$and E_{oed}. E_{50}. Refers to secant modulus from tri-axial test and E_{oed} refers to tangent modulus that corresponds to a modulus for stresses higher than the pre-consolidation pressure evaluated from the CRS test (Bringreve et al, 2010).

These moduli are stress-dependent and in the following formulas, the change of the three moduli can be observed by the change of soil stress, see the equations (2.14), (2.15), & (2.16) below (Bringreve et al, 2010).

$$E_{50} = E_{50}^{ref} \left(\frac{c\ cos\phi - \sigma_3'\ sin\phi}{c\ cos\phi + p^{ref}\ sin\phi}\right)^m \quad (2.14)$$

$$E_{oed} = E_{oed}^{ref} \left(\frac{c\ cos\phi - \sigma_1'\ sin\phi}{c\ cos\phi + p^{ref}\ sin\phi}\right)^m \quad (2.15)$$

$$E_{ur} = E_{ur}^{ref} \left(\frac{c\ cos\phi - \sigma_3'\ sin\phi}{c\ cos\phi + p^{ref}\ sin\phi}\right)^m \quad (2.16)$$

In the above-mentioned formulas, the $E_{ref}$is the reference Young`s modulus and $P_{ref}$is reference stress for the stiffness and is set to 100 kPa, as a default value in Plaxis. Usually E_{ur}^{ref} is set to three times $E_{50}^{ref}$and $E_{oed}^{ref}$is set to be equal to E_{50}^{ref}as a default value of the Plaxis software. However, Plaxis cannot handle big ratios between E_{50}^{ref} and E_{oed}^{ref} and therefore Plaxis software will sometimes recommend a value of one of the moduli to use. The exponent, m, is a factor that regulates the stress dependence of the modulus. For soft soils, the factor is set to 1 and for sands, it is set to value from 0.5 to 1. In the formulas, it can also be noticed that E_{oed} is dependent on the major principal stress σ1, E_{ur} and E_{50} are dependent on the minor principal stress σ3 (Bringreve et al, 2010).

2.8. Plane strain and axial symmetric conditions.

There are two conditions of stresses and strains that are common in geotechnical engineering. Which is the plane strain condition and axial symmetry or axisymmetric conditions.

a) Plane Strain Condition.

The plane strain model means the strains can only take place in the x y plane. Along the longitudinal axis (out of plane direction) the strain is assumed to be zero, $\varepsilon z = 0$. As an example of a plane strain condition, an element of soil behind a retaining wall can be considered. Because the displacement that is likely to occur in the Y-direction is small compared with the length in other directions (Budhu, 2010).

b) Axial symmetry Condition

Axisymmetric Condition is a condition that occurs in practical problems where two stresses are equal. Example of axisymmetric condition is like a water tank, an oil tank founded on a soil mass. The radial stresses and circumferential stresses on a cylindrical element of soil directly under the center of the circular structures are equal because of axial symmetry (Budhu, 2010).

2.9. Past studies on deformation analysis of soft clay.

Ismail et al.,(2011) analyzed deformations in soft clay due to unloading which is part of the undergoing new railway project in Sweden. It was carried out on a soil layer that consisted of silty sand for the first few meters and soft clay for deep layers below. Due to the construction conditions of the area, the excavation was taking place under the newly constructed bridge and the existing highway had to be diverted and placed under this bridge. The deformation analysis mainly deals with the unloading modulus based on field deformation measurements which are inclinometer device, field, and laboratory tests, and previously researched the unloading modulus of soft clay.

The main deformation analysis was performed with the help of the finite element-based geotechnical computer software called Plaxis. For the simulation, two soil models, namely; Mohr-Coulomb and Hardening soil, were used to catch the real deformation behavior of soft clay. The results of the analysis from these soil models are cross-checked with the field deformation measurements and the unloading modulus is estimated through calculation.

Saadeldin et al., (2011) was studied the Performance of Road Embankment on Cement Stabilized Soft Clay (CSC). The undrained shear strength of the soft clay was experimentally determined before and after stabilization with cement. The results of the experimental work were used to simulate the behavior of the foundation soil under the road embankment using a 2-D finite element model. The Hardening Soil (HS) model was selected to simulate the behavior of the soft clay layer and Mohr-Coulomb (MC) model was selected to simulate the behavior of CSC and compacted sand extending to relatively shallow depths ranging from 1 to 5m under the embankment. CSC having a variable thickness ranging from 1 to 5m, followed by a soft clay layer extending to 15m below the ground surface. The performance of the embankment founded on CSC was compared to that obtained if the CSC was replaced with compacted sand fill. The result of the analysis indicates Cement stabilization enhanced the performance of the embankment concerning safety against shear failure more than sand soil replacement.

Al-Taie et al., (2016) Evaluated Foundation Settlement under various Added Loads in Different Locations of Iraq Using Finite Element. The study was conducted settlement analysis in numerical (mathematical) and plaxis 2d software under different added loads. The analytical calculations were conducted based on the oedometer tests for cohesion soil layers. While, for cohesionless soil layers the immediate settlement calculations were conducted based on the SPT field tests and The finite element soil geometry model adopted for the analysis of raft foundation of (25 × 60) m, and the added loads (dead and live loads) 56; 63.5; 68; 75 and 93 kPa were used. The soil models used in the software were Harding soil (HS) for the cohesion soil and Mohr-Coulomb (MC) model for cohesionless soil. Finally, the result compared of plaxis software with mathematical calculations.

Por et al., (2017) studied the Deformation characteristics and stress responses of cement-treated expansive clay under confined one-dimensional swelling. The effects were evaluated by focusing on the unconfined compressive strength, swelling-shrinkage strains under various conditions, and the lateral coefficient of earth pressure during one-dimensional deformation for artificial mixtures of two different clays at three different ratios. The experimental results show that the cement addition led to marked decreases in the areal shrinkage strain and vertical free swelling strain in addition to the obvious improvement of strength and stiffness of soils.

Zambri et al., (2017) studied the deformation characteristics of lime stabilized peat soil using finite element method the researcher has used the different percentages of lime as a peat stabilizer to increase shear strength and reduce settlement of soil he conducted that different laboratory test and the researcher was used the Mohr-Coulomb soil model in PLAXIS 2D software to analyze deformation. The result showed that with the increase of percentages of lime, the deformation of treated peat soil can be reduced.

Gupta & Mital (2019) investigated the effects of the soil bearing pressure values of the rectangular shape footing located on the horizontal ground surface using PLAXIS software. The two dimensional (2D) and three dimensions (3D) model of rectangular footing were analyzed and compared. The rectangular footing was located at two positions which is at the left corner and center of the model soil and Mohr-Coulomb soil model was used. Finally the Allowable bearing pressure of the rectangular footing located at the left corner and center of the soil model was the same result which indicates there is no effect on the position of rectangular footing.

Sorsa et al., (2020)studied the Engineering Characterization of Subgrade Soils of Jimma Town, Ethiopia, for Roadway Design. The researcher was executed to determine engineering properties of subgrade soils by different laboratory tests. The soil characterization indicated that soft clay is the predominant subgrade soil type and that it has a very low load-bearing capacity, high plasticity, low strength and, high compressibility, which makes the soil unsuitable to serve as a highway subgrade without the help of soil improvement techniques. Moreover, the study recommended that future study will consider stabilization for the enhancement of soil properties.

2.10. Research gap from the literature review.

From the literature review, it was identified that engineering properties of soft clay soils and enhancement of engineering properties of soft clay soils as well as deformation analysis were studied in different works of literature. Moreover, different authors used different stabilizing agents for the investigation of deformation analysis with different percentages of stabilizers. This indicates that soft soils are highly influenced by the construction of any foundation. Additionally, most studies used the Mohr-Coulomb soil model for simulation of soil behavior. Zambri et al., (2017), Ismail et al., (2011) but Mohr coulomb soil model is its limitation which is used only five parameters (E, ν, ϕ c, andψ) for simulation purposes. and Por et al., (2017) conducted only an experimental study on deformation analysis. In addition, Sorsa et al., (2020) enhancement of soft soil properties by stabilization was suggested. And finally, simulations of deformation characteristics of soft clay soils are not studied in a specific area.

Therefore, the present study focuses on the deformation analysis of cement modified soft clay using a hardening soil model. A case of shallow foundation (square footing) overlying on both natural and stabilized soft clay with different added loads.

CHAPTER THREE

MATERIALS AND RESEARCH METHODOLOGY

3.1. Study area

The town of Jimma is located in the southwestern part of Ethiopia in Oromia regional state. It is found at a distance of approximately 352km from Addis Ababa, the capital city of Ethiopia. Its geographical coordinates are about 7° 13'- 8° 56N latitudes and 35°49'-38°38'E longitudes. The town is located at an elevation of 1718 to 2000 meters above mean sea level. According to the recent information on the population of Jimma town is 207,573 in 2012. It lies in the climatic zone locally known as Woyna-daga which is considered ideal for agriculture as well as human settlement (Wikipedia, 2020).

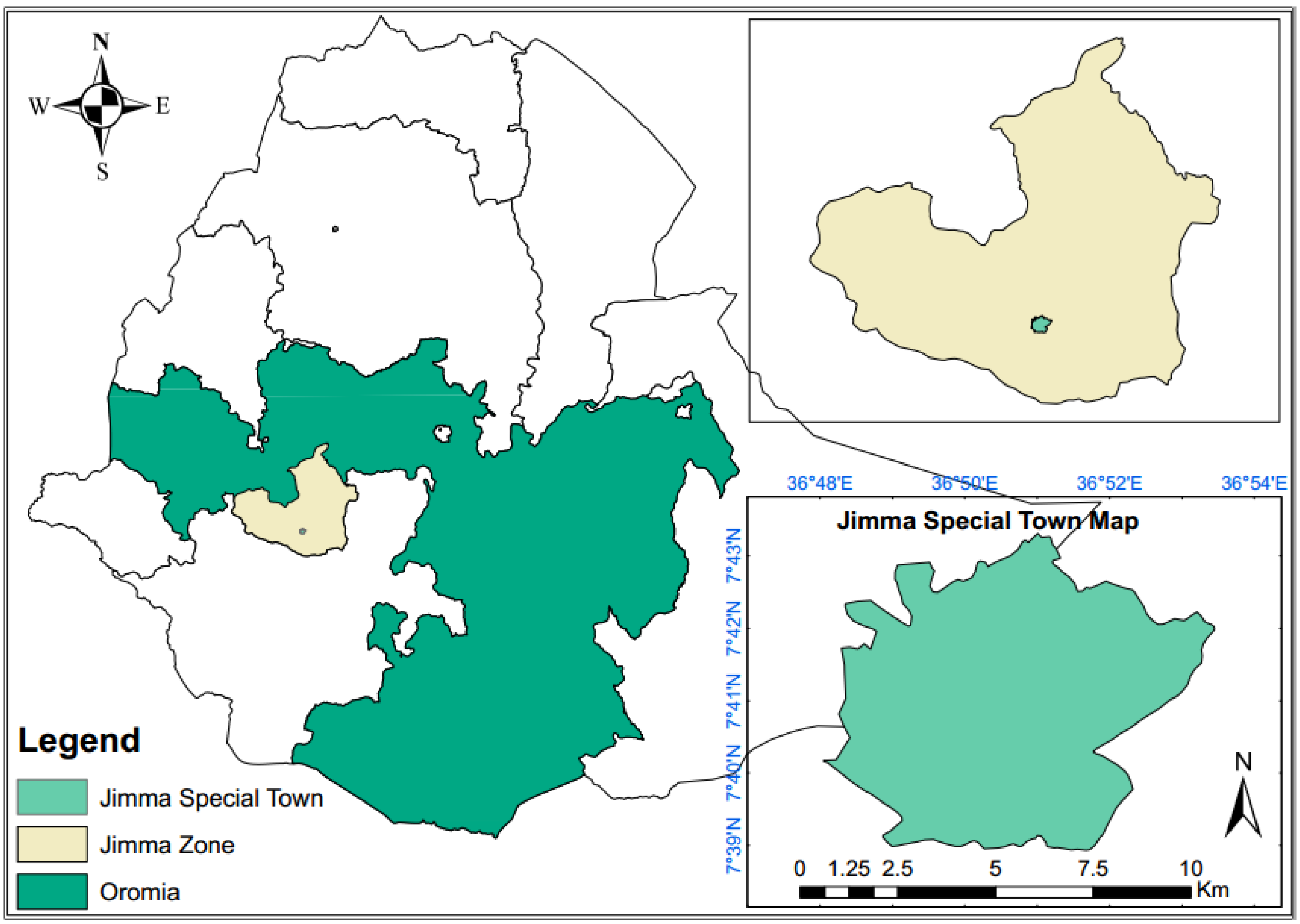


Figure 3.1: Map of the study area.

3.2. Geology of the Area

The town of Jimma zone is in the climatic zone, locally known as Woyna-Dega, which is very suitable for agriculture as well as human settlement(Wikipedia, 2020). In geological terms, the city is underlain by volcanic rocks of the Tertiary age, which seems to be mostly basalt. The rock unit of the area consists of medium to acid lava and, thus, of the so-called Trap formation (Yonas, 2002). Quaternary alluvial deposits, Sentema Basalts, and middle trachyte flow mainly cover Jimma, according to the Geological Survey of Ethiopia (GSE) report. The Quaternary alluvial deposit is often bogy with soft clayey soil. At places, they form loose thick soil of clayey-silty material that is not consolidated. The color is mostly light grey covered with grass or low vegetation such as reeds and bogs. The middle trachyte covers many areas of the Town especially around the northeast of Jimma and southeast of Jimma. In most of its exposure, the Middle Trachyte flows is light grey, fine to medium-grained trachyte.

3.3. Cement stabilization.

Cement is the most popular and common hydraulic binder used to stabilize organic soil. It can be applied in essentially all kinds of soil, significantly improving their geotechnical properties. Research shows that cement itself, and if mixed with granulated blast furnace slag, yields the best stabilization results in weak-bearing soil with high organic content. (Topolinski,2019).

Portland cement can be used either to modify and improve the quality of the soil or to transform the soil into a cemented mass with increased strength and durability. Portland cement is comprised of calcium-silicates and calcium-aluminates that hydrate to form cementitious products(AASHTO, 2008). Dangote Portland cement was bought from Jimma market.

Table 3.1: Chemical composition of ordinary Portland cement (Awol, 2011).

Constituent	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_2
Percentage	20.03	5.94	3.73	66.31	1.07	1.14

Cement is used to bind soil particles to increase soft soil’s strength and stiffness. Stabilization is done by mixing an appropriate amount of dry or wet cement throughout a volume of the soft soil. The increase of the strength of cement-stabilized soils comes from the physicochemical reactions between the soil and cement, such as the hydration of cement and the interaction between the substances in the soil and the products of the hydration of cement(Chen & Wang, 2006) evaluated experimentally ranges of percentages of cement by weight to be tested initially for different soil types table 3.2.

Table 3.2: Percentages of cement to be tested initially for different types of soils *(Saadeldin, 2011)*

Types of soil	Percentage of cement by weight
GW, GP, GM, SW	3-8
SC, GC	5-9
SP, SM	7-11
ML	7-12
CL, OL, MH	8-13
CH	9-15
OH	10-16

3.4. Research methodology

The methodology of this study was dividing into two phases as shown in figure 3.2. The first phase is the procedure for the determination of laboratory test results and the second phase is the simulation sequence and steps of the model simulation. For laboratory tests, the sample was stabilized 9%, 12%, and 15% of cement for both one-dimensional consolidation test and triaxial test to determine the parameters of soil which is used for simulation of plaxis 2D software.

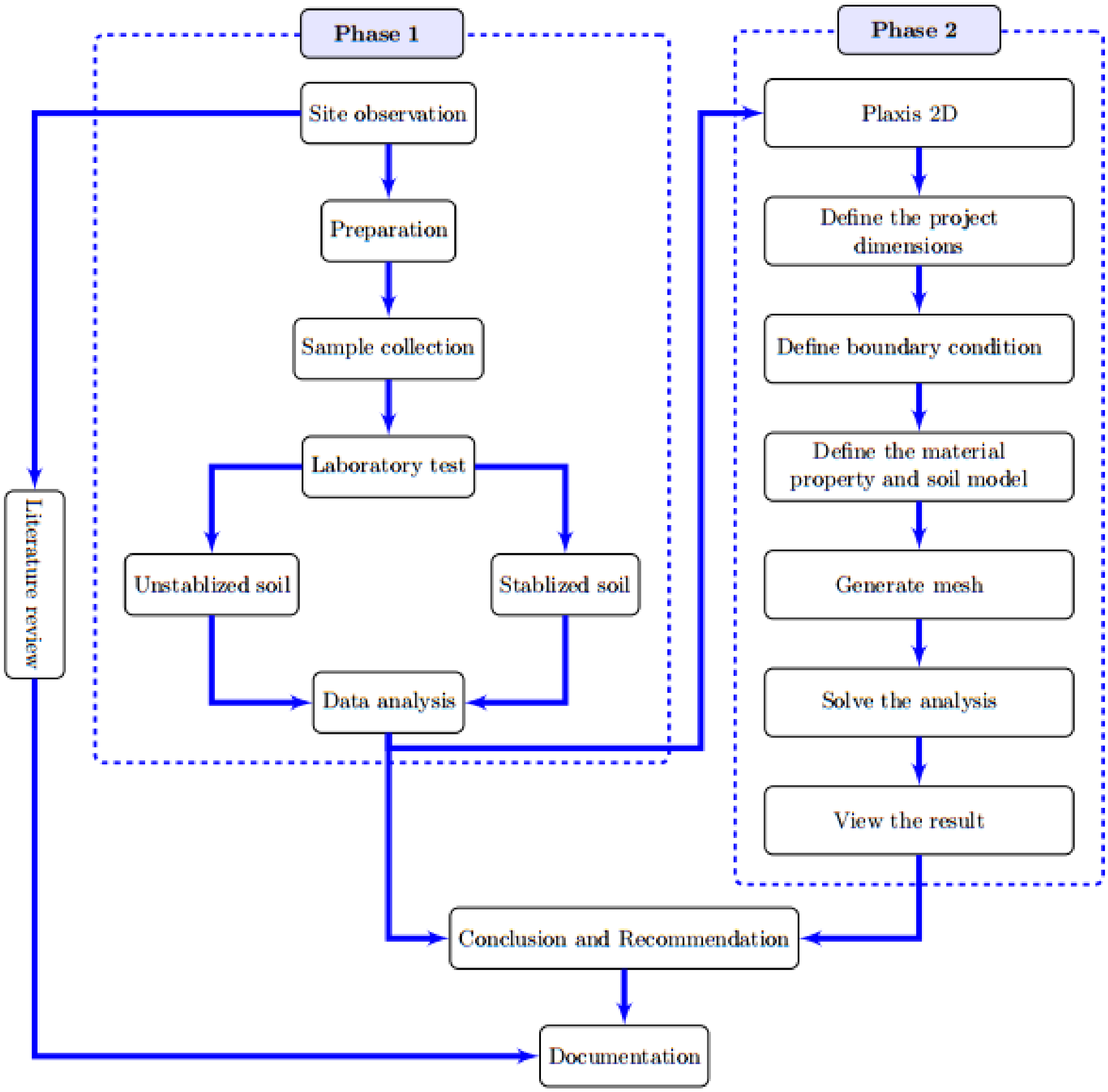


Figure 3.2: Research methodology.

3.5. Laboratory test

A. Moisture content

The natural water content of a soil mass is the fraction of the mass of water to that of the solids expressed as a percentage. The knowledge of the moisture content of the soil is a basic requirement in all studies that are related to geotechnical engineering. The natural moisture content of soil can also, be used to give some clues about the soil bearing capacity and settlement potential. This test is one of the simplest and cheapest tests to conduct in a laboratory. The moisture content of the samples in this research were determined while using the ASTM standards**.**

B. Grain Size Analysis

Grain size analyses are performed to determine the different percentages of particles found within a given soil sample. Both wet sieve and hydrometer tests were conducted. A Wet sieve is used to determine whether the soil consists of predominantly grave, sand, silt, and or clay sizes. The hydrometer method covers the quantitative determination of the particle size distribution of soil from coarse sand to clay size using sedimentation.

C. Specific Gravity

Specific gravity is the ratio of the mass of a 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 importance of determining the specific gravity in this study is to determine particle sizes in hydrometer analysis. The specific gravity of the samples was determined using *ASTM D 854-83.*

D. Atterberg Limits

The test was used to determine the plastic limit (PI), liquid limit (LL), and plasticity index (PI) of the soil. The experiment will be performed using ASTM D4318 -98 (Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils). Approximately 200g of a soil sample that will be passing through a 475 µm sieve for the Atterberg limit test will be needed. For liquid limit, determination Water will be added to the soil samples and mix well then to be placed in the cup of the Casagrande apparatus, leveled off parallel to the base, and to be divided by drawing the grooving tool along the diameter through the Centre of the hinge and the sample cup is raised and dropped a specified number of times for the groove to barely close. The soil moisture content at which this happens is the liquid limit. For plastic limit determination, the sample was taken, and rolling the soil passed until the soil crumbled when the thread is reached 3mm diameter and the moisture content will be determined. The plasticity index (PI) is the difference between LL and PL.

E. Free Swell

Free swell test gives the approximate value of expansiveness. The researcher was conducted free swell test based on ASTM standards. The percent of the free well was calculated as:

$$\%FS = \frac{Vf - V_o}{V_o} * 100 \tag{3.1}$$

Where Vf and Vo are symbols to notify final and initial volume respectively.

F. Unconfined Compressive Strength

The unconfined compression test can be performed on cohesive materials, such as clay. The resulting unconfined compressive strength is a reasonable indicator of the mechanical properties of soil. This test was conducted in the laboratory while using static loading that significantly influences the structural responses and performance of soil material under axial loading. It was conducted while using the unconfined compression strength apparatus,

Table 3.3: Consistency and unconfined compression strength of clays.

Consistency	q_u (kN/m^2)
Very soft	0-24
Soft	24-48
Medium	48-96
Stiff	96-192
Very stiff	192-383
Hard	>383

G. One-Dimensional Consolidation Test

One- dimensional consolidation test is used to obtain a compression parameter for the amount of settlement and the consolidation parameter *cv* for the settlement rate estimate. The test was performed on both undisturbed and remolded soil sample that is placed in a consolidation ring. Compression parameters such as coefficients of compressibility (av), compression index (Cc), swelling index (Cs), and coefficient of volume compressibility (mv) were obtained from void ratio versus applied pressure curve of consolidation test.

H. Triaxial test

When the deformation analysis of soil is performed by an advanced soil model in Plaxis there is a need for a triaxial test. This is because advanced models require more input of material data, for instance, the hardening soil model requires the input of three modules, such as Eoed obtained from CRS test, E50 and Eur, which are obtained by performing a triaxial test.

The triaxial test is one of the best laboratory tests for determining the shear strength of the soil. The unconsolidated undrained compressive strength of soil was determined in an undisturbed and remold condition while using a strain-controlled application of axial compression load with confining pressure. It was conducted using a triaxial compression test apparatus, according to the ASTM D2850-99 procedures.

3.6. Preparation of remolded samples

For the 1D consolidation and triaxial test $944\ cm^3$ the volume of mold specimens was used. After the soil and cement were mixed thoroughly until a uniform color was observed, then the water was added continuing to the soil-cement mixture. The amount of cement for each mixture was calculated based on the weight of dry soil. All specimens were prepared at their maximum dry unit weight and optimum moisture content, corresponding to the values obtained in standard Proctor compaction tests (ASTM D-698 2000). The specimens were statically compacted in three identical layers so that each layer reached the determined dry unit weight. The specimens were wrapped in plastic bags for 7days curing time before testing.

3.7. Soil Model and required parameters

In this study, Plaxis Hardening Soil Constitutive Model was used for simulation of untreated and treated soft clay because of according to Brinkgreve (2010), the hardening soil model is used to simulate both soft soil as well as the harder type of soil. Plaxis Hardening Soil Constitutive Model, ten basic input parameters are used. Which is ($E_{50}^{ref}, E_{oed}^{ref}, E_{ur}^{ref}, P^{ref}, c,\ \Psi, \phi, R_f, m, and\ u_{ur}$). These parameters were obtained from geotechnical laboratory tests from triaxial and odometer tests.

Table 3.4: Determination of soil parameters from triaxial test.

Parameters	Natural soil	9%	12%	15%
C	29.30	63.53	68.45	75.42
ϕ	6.52	35.1	38.5	39.52
ψ	0	3	5.85	6.41
E_{50}^{ref}	3883	42,187.5	66,382.3	90,266.6
E_{oed}^{ref}	3883	42,187.5	66,382.3	90,266.6
E_{ur}^{ref}	11500	126,562.5	199,147	270,800
γ_{unsat}(kN/m3)	11	13	14	15.2
γ_{sat}(kN/m3)	16	17	19	19.5

Table 3.4. Shows the summary results of the triaxial test for hardening soil parameters which would be used in the analysis of finite elements by plaxis.

3.8. Stress / Strain Conditions

As mentioned in the literature there are two conditions of stresses and strains that are commonly used for numerical modeling in geotechnical engineering problems. These are Axisymmetric Condition and Plane Stress/Strain condition. In this study, the plain strain condition was used to analyze the deformation analysis of square footing overlying on soft soil and cement-treated soil.

3.9. Calculation types

Plastic calculation, Consolidation analysis, and Phi-c reduction are the three forms of calculations available in Plaxis. If the user wants to do an elastoplastic deformation study without taking into account the magnitude of excess pore pressures over time, they can choose the Plastic calculation. Time effects are not taken into account in a plastic calculation. With soft soils, a plastic calculation may be used, but the loading history and consolidation cannot be tracked; instead, an accurate estimate of the final situation is given. When it is important to track the evolution of excess pore pressure in soft soils over time, the consolidation analysis should be used. The phi-c reduction in Plaxis is a safety analysis that is useful when the situation in the problem necessitates the estimation of the safety factor. A safety analysis may be performed after each individual calculation step, but it is preferable to do so after all calculation phases have been established. It is particularly unwise to begin the calculation with a safety analogue (Brinkgreve, 2005). In this research, the pressure vs. vertical settlement was calculated using the plastic calculation phase.

3.10. Structural parameters.

In Engineering, it acknowledged that the size and shape of a footing might have influences on the bearing pressure and settling characteristics of a footing. However, in in the construction of building opposed to rectangular footings the circular footings are used, hence the circular footings are more cost effective (Luévanos-rojas, 2016). Where as in direct vertical bearing loads, a square footing typically used because it uniformly distributes the load on the ground and provides the most stability to the column.

The square footing had higher bearing pressure than the circular footing, which may be contributed to the square footing's more confining impact, as the area of a square is greater than the area of a circle for the same lateral size (Patel & Bhoi, 2019). For this study, a square shape footing of size 2m (length) x2m (width) located on the soil model selected, and the structural parameters are unit weight $24kN/m^3$, young's modulus $24 * 10^6\ kN/m^2,$ and poission's ratio 0.2 are used hence the direct vertical loads is considered.

3.11. Model geometry

The generation of the finite element model with plaxis hardening begins with the geometry model. The creation of the geometry model is begun by drawing the geometry. In addition, the materials, load, and boundary conditions are specified and different added loads (100,200,300,400, and 500) kPa was applied on the top of the foundation (square footing) with a dimension of (2mx2m). After the geometry model has been created, data set of materials parameters for untreated and treated soils should be composed. The depth of the soil is 8m with a homogenous layer which is a soft clay soil and for stabilized soil there are two layers. The upper layer up to three meter is cement stabilized clay (CSC) and the second layer is soft soil.

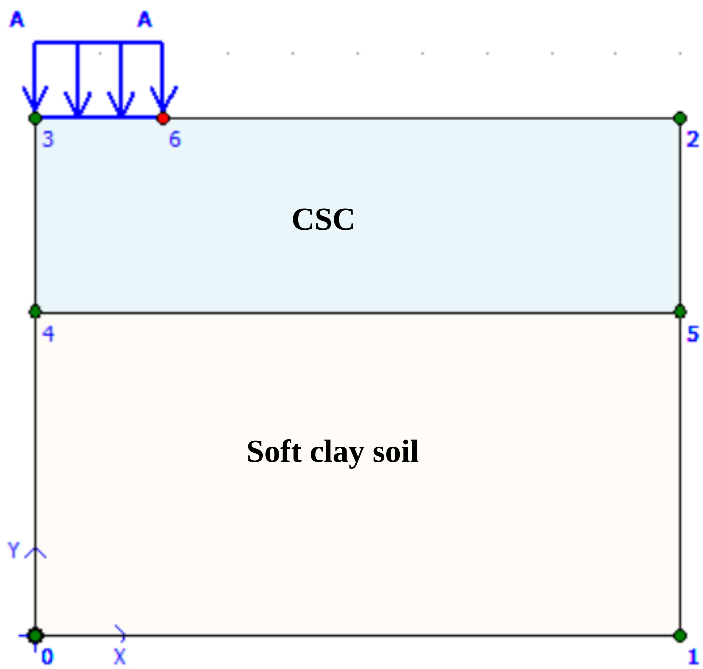


Figure 3.3: Geometry model.

3.11.1. Boundary Conditions

In finite element analysis, boundary conditions are necessary for the solution of a boundary value problem. Lateral and bottom boundaries were set at 10m horizontally and 4B vertically were used B is width of the footing. Therefore, on the bottom side, both the vertical and horizontal components of the displacement are fixed (Ux=Uy=0), besides, the two vertical boundary edges of the geometry were set free to the vertical displacement and zero horizontal displacements (zero x displacement).

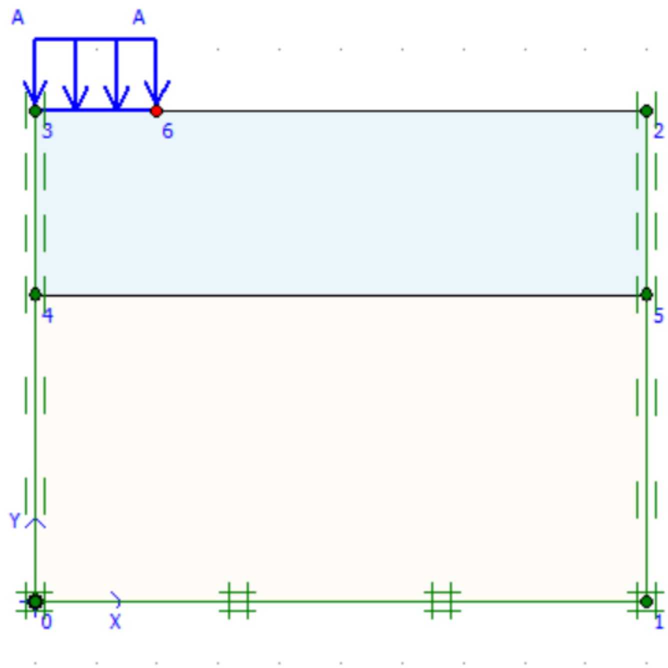


Figure 3.4: Boundary conditions.

3.11.2. Discretization (Meshing)

Discretization or meshing is the fundamental aspect of finite element modeling next to defining material properties and boundary conditions. Mesh size is the major factor contributing to the precision and accuracy of analyses in the finite element method. In this study, the finer and triangular mesh was used.

CHAPTER FOUR

RESULT AND DISCUSSION

4.1. Natural moisture content

The natural (in-situ) moisture content of the soil samples was determined from the undisturbed samples using *ASTM D 4643-00*, and Table 4.1 presents the results.

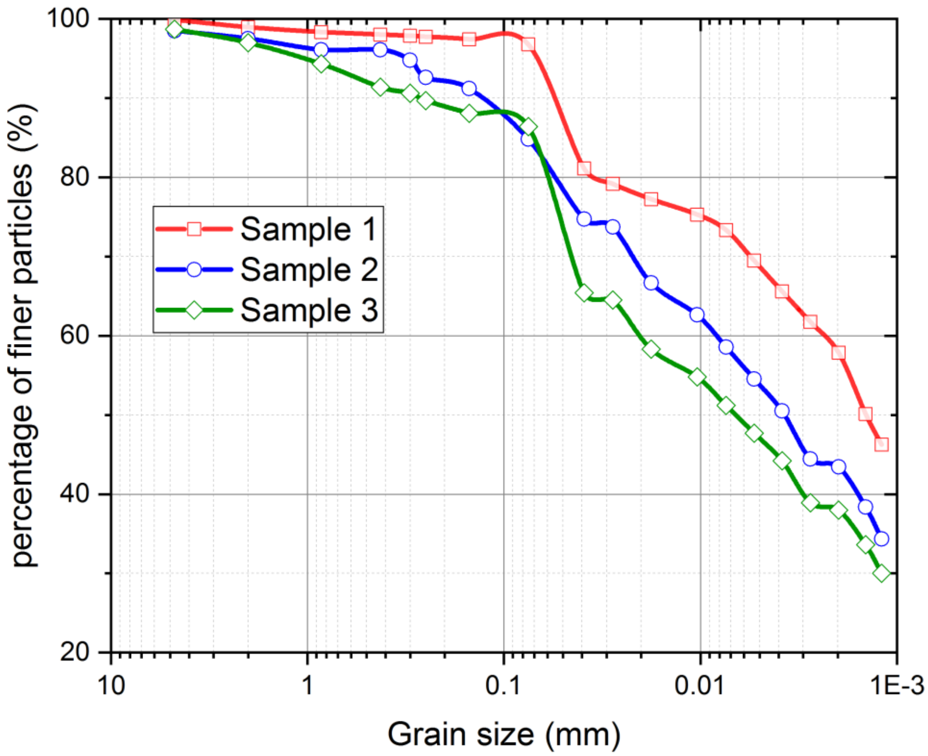
Table 4.1: Natural moisture content result.

Sample number	1		2		3	
Trials	1	2	1	2	1	2
Moisture content	63.58	59.56	41.21	40.38	50.5	49.86
Average moisture content	61.57		40.80		50.18	

Natural moisture content is a key parameter to understand the characteristics and the performance of soil and its degree of compaction at the site. As stated by (Terzaghi et al., 1996), most of the typical values of the natural moisture content of clay soil are within the ranges of 22–70%. The study results are also in agreement with the specified standard.

4.2. Grain size analysis

In this study wet sample preparation by *ASTM D422-98* was applied. Mechanical analysis was used for the coarse-sized soils by using a set of sieves and whereas hydrometer analysis was used for fine-grained soils. Here sodium hexa meta phosphate is used as a dispersing agent. For soils comprising coarser and finer sizes, both mechanical and hydrometer testing methods are performed. Oven-dried samples were used Results from the two methods were combined while using accepted standards and the particle size distribution curves that are shown in Figure were obtained. Detail tabular analysis of grain size analysis is included in the appendix.



Gravel	Sand	Silt	Clay

75mm **4.25mm** **0.075 mm** **0.005 mm** **0.001mm**

Figure 4.1: Combined grain size distribution curve.

The results of grain size analysis show that more than 85% of the total mass pass through a sieve size of 75µm. This indicates that the samples are fine-grained soil. From hydrometer analysis, the size of more than 45% is less than 5µm (clay content as per ASTM boundaries criteria).

4.3. Atterberg Limits

The Atterberg Limits, or the soil index property test, involves the determination of the plastic limit (PL), liquid limit (LL), and plasticity index (PI) of the soil. The liquid limit and the plastic limit are moisture contents that correspond to the liquid and plastic state thresholds of the soil, respectively. Value of liquid limit is used to classify fine-grained soil. It gives us information regarding the state of consistency of soil on site.

Table 4.2: Atterberg limit result.

Sample no	LL%	PL%	PI%
1	80	39	41
2	67	31	35
3	71	36	35

4.3.1. Soil classification

The most widely used soil classification systems are AASHTO and USCS systems. The USCS system is based on grain size, gradation, plasticity, and compressibility. It divides soils into three major groups: coarse-grained soils, fine-grained soil, and organic soils.

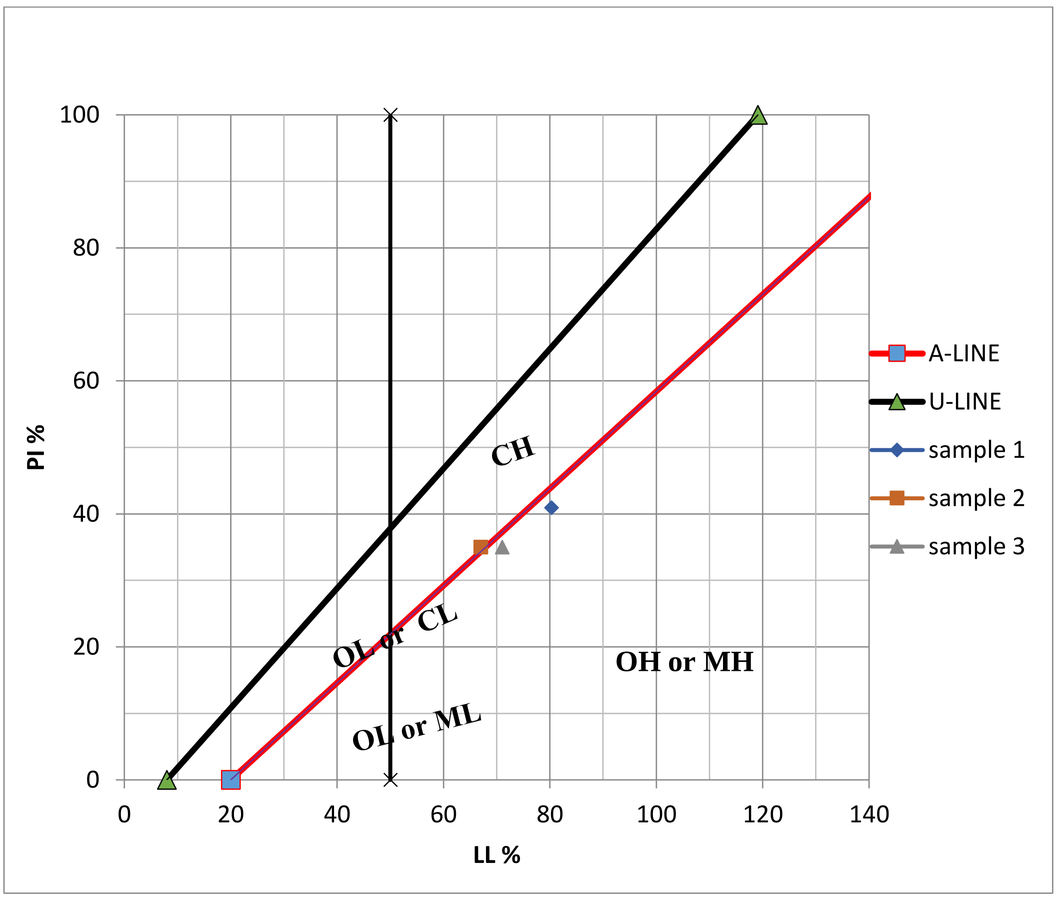


Figure 4.2: Soil classification according to USCS.

Table 4.3: Soil classification.

Sample no	Atterberg limit			Soil classification
	LL	PL	PI	USCS
1	80	39	41	CH
2	67	31	35	CH
3	71	36	35	CH

4.4. Specific gravity

The result of specific gravity of natural soil and 9%, 12%, and 15% of cement-treated soils is 2.58, 2.65, 2.66, and 2.67 respectively the result shows that as cement percent increases the specific gravity also increases.

4.5. Free swell result

The test is executed by pouring 10 cc. air-dried soil passing through sieve No 40 into a graduated cylinder. Then add clean water to the graduated cylinder up to 100 cc calibration point. Allow the soil particles to settle by setting the cylinder aside. The free swell value in percent is then calculated.

Table 4.4: Free swell result.

Sample name	Free swell %
Sample 1	65
Sample 2	58
Sample 3	52

This result indicated that the soils were highly compressible. It was supported by Rao (2007) Soils are called highly compressible when the free swell index exceeds 50% and such soils undergo volumetric changes due to seasonal wetting and drying.

4.6. Unconfined Compressive Strength

This test was conducted using the unconfined compression strength apparatus based on the *ASTM D2166-00* procedure. Figure 4.3 shows that, the stress-strain result of soil from the unconfined compression test (UCS). From the result, it is observed that the soils are soft soil especially sample 1. The specimens failed at a pressure of less than 50 kPa and greater than 50 kPa. Clay soils with unconfined compression strength less than 50 kPa and liquid limit greater than 50% are categorized under soft soils (Ara et al., 2021). Therefore, the results show that the soil samples meet the general principles of soft soil expressed in the literature.

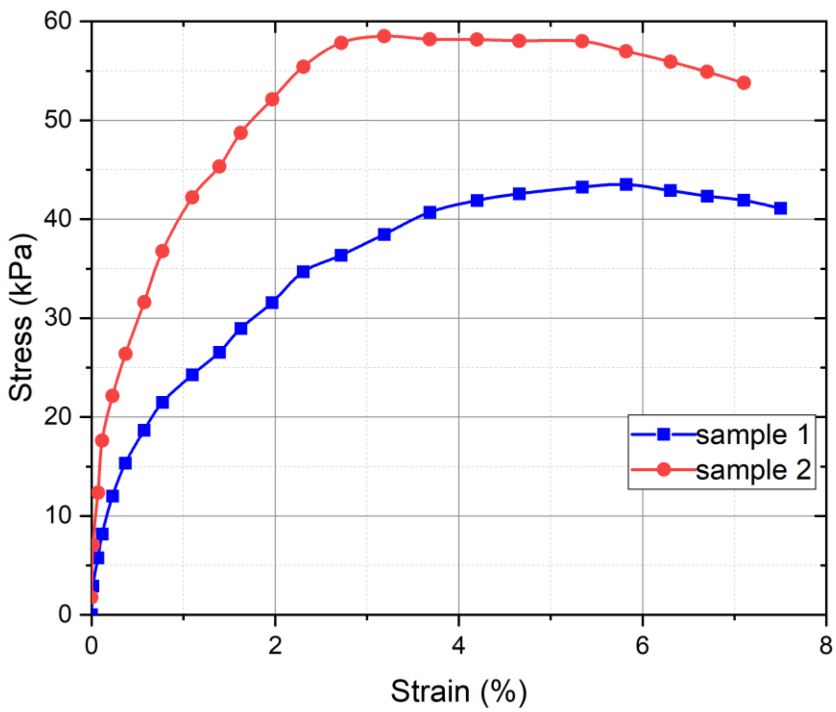


Figure 4.3: Stress-strain curves from UCS test.

Table 4.5: Unconfined compressive strength test result for two samples.

Sample no	$q_u(kPa)$	$C_u(kPa)$
S1	43.5	22.75
S2	58.5	29.25

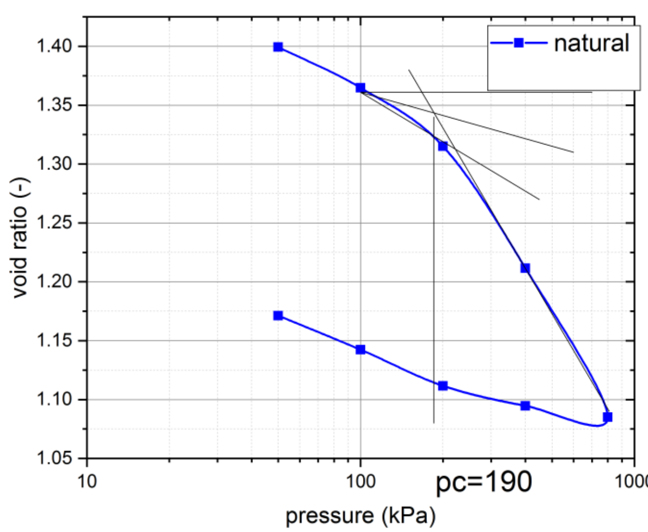
Table 4.5 shows a summary of the laboratory results from the unconfined compression strength test on the three samples.

4.7. One-Dimensional Consolidation Test

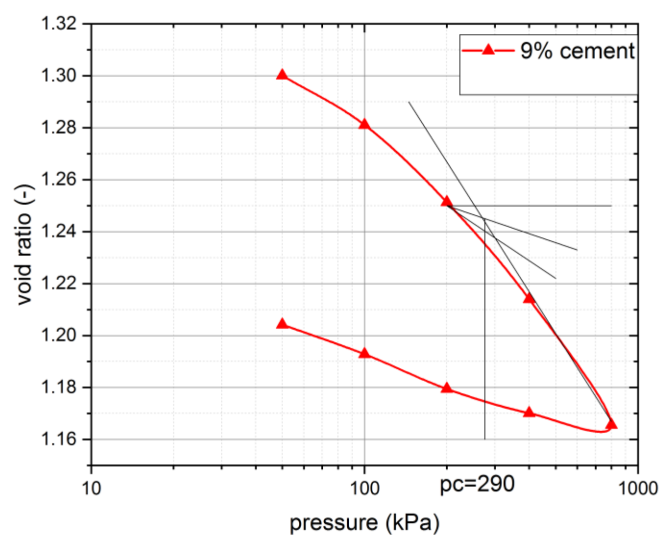
In one-dimensional consolidation test apparatus natural and remold stabilized sample was conducted to obtain compression parameters from void ratio versus applied pressure curve. The parameters such as the logarithmic compression index Cc, swelling index Cs, pre-consolidation stress, pc, and the initial void ratio, e.

4.7.1. Pre-Consolidation Pressure

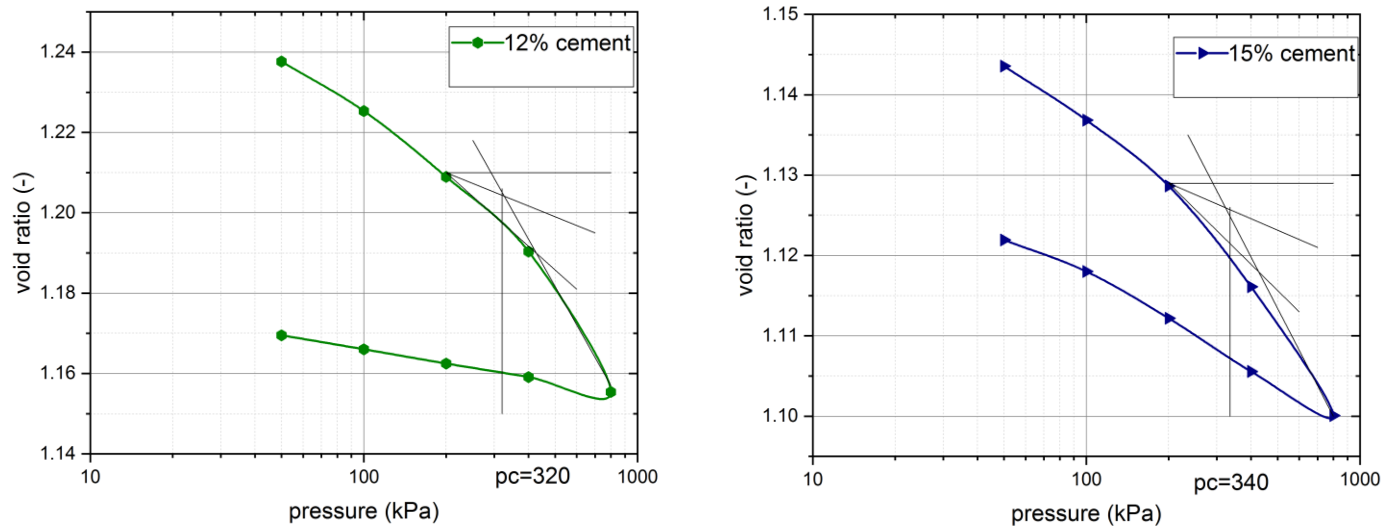
The pre-consolidation pressure, Pc for each of the soil samples was determined from the Void ratio versus log pressure curve using Casagrande's method**.** The pre-consolidation pressure of soil is defined as the highest stress the soil ever felt in its history. It is the pressure at which major structural changes including the breakdown of inter-particle bonds and inter-particle displacement begin to occur. The practical significance of the pre-consolidation load appears in calculating the settlements of structures. The effect of adding cement to the soft soil on pre-consolidation pressure is shown in Figure 4.5.



(a)



(b)



(c)

(d)

Figure 4.4: Void ratio-pressure for: (a) natural soil, (b) 9% cement, (c) 12% cement, (d) 15% cement

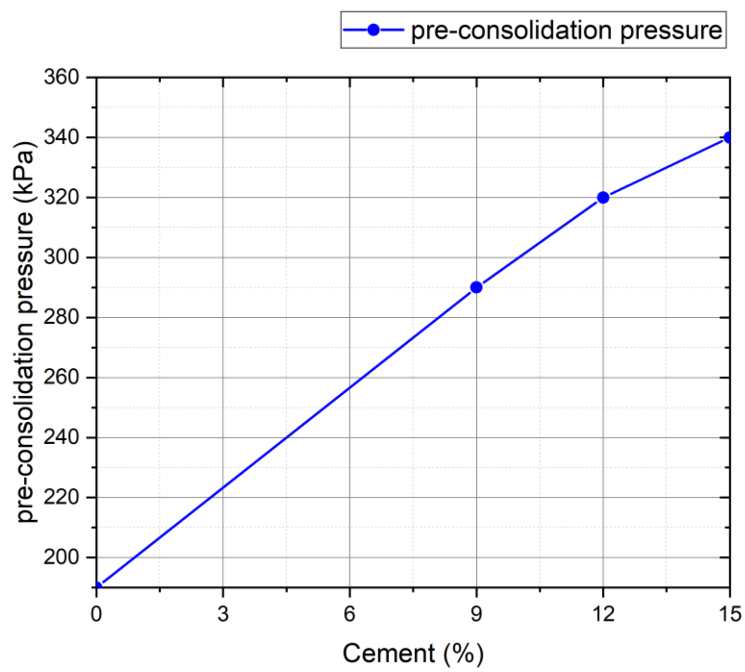


Figure 4.5: Variation of pre-consolidation pressure with cement content.

It can be noticed that the pre-consolidation pressure increased with increasing stabilizer content. This is a result of the soil-cement reaction. One can notice an increase in pre-consolidation pressure with the addition of cement mix and optimum percent of cement stabilization is 15% which is agreed with (Saadeldin & Siddiqua, 2013) .

4.7.2. Compression Index

The compression index (Cc) is an important parameter used in geotechnical engineering as it is related to the amount of anticipated consolidation settlement that a soil stratum will experience when introduced to loads that are greater than experienced in the past. The slope of the linear portion of the e log p curve is designated as the Cc. Results of consolidation tests with different percentages of cement are drawn figure and table.

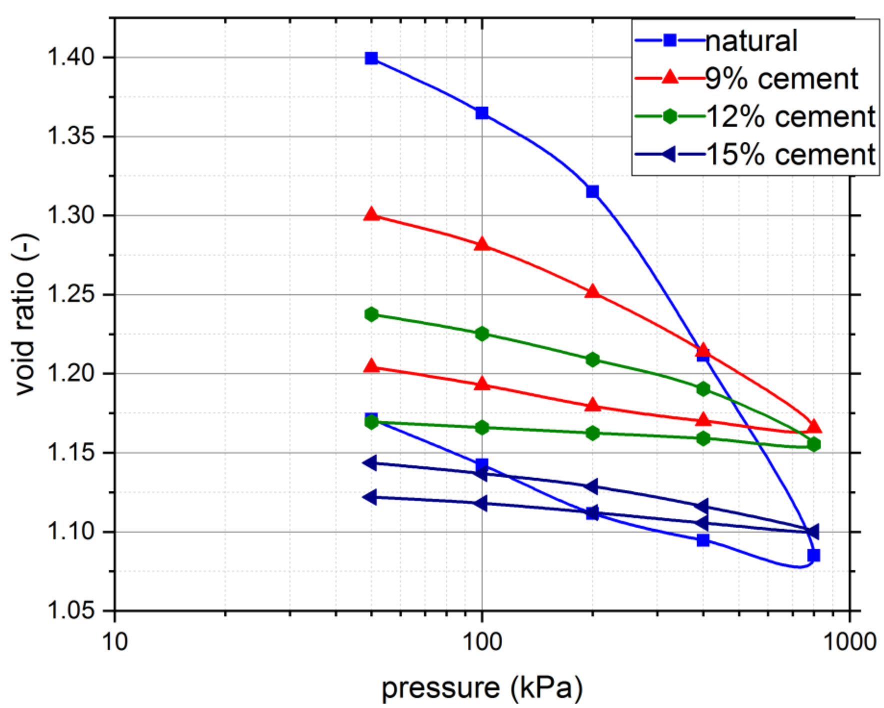


Figure 4.6: Void ratio versus effective stress curves from the consolidation test on soil stabilized with different percent of cement.

From Figure 4.6 above observed that the compressibility of natural soft clay soil is high which agreed with (Sorsa 2020) moreover, as increasing the cement content the compressibility of soils is decreased.

Table 4.6: Consolidation test result for natural and blended soft soil.

Location	Depth of sample(m)	Cement (%)	Intial void ratio (e_o)	Cc	Cs	Pc (Kpa)
Kebele-5	3m	0%	1.43	0.33	0.069	190
		9%	1.29	0.22	0.03	290
		12%	1.24	0.12	0.02	320
		15%	1.2	0.07	0.016	340

4.8. Triaxial test result

The triaxial test is a more useful test for determining the shear strength of fine-grained soil.

- Determination of parameters of triaxial test from diavotric stress- axial strain graph.

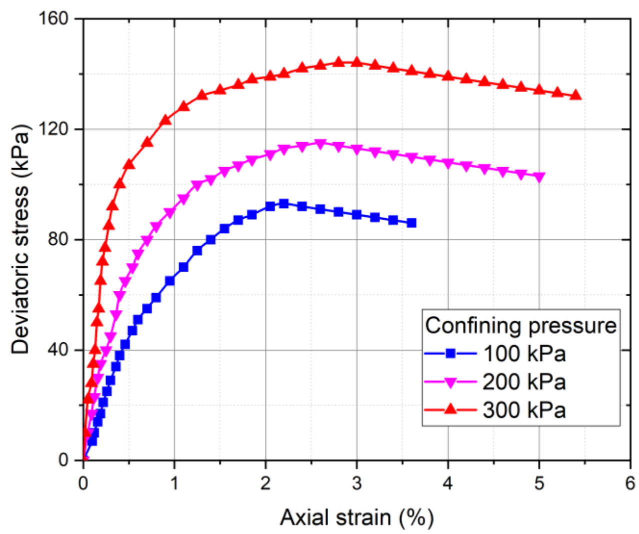


Figure 4.7: Deviatoric stress -axial strain of triaxial test of natural soil.

<u>For natural soil</u>

<u>At σ_3=200 kPa</u>

Peak diavotric stress =115 kPa; 50% diavotric stress = 57.5 kPa; 50% axial strain = 1.48%; therefore;

$E_{50}^{ref} = \frac{57.5}{0.0148}$=3883 kPa = E_{oed}^{ref}

Although $E_{ur}^{ref} = 3E_{50}^{ref}$ is sugessted by plaxis

E_{ur}^{ref} = 3*3883= 11500 kPa

4.9. Finite element method results

4.9.1 General

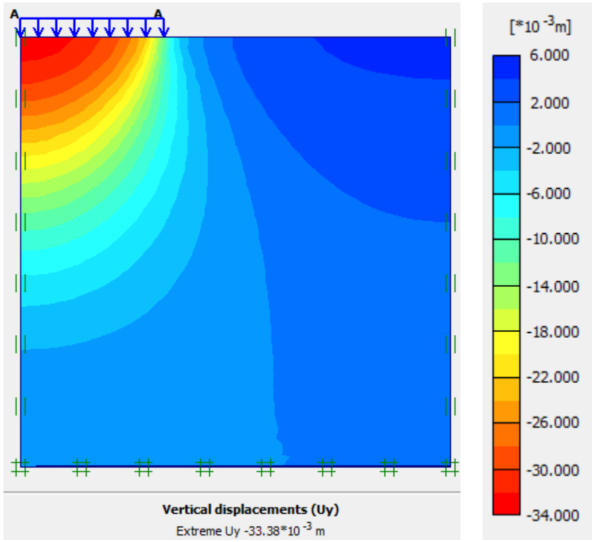
A finite element method is a common tool within various fields of engineering. It is used for advanced numerical calculations and is developed from the theories of continuum mechanics, which studies equilibrium, motion, and deformation of physical solids. FEM prerequisites that the mathematical models which describe the motions of the media have to be based on continuous functions. In FEM the continuous functions are approximated by a discrete model where the body to be studied is divided into several smaller parts, so-called elements. The discretized model is composed of several element functions that are continuous over each separate element. These elements are connected in nodes, which is primarily where the calculations are made.

4.10. Deformation results (vertical settlement)

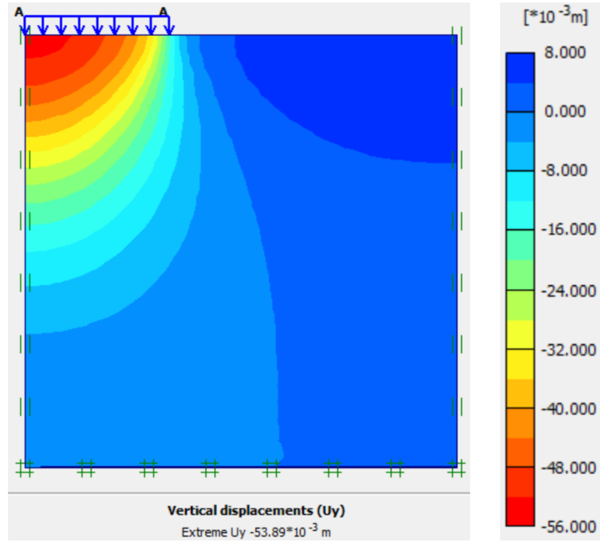
The simulation of the model geometry was carried out for vertical displacement by using the hardening soil model. The settlement graphical illustration of the model geometry with different added loads with different percentages of cement was presented.

4.10.1. Natural soil with different added loads.

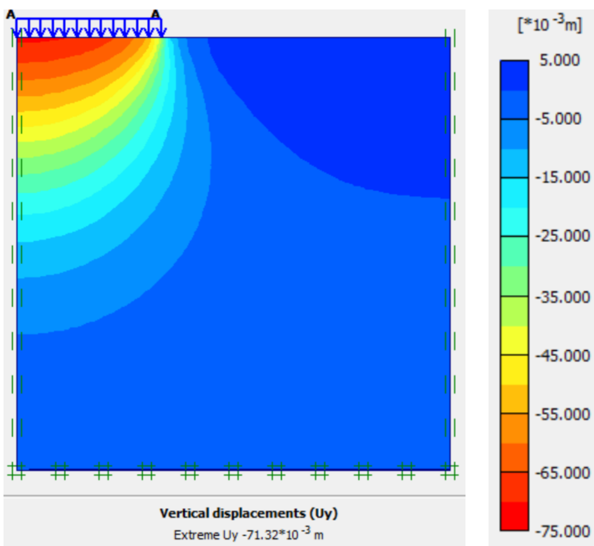
As depicted in Figure 4.8 below, the result of vertical displacement of natural soft clay by adding different loads in finite element analysis. The last value of settlement increased not linearly which main that soil could not hold that amount of load and was highly compressed. The reason for that is the soil is a soft and low value of bearing capacity.


[*10 -3m]
6.000
2.000
-2.000
-6.000
-10.000
-14.000
-18.000
-22.000
-26.000
-30.000
-34.000
Vertical displacements (Uy)
Extreme Uy -33.38*10 -3 m

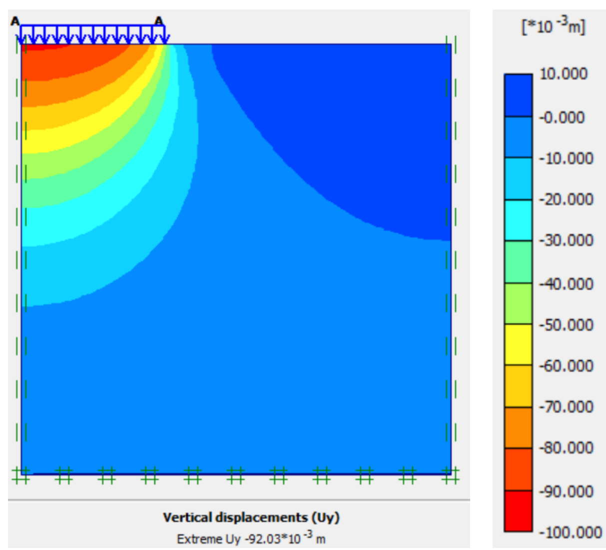
(a)


[*10 -3m]
8.000
0.000
-8.000
-16.000
-24.000
-32.000
-40.000
-48.000
-56.000
Vertical displacements (Uy)
Extreme Uy -53.89*10 -3 m

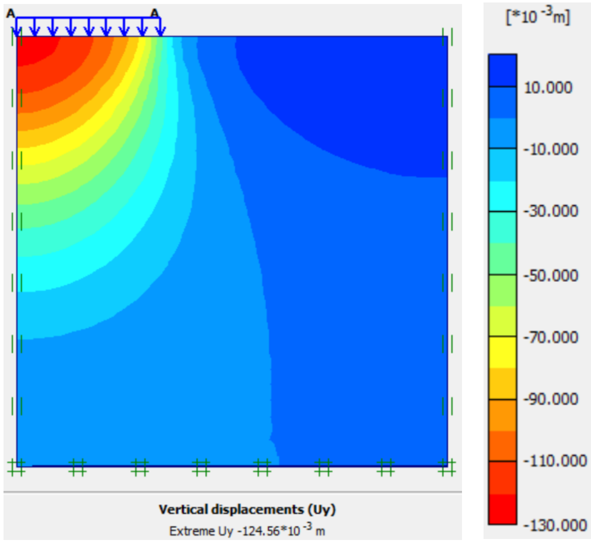
(b)

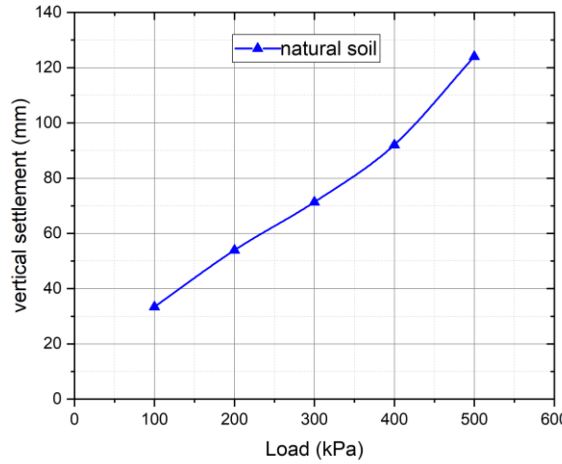

[*10 -3m]
5.000
-5.000
-15.000
-25.000
-35.000
-45.000
-55.000
-65.000
-75.000
Vertical displacements (Uy)
Extreme Uy -71.32*10 -3 m

(c)


[*10 -3m]
10.000
-0.000
-10.000
-20.000
-30.000
-40.000
-50.000
-60.000
-70.000
-80.000
-90.000
-100.000
Vertical displacements (Uy)
Extreme Uy -92.03*10 -3 m

(d)



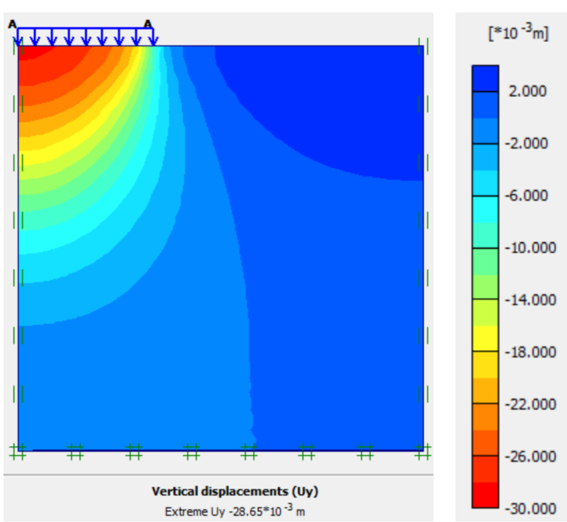


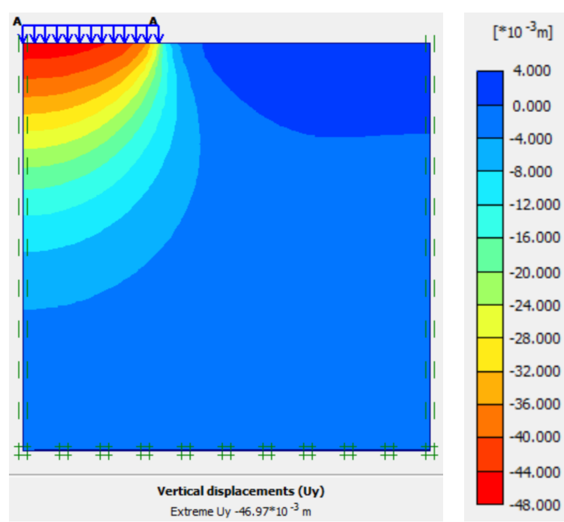
(e) (f)

Figure 4.8: Load-settlement curve of natural soil.

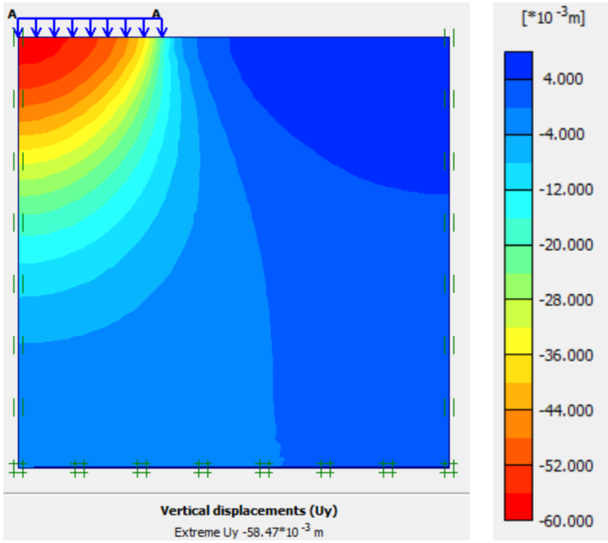
4.10.2. Nine percent cement stabilized soil with different added loads.

Figure 4.9 shows the relationship between settlement and load under 9% of cement stabilized soft clay. From the graph as increasing the applied loads the settlement also increases but when compared to natural soft clay the magnitude of the vertical settlement in the final applied load is 81.68mm. It is also clearly shown that the graph is increasing in linear scale with low values settlement due to soil-cement reaction.

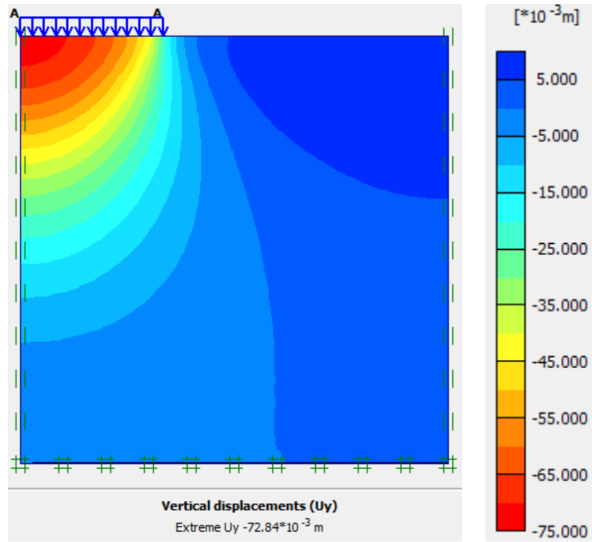




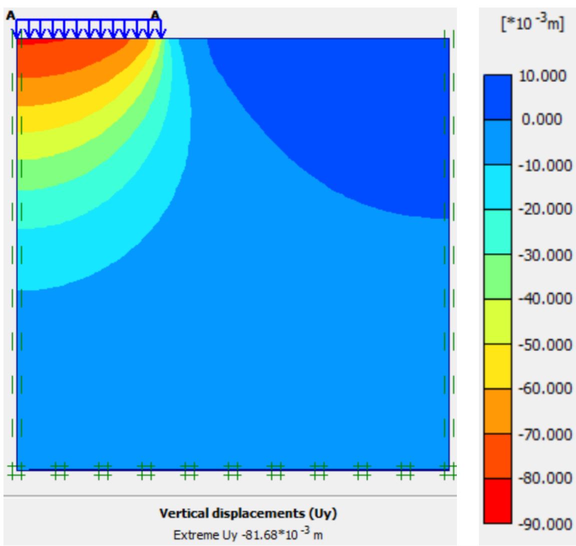
(a) (b)



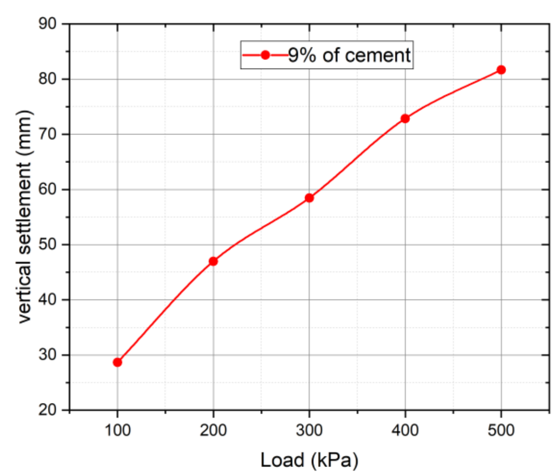
(c)



(d)



(e)

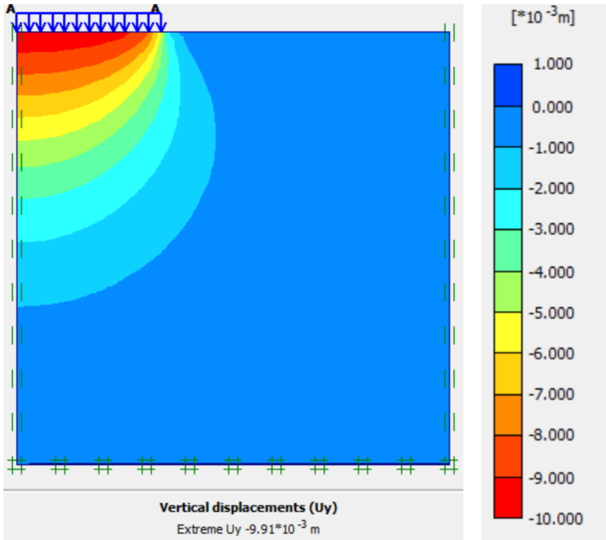


(f)

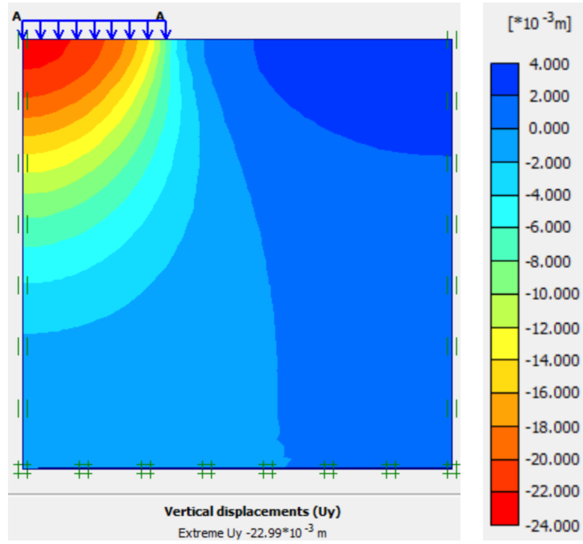
Figure 4.9: Load-settlement curve of 9% of cement stabilized soil.

4.10.3. Twelve percent of cement stabilized soil with different added loads.

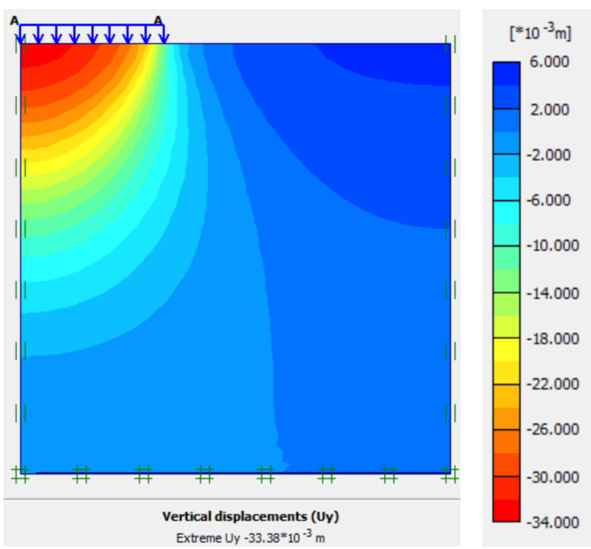
Figure 4.10 illustrated below the results extracted from PLAXIS software for 12% of cement stabilized soft clay. The settlement values were less due to the high value of bearing capacity as compared to 9% of cement stabilized.


[*10 -3m]
1.000
0.000
-1.000
-2.000
-3.000
-4.000
-5.000
-6.000
-7.000
-8.000
-9.000
-10.000
Vertical displacements (Uy)
Extreme Uy -9.91*10 -3 m

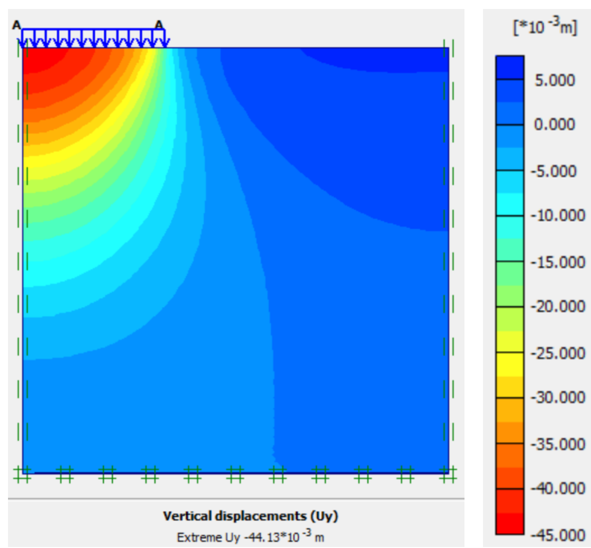
(a)


[*10 -3m]
4.000
2.000
0.000
-2.000
-4.000
-6.000
-8.000
-10.000
-12.000
-14.000
-16.000
-18.000
-20.000
-22.000
-24.000
Vertical displacements (Uy)
Extreme Uy -22.99*10 -3 m

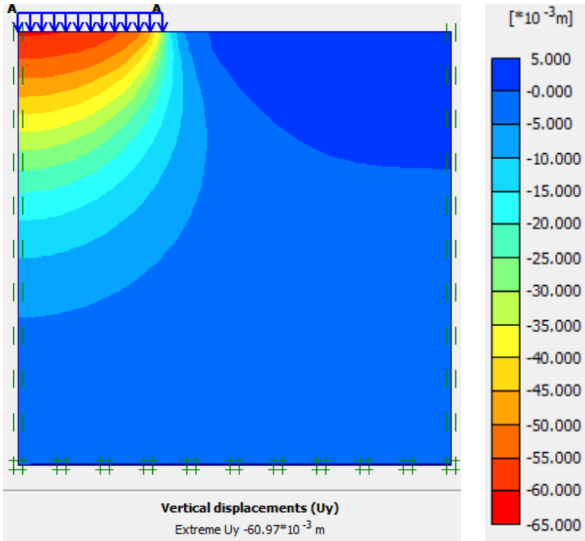
(b)


[*10 -3m]
6.000
2.000
-2.000
-6.000
-10.000
-14.000
-18.000
-22.000
-26.000
-30.000
-34.000
Vertical displacements (Uy)
Extreme Uy -33.38*10 -3 m

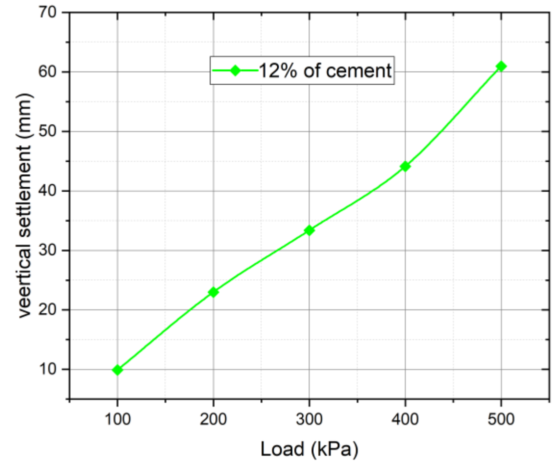
(c)


[*10 -3m]
5.000
0.000
-5.000
-10.000
-15.000
-20.000
-25.000
-30.000
-35.000
-40.000
-45.000
Vertical displacements (Uy)
Extreme Uy -44.13*10 -3 m

(d)



(e)

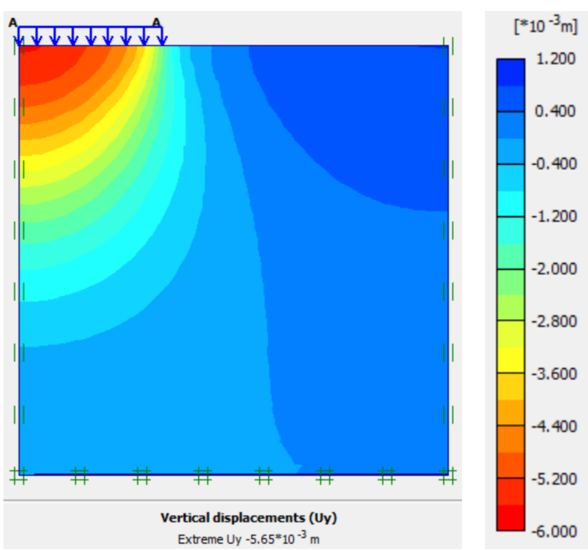


(f)

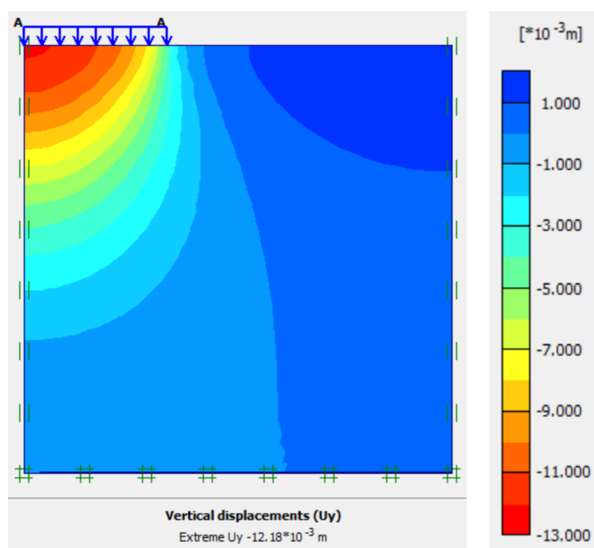
Figure 4.10: Load-settlement curve of 12% cement stabilized soil.

4.10.4. Fifteen percent of cement stabilized soil with different added loads.

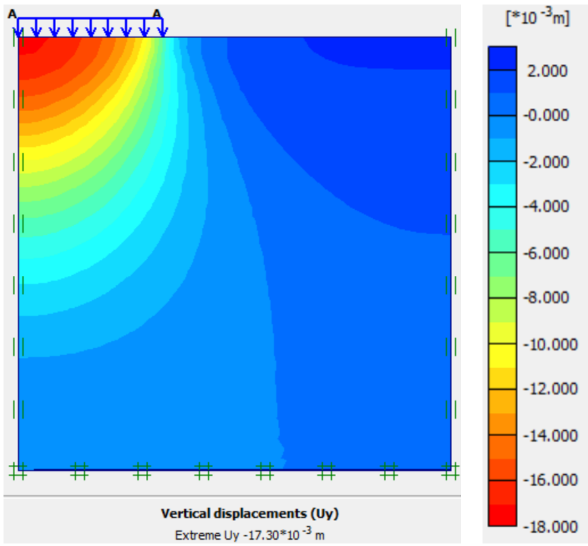
Figure 4.11. Relationship between load-vertical settlement of 15% of cement stabilized soft clay. The settlement value was less compared to 12% of cement stabilized soil. It is also clearly shown that the graph is increasing in linear scale with low values due to soil-cement reaction and high bearing capacity value.



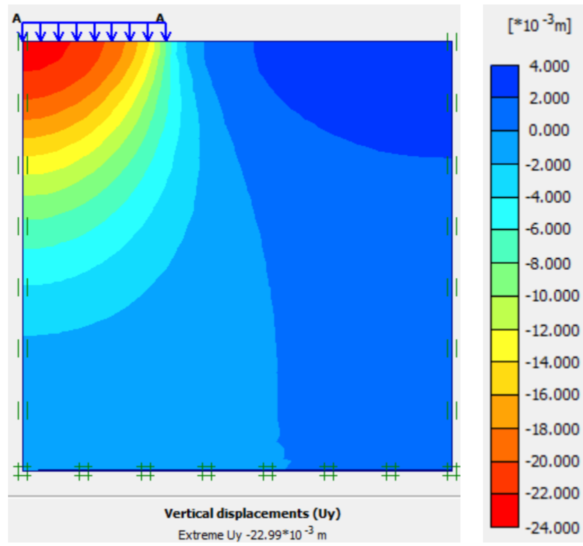
(a)



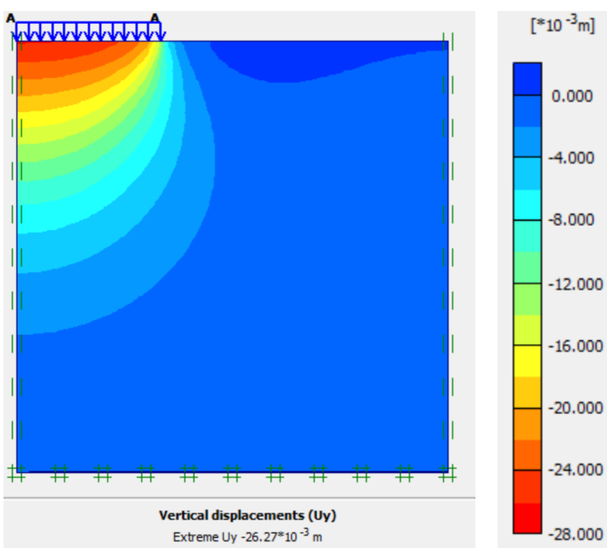
(b)



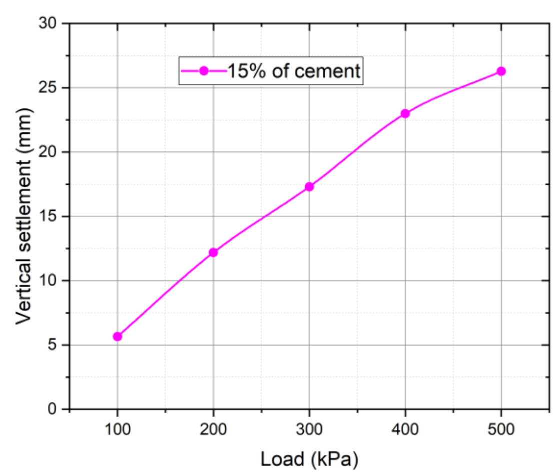
(c)



(d)



(e)



(f)

Figure 4.11: Load-settlement curve of 15% of cement stabilized soil.

4.11. Comparison of untreated soft soil with treated soft soil from FEM

The main aim of the study is to investigate the deformation analysis of cement-modified soft clay through numerical modeling and comparing the result of untreated soft soil with treated soft soil.

From finite element analysis the displacement encountered in untreated soft clay was 124 mm. and the displacement of 15% of cement is 26.77 mm. It is observed that the displacement is improved when soft clay soil is modified with 15% of cement. There was a high difference in the magnitude of untreated and treated soft soils this is a result of soil cement reaction and the curing period of the specimen also affects the results.

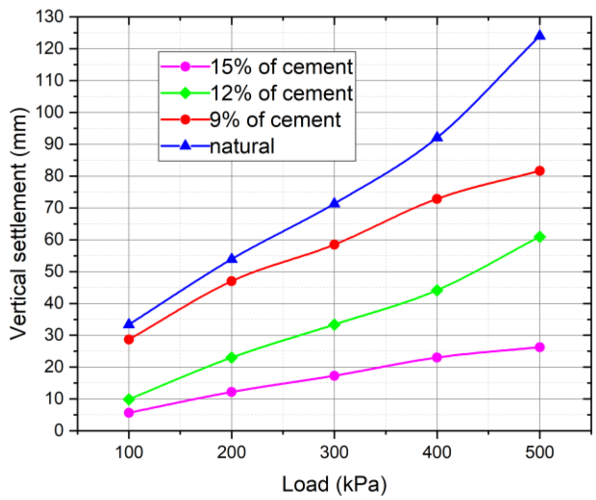


Figure 4.12: Load-settlement curve extracted from plaxis software.

4.13. Validation of finite element method with literature.

This section presents the validation of the soil model with literature. The finite element analysis for the soil geometry of raft foundation (25x60) m under various added loads of 56, 63.5, 68, 75, and 93 kPa studied using Hardening soil model. The computational geometry, the applied loads and the soil model are similar with Al-Taie et al., (2016) however the input parameters used in HS model for the simulation are obtained from the experimental result of this work. As a result, the vertical settlement of this work is in good agreement with the literature as shown in Figure 4.13. Detail analysis is presented in appendix C.

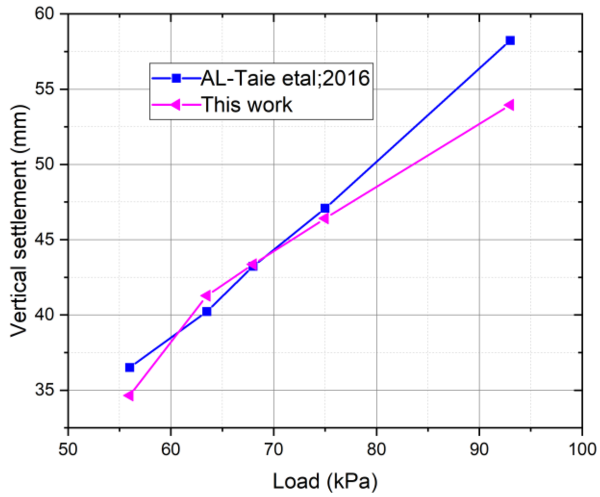


Figure 4.13: Validation of the FEM result with literature.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1. CONCLUSIONS

This research had been successfully conducted and showed that soft soil stabilization by cement would increase the shear strength and reduced the deformation that occurred. From the findings of this research, the objective which was to investigate the deformation of stabilized soft soil with cement had been achieved. Detailed descriptions of the outcome can be concluded as follows:

Based on the laboratory test results obtained the following conclusion may be drawn:

Mechanical grain size analysis shows that more than 85% of the total masses pass through a sieve size of 75μm. This indicates that the selected soil samples can be taken as fine-grained soils. From hydrometer analysis, for most of the samples, more than 45% of their grain sizes are less than 5μm (clay size range as per ASTM boundaries criteria). This suggests that these types of soils are highly influenced by the presence of clay content.

From the unconfined compressive strength result, it is observed that the soils are soft soil. The specimens failed at a pressure of less than 50 kPa and greater than 50 kPa. Clay soils with unconfined compression strength less than 50 kPa and liquid limit greater than 50% are categorized under soft soils.

The pre-consolidation pressure increased with increasing stabilizer content. The result of pre-consolidation pressure ranges from 190kPa to 340kPa and the optimum percent of cement is 15%. The compression and swelling indices of untreated and treated soils are varied from 0.33 to 0.07 and 0.069 to 0.016 respectively. This indicates that decreases in the compressibility index and swelling indices as cement stabilization increases.

Based on Plaxis 2d result, the vertical displacement of natural soft clay was investigated by adding different loads in finite element analysis the result indicates that the last value of settlement increased not linearly which main that soil could not hold that amount of load and was highly compressed. From the graph of 9% of cement stabilized soil, as increasing the applied loads the settlement also increases but when compared to natural soft clay the magnitude of the vertical settlement in the final applied load is 81.68mm. It is also clearly shown that the graph is increasing in linear scale with low values settlement which holds the amount of loads due to soil-cement reaction. for 12% of cement stabilized soft clay the settlement values were less compared to 9% of cement stabilized soil these indicates that as increasing the percent of cement the settlement value is decreases.

The displacement encountered in untreated soft clay was 124 mm. and the displacement of 15% of cement is 26.77 mm. It is observed that the displacement is improved when soft clay soil is modified with 15% of cement. There was a high difference in the magnitude of untreated and treated soft soils this is a result of soil cement reaction and the curing period of the specimen also affects the results.

5.2. RECOMMENDATIONS

Scope for Future Works

- ✓ Other shapes of foundations can be considered. Axisymmetric problems can be analyzed using PLAXIS.
- ✓ PLAXIS 3D can be recommended for further study.
- ✓ Other FEM software like Abaqus can be used for analysis.
- ✓ The study conducted cement stabilization for the analysis of deformation of the soft soil, for further studies locally available materials such as fly ash, stone dust, and selective materials as low-cost to soft soil stabilization to be recommended.

REFERENCES

AASHTO. (2008). Recommended Practice for Stabilization of Subgrade Soils and Base Materials. In *Recommended Practice for Stabilization of Subgrade Soils and Base Materials* (Issue August). https://doi.org/10.17226/22999

Al-Khafaji, Z. S., Jafer, H., Dulaimi, A., Atherton, W., & Al Masoodi, Z. (2017). The Soft Soil Stabilisation Using Binary Blending of Ordinary Portland Cement And High Alumina Silica Waste Material. *The 3rd BUiD Doctoral Research Conference, the British University in Dubai, 13th May 2017, UAE*.

Al-Taie, Entidhar, Al-Ansari, Nadhir, Knutsson, & Sven. (2016). Evaluation of Foundation Settlement under Various Added Loads in Different Locations of Iraq Using Finite Element. *Engineering*, *08*(05), 257–268. https://doi.org/10.4236/eng.2016.85022

Ara, S., Uddin, M. S., & Showkat, N. H. (2021). *Correlation of Compression Index with Index Properties of Soil Samples from Several Places in Chattogram , Bangladesh* . 752–755.

Bowles. (1996). Foundation analysis and design.Mcgraw-hill book co.Fifth edition. In *Civil Engineering Materials*. https://doi.org/10.1007/978-1-349-13729-9_26

Brinkgreve. (2005). Selection of soil model and parameters for geotechnical engineering application. In *Geo-Frontiers Congress 2005*.

Budhu, M. (2010). *Soil mechanics and foundations*.

Chen, H., & Wang, Q. (2006). The behaviour of organic matter in the process of soft soil stabilization using cement. *Bulletin of Engineering Geology and the Environment*, *65*(4), 445–448. https://doi.org/10.1007/s10064-005-0030-1

Chin. (2005). *Embankment over Soft Clay – Design and Construction Control*. *September*, 1–15.

Devi, D. (2014). *On the determination of modified cam clay model parameters*. *3*(4), 250–253.

Dixit, M. S. (2016). Damage mechanism in problematic soils. *International Journal of Civil Engineering and Technology*, *7*(5), 232–241.

Diyana, R, N. U., Nordin, & Binti. (2010). *the Potential of Cement Stabilization in Soft Soil* (Issue

December).

Fox, P. J. (2003). *19.1 Components of Total Settlement* (pp. 1–15).

Gebby, Y. (2005). *Prefabricated Vertical Pvc Drainage System for Construction of Embankment on Compressible Soft Soil Report No . Ge- R-68*. 1–32.

Gebreselassie, B., & Kempfert, H.-G. (2006). *Excavations and Foundations in soft soils*.

Gens, A., Potts, & M., D. (1988). Critical state models in computational geomechanic*. *Engineering Computations*, *5*(3), 178–197. https://doi.org/10.1108/eb023736

Gupta, S., & Mital, A. (2019). Numerical analysis of bearing capacity of rectangular footing. *Journal of Physics: Conference Series*, *1240*(1). https://doi.org/10.1088/1742-6596/1240/1/012039

Hawsan. (2004). *Prefabricated Vertical Pvc Drainage System for Construction of Embankment on Compressible Soft Soil*. 1–32.

Hinchberger. (2009). Three-dimensional effects on the behaviour of test embankments. *Canadian Geotechnical Journal*, *46*(11), 1356–1370. https://doi.org/10.1139/T09-060

Ismail, Araz, Teshome, & Fitsum. (2011). *Analysis of deformations in soft clay due to unloading*.

Jiang, J. (2011). Physical and mechanical characteristics of soft soil in lower reaches of Yangtze River. *Proceedings of the 2011 2nd International Conference on Digital Manufacturing and Automation, ICDMA 2011*, *1*, 851–854. https://doi.org/10.1109/ICDMA.2011.210

Kalantari, B. (2012). Foundations on expansive soils: A review. *Research Journal of Applied Sciences, Engineering and Technology*, *4*(18), 3231–3237.

Lee, boris. (1990). implicit integration of elasto plastic constitutive relations. *International Journal for Numerical and Analytical Methods in Geomechanics*, 49–72. https://doi.org/10.1002/nag.155

Luévanos-rojas, A. (2016). *A comparative study for the design of rectangular and circular isolated footings using new models Un estudio comparativo para diseño de zapatas aisladas de forma rectangular y circular usando nuevos modelos*. *83*(196), 149–158.

Makusa P., & Gregory. (2013). *A review of geotechnical behavior of stabilized soils design and analysis considerations*.

Maranha, J. R., Neves, Das, M., & E. (2000). *The experimental determination of the angle of dilatancy in soils*.

Muhammed, J. J., & Teferra, A. (2014). In-depth Investigation into Engineeering Characteristics of Jimma Soils. In *AAU-AiT*.

Murthy, V. N. S. (1990). *Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering*.

Nelson, J. D., Chao, K. C., Overton, D. D., & Nelson, E. J. (2015). Foundation Engineering for Expansive Soils. In *Foundation Engineering for Expansive Soils*. https://doi.org/10.1002/9781118996096

Obrzud, R. (2016). *Constitutive models for geotechnical practice*. *August*.

Onyelowe, Kennedy, Van, Bui, Duc, Igboayaka, Clifford, Orji, Francis, Ugwuanyi, & Henry. (2019). Rheology of mechanical properties of soft soil and stabilization protocols in the developing countries-Nigeria. *Materials Science for Energy Technologies*, *2*(1), 8–14. https://doi.org/10.1016/j.mset.2018.10.001

Patel, M., & Bhoi, M. (2019). Effect of different shape of footing on its load-settlement behaviour (circular, square and rectangular). *World Congress on Civil, Structural, and Environmental Engineering*, 1–7. https://doi.org/10.11159/icgre19.168

Plaxis, M. (2000). *PLAXIS Version 8 Manual*.

Por, S., Nishimura, Satoshi, & Likitlersuang, S. (2017). Deformation characteristics and stress responses of cement-treated expansive clay under confined one-dimensional swelling. *Applied Clay Science*, *146*(March), 316–324. https://doi.org/10.1016/j.clay.2017.06.022

Rezaei, M., Ajalloeian, R., & Ghafoori, M. (2012). Geotechnical Properties of Problematic Soils Emphasis on Collapsible Cases. In *International Journal of Geosciences* (Vol. 03, Issue 01). https://doi.org/10.4236/ijg.2012.31012

Saadeldin, R., & Siddiqua, S. (2013). Geotechnical characterization of a clay-cement mix. *Bulletin of Engineering Geology and the Environment*, *72*(3–4), 601–608. https://doi.org/10.1007/s10064-013-0531-2

Saadeldin, Ramy, Salem, A, M., Lotfi, & A, H. (2011). Performance of Road Embankment on Cement Stabilized Soft Clay. *Pan-Am CGS Geotechnical Confrence*.

Shahangian, S. (2011). *Variable Cohesion Model for Soil Shear Strength Evaluation*. *C*.

Shooshpasha, I., Amiri, I., & MolaAbasi, H. (2015). An investigation of friction angle correlation with geotechnical properties for granular soils using GMDH type neural networks. *Scientia Iranica*, *22*(1), 157–164.

Sorsa, A., Senadheera, S., & Birru, Y. (2020). Engineering characterization of subgrade soils of Jimma town, Ethiopia, for roadway design. *Geosciences (Switzerland)*, *10*(3), 1–17. https://doi.org/10.3390/geosciences10030094

Terzaghi, Karl, Peck, B., R., & Mesri, G. (1996). *Terzaghi, K. Peck R. - Soil Mechanics in Engineering Practice*.

Topolinski, S. (2019). Unconfined Compressive Strength Properties of a Cement- Organic Soil Composite. *IOP Conference Series: Materials Science and Engineering*, *471*(4), 0–8. https://doi.org/10.1088/1757-899X/471/4/042018

Towhata, & Ikuo. (2009). Soils and Foundations: *Soils and Foundations*, *49*(6). https://doi.org/10.3208/sandf.49.F2

Wikipedia. (2020). *Jimma - Wikipedia*. https://en.wikipedia.org/wiki/Jimma

Yonas, S. (2002). *a Historical Survey of Jimma Town* (Issue June).

Zambri, Mohd, N., Limit, & Atterberg, A. (2017). *Deformation Characteristics of Lime Stabilized Peat*. *December*, 21–25.

Zdravkovic, potts and. (2001). Finite Element Analysis in Geotechnical Engineering: Volume One - Theory. *Finite Element Analysis in Geotechnical Engineering: Volume One - Theory*. https://doi.org/10.1680/feaiget.27534

APPENDICES

APPENDIX A

Table A.1: Natural moisture content.

Sample location	S1	
Can number	1	2
Mass of moisture can(Mc)	17.5	17.8
Mass of moisture can + mass of moist soil (Mcms)	139.75	142.2
Mass of moisture can + mass of oven dried soil(Mcds)	92.23	95.76
Mass of water (Mw)	47.52	46.44
Mass of dry soil(Md)	74.73	77.96
Water content (W) %	63.58	59.56
Average water content %	61.57	

Grain size analysis

Table A.2: Wet sieve analysis.

Sieve size (mm)	Mass retained (gm)	Percentage retained (%)	Cumulative percentage retained (%)	Percentage passing (%)
4.75	0.8	0.08	0.08	99.92
2	9.5	0.95	1.03	98.97
0.85	6.2	0.62	1.65	98.35
0.425	3.3	0.33	1.98	98.02
0.3	1.5	0.15	2.13	97.87
0.25	1.2	0.12	2.25	97.75
0.15	3.1	0.31	2.56	97.44
0.075	6.5	0.65	3.21	96.79
Pan	967.9	96.79	100	0
Total mass	1000			

Table A.3: Hydro meter analysis.

Time (min)	Rh	T	Tc	Rc	% finer	Re-corrected meniscus	L	K	D(mm)	Actual finer(%)
1	48	23	0.9	41.9	83.8	49	8.428	0.013445	0.03903	81.11002
2	47	23	0.9	40.9	81.8	48	8.592	0.013445	0.02787	79.17422
5	46	23	0.9	39.9	79.8	47	8.756	0.013445	0.01779	77.23842
15	45	23	0.9	38.9	77.8	46	8.92	0.013445	0.01037	75.30262
30	44	23	0.9	37.9	75.8	45	9.084	0.013445	0.0074	73.36682
60	42	23	0.9	35.9	71.8	43	9.412	0.013445	0.00532	69.49522
120	40	23	0.9	33.9	67.8	41	9.74	0.013445	0.00383	65.62362
240	38	23	0.9	31.9	63.8	39	10.068	0.013445	0.00275	61.75202
480	36	23	0.9	29.9	59.8	37	10.396	0.013445	0.00198	57.88042
960	32	23	0.9	25.9	51.8	33	11.052	0.013445	0.00144	50.13722
1440	30	23	0.9	23.9	47.8	24	11.38	0.013445	0.0012	46.26562

Table A.4: Combined sieve analysis.

	kebele 5	Ju stadium	Merkato
grain size (mm)	**combined % passing**	% Passing	% Passing
4.75	99.92	98.5	98.68
2	98.97	97.5	96.98
0.85	98.35	96.1	94.31
0.425	98.02	96.1	91.38
0.3	97.87	94.8	90.62
0.25	97.75	92.6	89.68
0.15	97.44	91.2	88.1
0.075	96.79	84.8	86.4

0.03903	81.11002	74.74	65.4
0.02787	79.17422	73.73	64.5
0.01779	77.23842	66.66	58.3
0.01037	75.30262	62.62	54.8
0.00740	73.36682	58.58	51.2
0.00532	69.49522	54.54	47.7
0.00383	65.62362	50.5	44.2
0.00275	61.75202	44.44	38.9
0.00198	57.88042	43.43	38
0.00144	50.13722	38.38	33.6
0.00120	46.26562	34.34	30

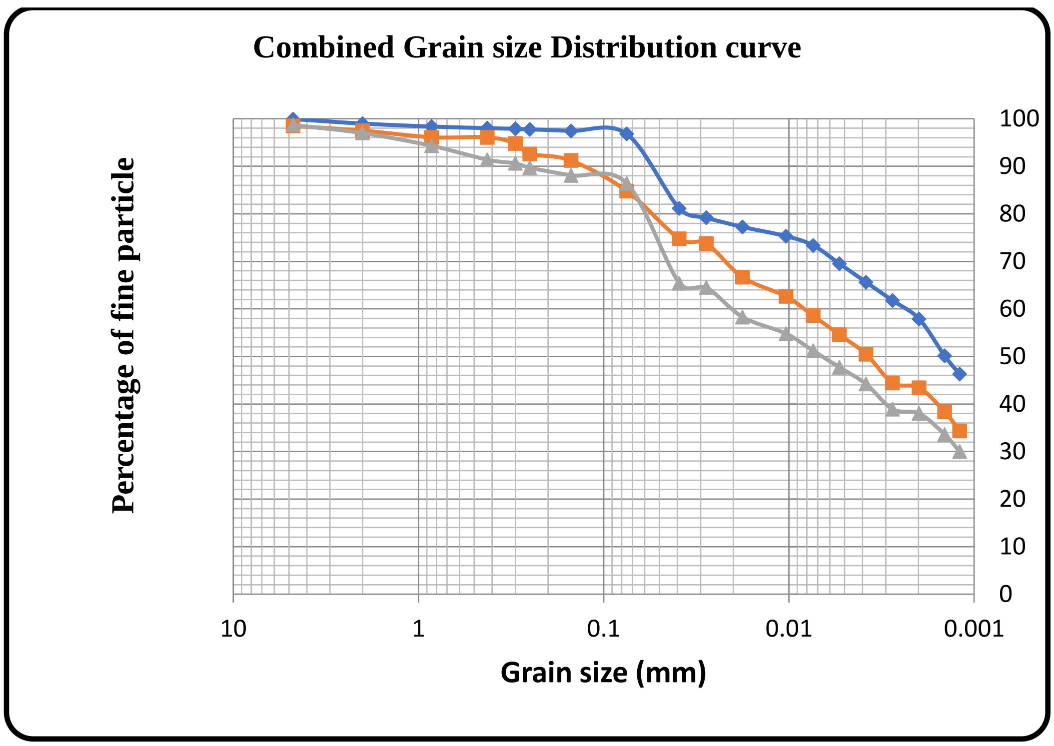


Figure A.1: Combined grain size distribution curve.

Table A.5: Specific gravity of natural soil (kebele 5).

Trial	1	2	3
Mass of dry, clean Calibrated pycnometer, M_p	25.995	22.544	27.45
Mass of specimen + pycnometer, $M_{ps,}$ in g	50.995	47.544	52.45
Mass of pycnometer + soil + water, M_{psw}, in g	136.564	133.996	139.836
Temperature of contents of pycnometer when M_{psw} was taken, Ti, in ^{o}c	25	25	25
Density of water @ Ti in g/cm^3	0.99707	0.99707	0.99707
Mass of pycnometer + water at temperature Ti,g	121.37	118.631	124.332
Tx in $^{o}{}_{c}$	24	24	24
Density of water @ Tx in g/cm^3	0.99732	0.99732	0.99732
K @ Tx	0.9991	0.9991	0.9991
Mass of pycnometer + water at temperature Tx,g	121.39	118.66	124.36
Specific gravity @ 20^{o}c	2.54	2.59	**2.62**
Average Specific gravity at 20^{o}c, G_s	**2.58**		

Table A.6: 9% cement of specific gravity.

Trial	1	2	3
Mass of dry, clean Calibrated pycnometer, M_p	22	26.9	26.6
Mass of specimen + pycnometer, $M_{ps,}$ in g	47	51.9	51.6
Mass of pycnometer + soil + water, M_{psw}, in g	133.2	133.6	132.8
Temperature of contents of pycnometer when M_{psw} was taken, Ti, in ^{o}c	24	24	24
Density of water @ Ti in g/cm^3	0.99732	0.99732	0.99732
Mass of pycnometer + water at temperature Ti,g	117.6	117.9	117.3
Tx in $^{o}{}_{c}$	22	22	22
Density of water @ Tx in g/cm^3	0.9978	0.9978	0.9978
K @ Tx	0.9996	0.9996	0.9996
Mass of pycnometer + water at temperature Tx,g	117.65	117.94	117.34
Specific gravity @ 20^{o}c	2.65	2.67	**2.62**
Average Specific gravity at 20^{o}c, G_s	**2.65**		

Table A.7: 12% of cement specific gravity.

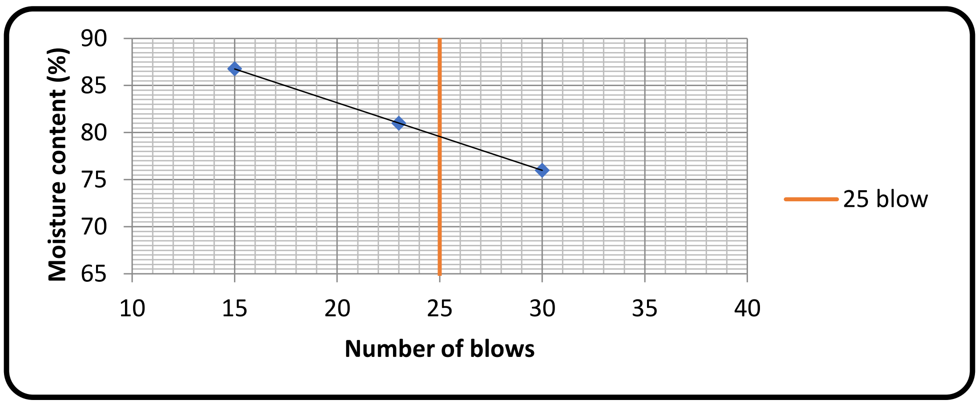
Trial	1	2	3
Mass of dry, clean Calibrated pycnometer, M_p	23	26.8	26.6
Mass of specimen + pycnometer, $M_{ps,}$ in g	48	51.9	51.7
Mass of pycnometer + soil + water, M_{psw}, in g	133.4	133.6	132.9
Temperature of contents of pycnometer when M_{psw} was taken, Ti, in ^oc	24	24	24
Density of water @ Ti in g/cm^3	0.99732	0.99732	0.99732
Mass of pycnometer + water at temperature Ti,g	117.6	117.9	117.3
Tx in $^o{}_c$	22	22	22
Density of water @ Tx in g/cm^3	0.9978	0.9978	0.9978
K @ Tx	0.9996	0.9996	0.9996
Mass of pycnometer + water at temperature Tx,g	117.65	117.94	117.34
Specific gravity @ 20^{o}c	2.70	2.66	**2.63**
Average Specific gravity at 20^{o}c, G_s	**2.66**		

Table A.8: 15% of cement specific gravity.

Trial	1	2	3
Mass of dry, clean Calibrated pycnometer, M_p	25.995	22.544	27.45
Mass of specimen + pycnometer, $M_{ps,}$ in g	50.995	47.544	52.45
Mass of pycnometer + soil + water, M_{psw}, in g	137.164	134.2	139.936
Temperature of contents of pycnometer when M_{psw} was taken, Ti, in ^oc	25	25	25
Density of water @ Ti in g/cm^3	0.99707	0.99707	0.99707
Mass of pycnometer + water at temperature Ti,g	121.37	118.631	124.332
Tx in $^o{}_c$	24	24	24
Density of water @ Tx in g/cm^3	0.99737	0.99737	0.99737
K @ Tx	0.9991	0.9991	0.9991
Mass of pycnometer + water at temperature Tx,g	121.40	118.66	124.36
Specific gravity @ 20^{o}c	2.70	2.64	**2.65**
Average Specific gravity at 20^{o}c, G_s	**2.67**		

Table A.9: Liquid limit test result for sample one.

Determination	***Liquid Limit (D-4318)***		
Number of blows	30	23	15
Test N<u>o</u>	01	02	03
Wt. of Container, (g)	25.2	16.4	25.2
Wt. of container + wet soil, (g)	49.1	46.6	50.9
Wt. of container + dry soil, (g)	38.66	33.5	38.96
Wt. of water, (g)	10.44	13.10	11.94
Wt. of dry soil, (g)	13.46	17.10	13.76
Moisture container, (%)	77.56	76.61	86.77
Average	80.31		



LL= 80.31%

Figure A.2: Liquid limit graph

Table A.10: Plastic limit test result.

PI=LL-PL

PI=80.31-39.36

PI=40.95

Plastic limit test		
Can no.	01	02
Mass of can	6.5	6
Mass of can plus wet soil	15.8	17.5
Mass of can plus dry soil	13.05	14.41
Mass of moisture	2.75	3.09
Mass of dry soil	6.55	8.41
Mosture content,W	41.98	36.74
Plastic limit %		**39.36**

APPENDIX B

For sample one (kebele 5)

Table B.1: Unconfined compressive strength.

Load in kN	Sample deformation (mm)	Strain in %	Corrected area in cm2	Stress in kPa
0.00	0	0	11.3354	0
6.00	0.2	0.01711	11.3653	2.88251
10.00	0.41	0.07105	11.3969	5.76279
12.00	0.7	0.11711	11.4408	8.15953
14.00	1	0.22895	11.4865	11.98619
16.00	1.3	0.36974	11.5327	15.32063
18.00	1.6	0.575	11.5792	18.63353
20.00	1.9	0.77368	11.6261	21.45699
24.00	2.1	1.09737	11.5636	24.23923
27.00	2.3	1.39342	11.5948	26.53541
30.00	2.5	1.62737	11.6261	28.94523

33.00	2.7	1.96763	11.6575	31.55975
36.00	2.9	2.30789	11.6892	34.66847
39.00	3.1	2.71974	11.7210	36.3507
42.00	3.3	3.18763	11.7529	38.45367
45.00	3.5	3.68474	11.7851	40.70031
48.00	3.7	4.19842	11.8174	41.88629
51.00	4.1	4.65789	11.8499	42.56732
54.00	4.3	5.34474	11.8826	43.24322
52.00	4.5	5.82211	11.9155	43.50425
50.00	4.7	6.30526	11.9485	42.87795
48.00	4.9	6.70428	11.9818	42.3205
48.00	5.3	7.10526	12.0152	41.90341
48.00	5.5	7.50428	12.0488	41.10003

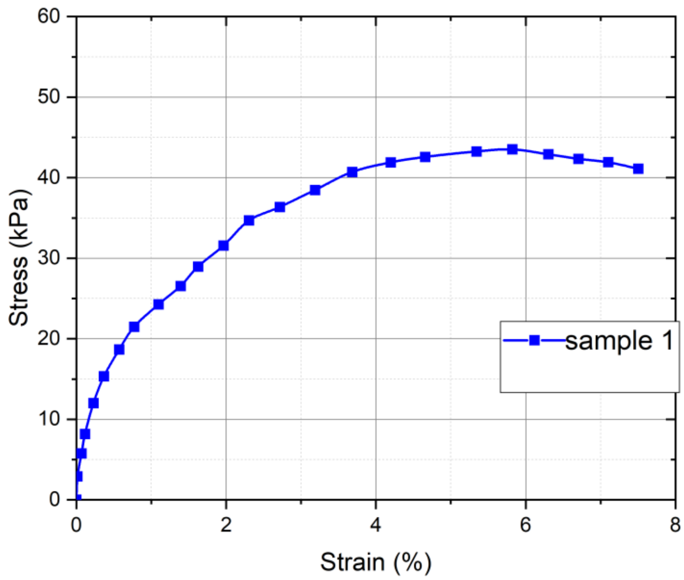


Figure B.1: Stress- strain curve from ucs test.

One-dimensional Consolidation (Oedometer) test laboratory results

Table B.2: Dial guage readings for both loading and unloading cases for soft clay soil.

Time (min)	Loading					Unloading			
	50 (kPa)	100 (kPa)	200 (kPa)	400 (kPa)	800 (kPa)	400 (kPa)	200 (kPa)	100 (kPa)	50 (kPa)
0.00	0.038	0.446	0.772	1.35	2.208	3.074	2.88	2.643	2.338
0.10	0.040	0.448	0.774	1.398	2.264	3.058	2.836	2.578	2.326
0.25	0.061	0.524	0.798	1.462	2.418	3.034	2.822	2.564	2.318
0.50	0.118	0.548	0.816	1.478	2.43	3.032	2.822	2.564	2.318
1.00	0.162	0.56	0.822	1.496	2.448	3.03	2.82	2.564	2.316
2.00	0.164	0.574	0.868	1.52	2.47	3.028	2.818	2.562	2.316
4.00	0.232	0.582	0.96	1.552	2.52	3.024	2.816	2.56	2.314
8.00	0.286	0.594	0.986	1.596	2.544	3.018	2.812	2.558	2.314
15.00	0.318	0.608	0.992	1.652	2.598	3.01	2.796	2.554	2.31
30.00	0.348	0.628	1.018	1.724	2.676	2.996	2.794	2.548	2.306
60.00	0.370	0.654	1.082	1.822	2.79	2.974	2.776	2.536	2.296
120.00	0.386	0.682	1.188	1.936	2.882	2.942	2.746	2.512	2.274
240.00	0.414	0.72	1.28	2.062	2.924	2.92	2.698	2.47	2.234
480.00	0.438	0.758	1.348	2.196	3.022	2.882	2.646	2.406	2.178
1440.00	0.446	0.772	1.35	2.208	3.074	2.88	2.643	2.338	2.106
2880.00	0.446	0.772			3.074			2.337	

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation Dial reading Representing 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen,ΔH	Cumulative of change height of specimen	Change in Void Ratio [$\Delta e=\Delta H/H_s$]	Void Ratio [$e=e_0-\Delta e$]
50	0.061	0.196	0.33	1.5	19.805	9.9	0.038	0.292	0.292	0.036	1.40

100	0.372	0.551	0.73	13	19.449	9.7	0.446	0.284	0.576	0.070	1.36
200	0.684	0.992	1.3	15	19.008	9.5	0.892	0.408	0.984	0.120	1.32
400	1.334	1.667	2	30	18.333	9.2	1.15	0.85	1.834	0.223	1.21
800	2.110	2.625	3.14	9	17.375	8.7	2.1	1.04	2.874	0.350	1.09
400	3.082	3.039	2.995	5	16.962	8.5	3.074	0.079	2.795	0.340	1.09
200	2.850	2.795	2.74	30	17.205	8.6	2.88	0.14	2.655	0.323	1.11
100	2.592	2.466	2.34	10	17.534	8.8	2.592	0.252	2.403	0.293	1.14
50	2.334	2.217	2.1	9	17.783	8.9	2.338	0.238	2.165	0.264	1.17

Pressure (KPa)	Coefficient of consolidation Cv (cm2/minute)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)
50	*0.13*			3.70
100	*0.01*			2.88
200	*0.01*			2.10
400	*0.01*			2.23
800	*0.02*	**0.33**		1.43
400	*0.03*			0.12
200	*0.00*			0.41
100	*0.02*			1.45
50	*0.02*		0.069	2.71

9% of cement stabilized soil of consolidation test

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation Dial reading Representing 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen,ΔH	Cumulative of change height of specimen	Change in Void Ratio [Δe=ΔH/H_s]	Void Ratio [e=e_0-Δe]

50	0.018	0.033	0.047	10.5	19.968	10.0	0.008	0.039	0.039	0.004	1.30
100	0.280	0.297	0.313	9	19.704	9.9	0.148	0.165	0.204	0.024	1.28
200	0.446	0.531	0.616	0.9	19.469	9.7	0.358	0.258	0.462	0.053	1.25
400	0.646	0.754	0.862	5.6	19.246	9.6	0.538	0.324	0.786	0.091	1.21
800	1.018	1.131	1.244	1	18.869	9.4	0.824	0.42	1.206	0.139	1.17
400	1.146	1.134	1.121	8	18.867	9.4	1.16	0.039	1.167	0.134	1.17
200	1.132	1.111	1.089	0.25	18.890	9.4	1.17	0.081	1.086	0.125	1.18
100	1.072	1.062	1.052	30	18.938	9.5	1.168	0.116	0.97	0.112	1.19
50	1.034	1.029	1.023	30	18.972	9.5	1.122	0.099	0.871	0.100	1.20

Pressure (KPa)	Coefficient of consolidation Cv (cm2/minute)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)
50	*0.02*			12.74
100	*0.02*			1.65
200	*0.21*			1.30
400	*0.03*			0.83
800	*0.18*	**0.22**		0.55
400	*0.02*			0.05
200	*0.70*			0.22
100	*0.01*			0.61
50	*0.01*		0.03	1.04

12% of cement stabilized soil of consolidation test

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation Dial reading Representing 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen, ΔH	Cumulative of change height of specimen	Change in Void Ratio [$\Delta e=\Delta H/H_s$]	Void Ratio [$e=e_0-\Delta e$]
50	0.004	0.020	0.035	3	19.981	10.0	0.006	0.029	0.029	0.003	1.24
100	0.056	0.103	0.149	1.5	19.898	9.9	0.039	0.11	0.139	0.016	1.23
200	0.208	0.251	0.294	1.5	19.749	9.9	0.148	0.146	0.285	0.032	1.21
400	0.466	0.483	0.5	1	19.517	9.8	0.334	0.166	0.451	0.051	1.19
800	0.678	0.807	0.935	2	19.194	9.6	0.623	0.312	0.763	0.085	1.16
400	0.816	0.811	0.8069	50	19.189	9.6	0.84	0.033	0.73	0.082	1.16
200	0.782	0.779	0.776	40	19.221	9.6	0.806	0.03	0.7	0.078	1.16
100	0.755	0.750	0.7442	10.5	19.250	9.6	0.776	0.032	0.668	0.075	1.17
50	0.718	0.715	0.712	10.5	19.285	9.6	0.743	0.031	0.637	0.071	1.17

Pressure (KPa)	Cumulative of change height of specimen	Change in Void Ratio [$\Delta e=\Delta H/H_s$]	Void Ratio [$e=e_0-\Delta e$]	Coefficient of consolidation Cv (cm2/minute)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)
50	0.029	0.003	1.24	*0.07*			18.43
100	0.139	0.016	1.23	*0.13*			1.10
200	0.285	0.032	1.21	*0.13*			0.74
400	0.451	0.051	1.19	*0.19*			0.42
800	0.763	0.085	1.16	*0.09*	**0.12**		0.40
400	0.73	0.082	1.16	*0.00*			0.04
200	0.7	0.078	1.16	*0.00*			0.08
100	0.668	0.075	1.17	*0.02*			0.16
50	0.637	0.071	1.17	*0.02*		0.02	0.32

15% of cement stabilized soil of consolidation test

Pressure (KPa)	Do	Deformation dial reading at 50% consolidation	Deformation Dial reading Representing 100% Primary Consolidation	Time for 50% consolidation	Thickness of specimen at 50% consolidation	Half-thickness of specimen at 50% consolidation	Initial deformation reading	Change in Thickness of Specimen, ΔH	Cumulative of change height of specimen	Change in Void Ratio [$\Delta e=\Delta H/H_s$]	Void Ratio [$e=e_0-\Delta e$]
50	0.068	0.095	0.122	0.5	19.905	10.0	0.008	0.114	0.114	0.013	1.14
100	0.175	0.198	0.22	9	19.803	9.9	0.16	0.06	0.174	0.019	1.136
200	0.324	0.348	0.372	9.5	19.652	9.8	0.299	0.073	0.247	0.028	1.13
400	0.512	0.553	0.594	1	19.447	9.7	0.482	0.112	0.359	0.040	1.12
800	0.780	0.809	0.837	5	19.192	9.6	0.694	0.143	0.502	0.056	1.10
400	0.806	0.800	0.793	10.5	19.201	9.6	0.842	0.049	0.453	0.051	1.11
200	0.748	0.741	0.734	30	19.259	9.6	0.793	0.059	0.394	0.044	1.11
100	0.698	0.690	0.682	60	19.310	9.7	0.734	0.052	0.342	0.038	1.12
50	0.655	0.651	0.647	4	19.349	9.7	0.682	0.035	0.307	0.034	1.12

Pressure (KPa)	Coefficient of consolidation Cv (cm2/minute)	Compression index (CC)	Swelling index (CS)	Coefficient of compressibility (av) 10-4 (m2/kN)
50	*0.39*			27.00
100	*0.02*			0.63
200	*0.02*			0.38
400	*0.19*			0.29
800	*0.04*	**0.07**		0.19
400	*0.02*			0.07
200	*0.01*			0.16
100	*0.00*			0.28
50	*0.05*		0.016	0.37

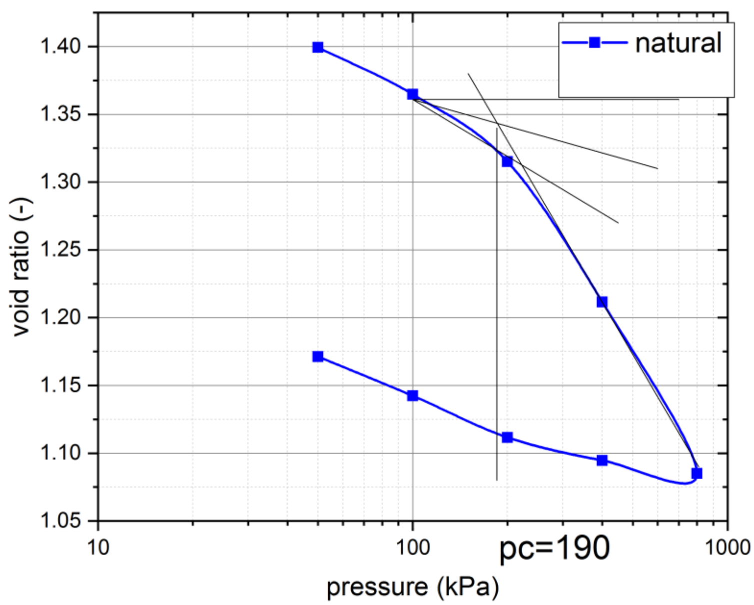


Figure B.2: Void ratio-pressure of natural soil.

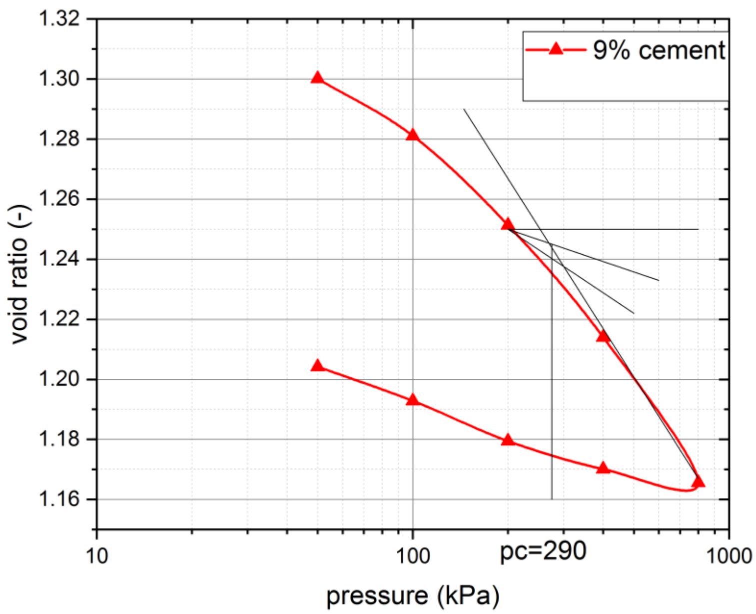


Figure B.3: Void ratio-pressure of 9% cement.

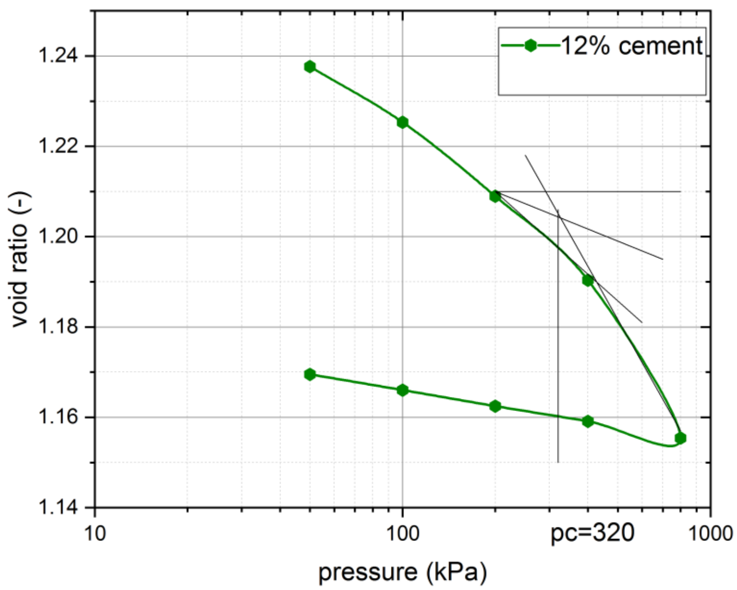


Figure B.4: Void ratio-pressure of 12% cement.

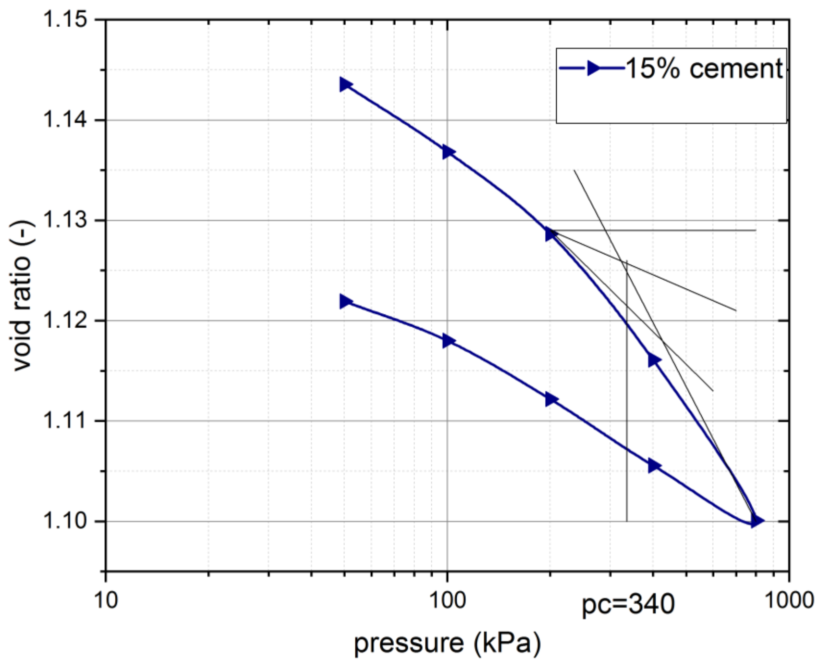


Figure B.5: Void ratio-pressure of 15% cement.

Triaxial test result

Detail calculations for determination of E_{50}^{ref} and E_{ur}^{ref} at σ_3=200 kPa with refference pressure =100 kPa (the atmospheric pressure)

<u>For natural soil</u>

<u>At σ_3=200 kPa</u>

Peak diavotric stress =115 kPa ; 50% diavotric stress = 57.5 kPa; 50% axial strain = 1.48%; therefore;

$E_{50}^{ref} = \frac{57.5}{0.0148}$=3883 kPa = E_{oed}^{ref}

Although $E_{ur}^{ref} = 3E_{50}^{ref}$ is sugessted by plaxis

E_{ur}^{ref} = 3*3883= 11500 kPa

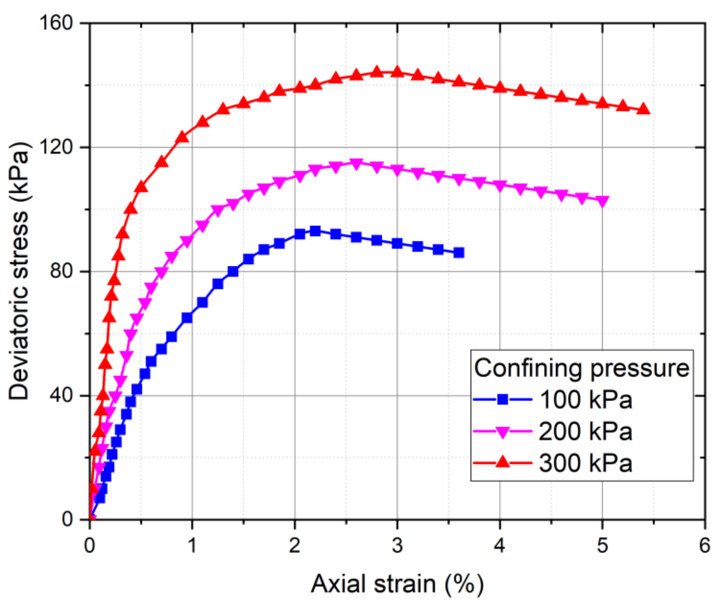


Figure B.6: Triaxial test result of stress- strain curve for natural soil

<u>For 9% cement of soil</u>

<u>At σ_3=200 kPa</u>

Peak diavotric stress = 720 kPa; 50% diavotric stress =360 kPa; 50% axial strain 1.51%; therefore;

$E_{50}^{ref} = \frac{360}{0.0151}$=23684.21 kPa = E_{oed}^{ref}

Although $E_{ur}^{ref} = 3E_{50}^{ref}$ is sugessted by plaxis

E_{ur}^{ref} = 3*23684.21= 71052.63 kPa

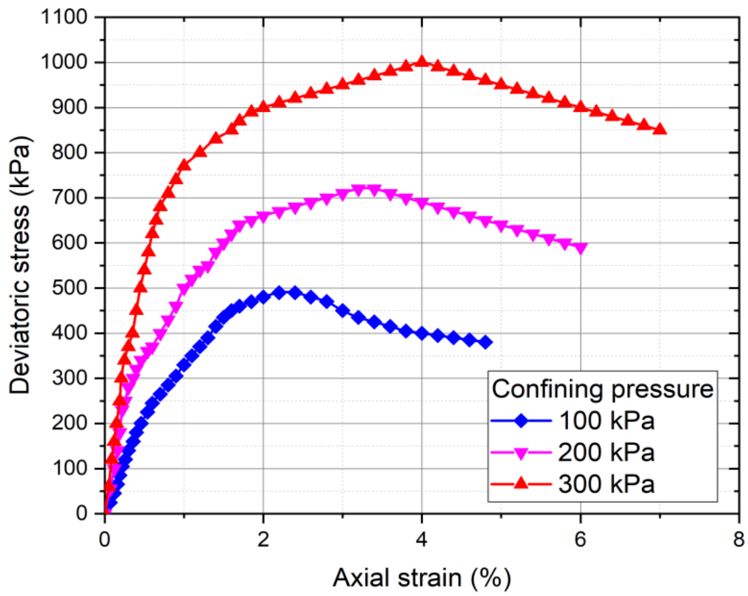


Figure B.7: Triaxial test result of stress- strain curve for 9% cement

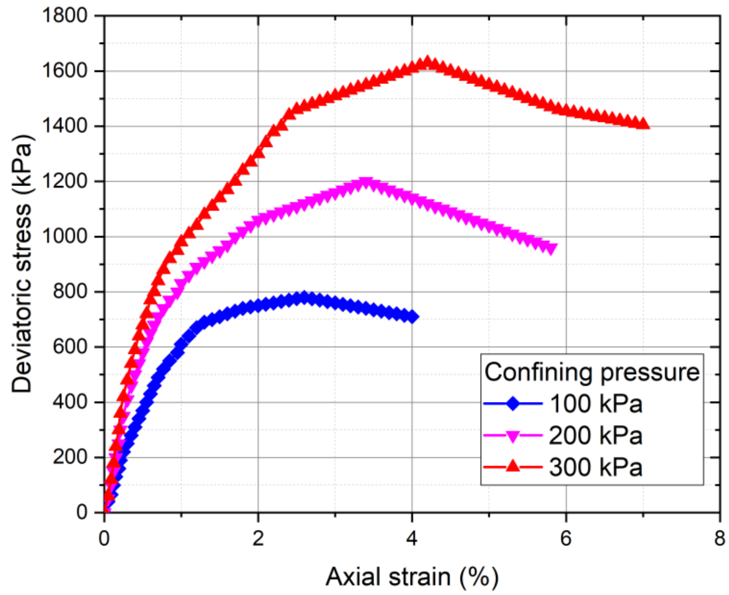


Figure B.8: Triaxial test result of stress- strain curve for 12% cement.

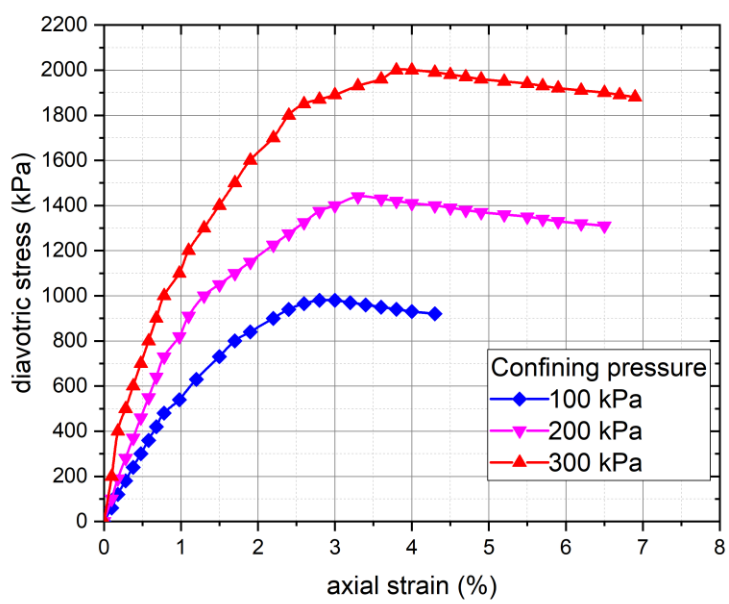


Figure B.9: triaxial test result of stress- strain curve for 15% cement.

APPENDIX C

Finite element result

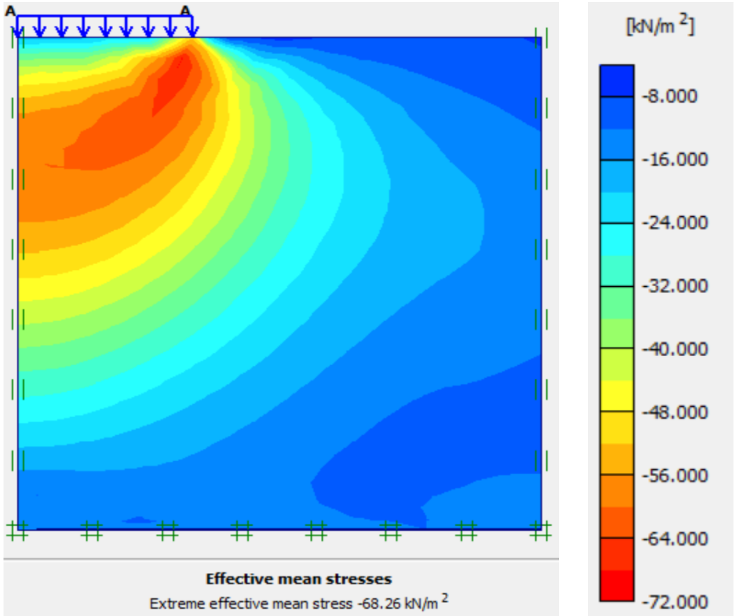


Figure C.1: Natural soil of effective stress.

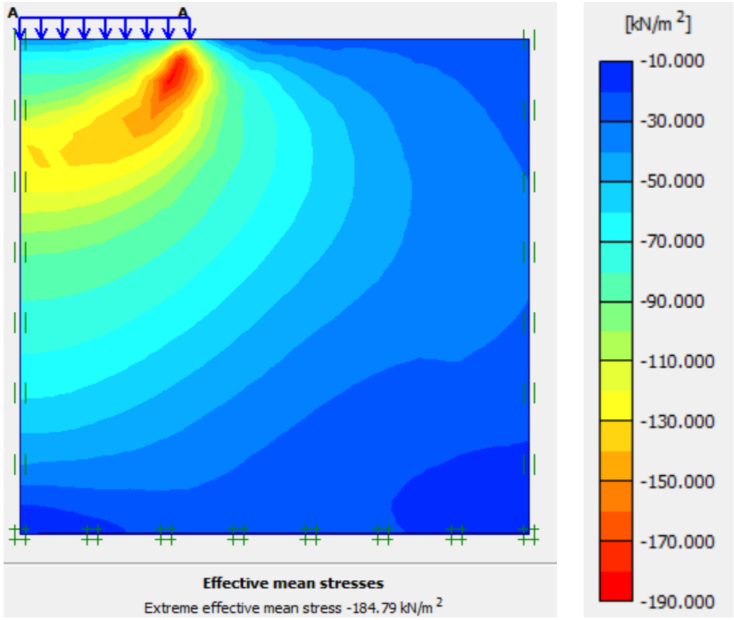


Figure C.2: 9% cement stabilized soil of effective stress.

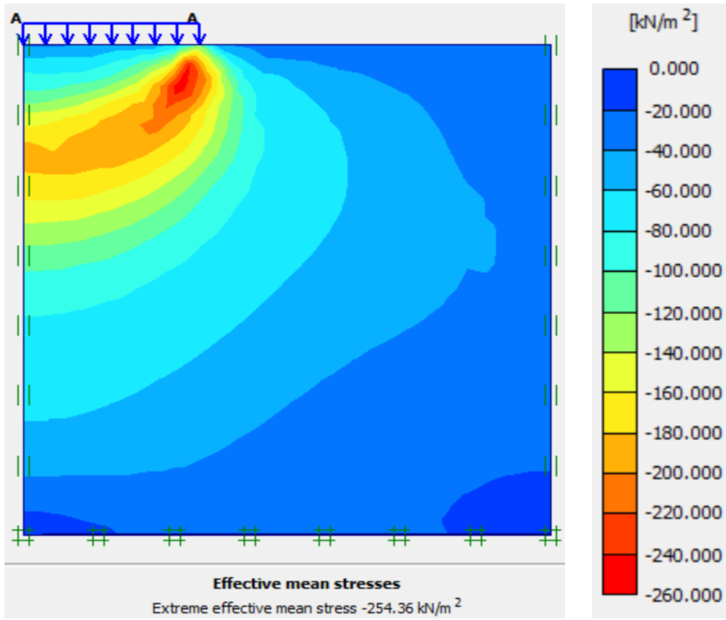


Figure C.3: 12% of cement stabilized soil of effective stress.

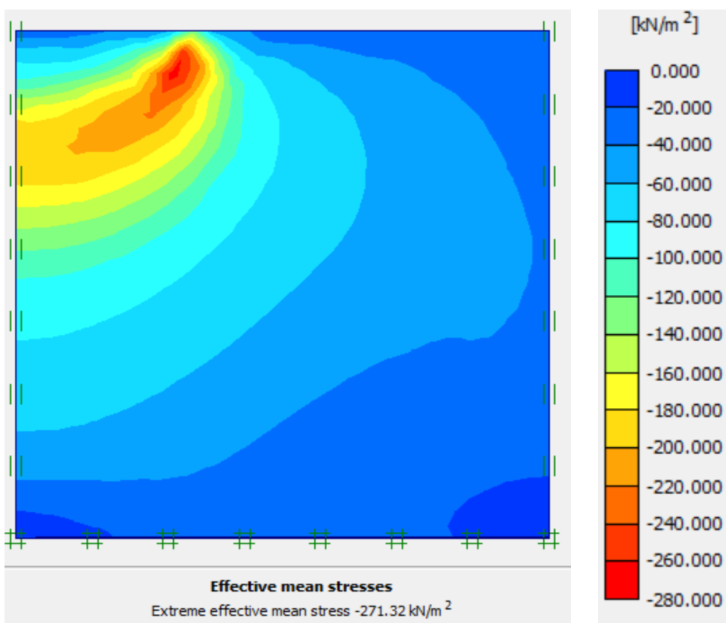
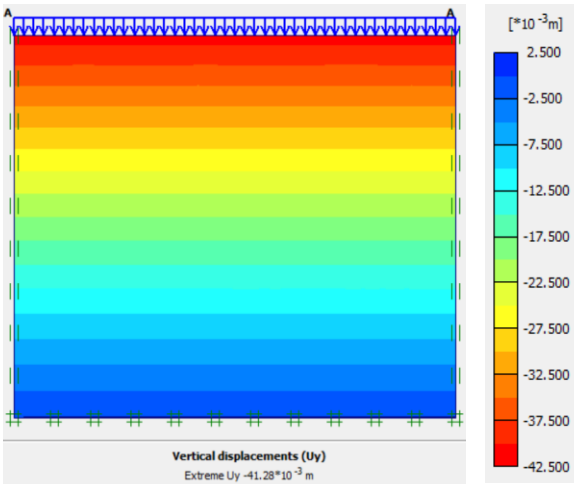
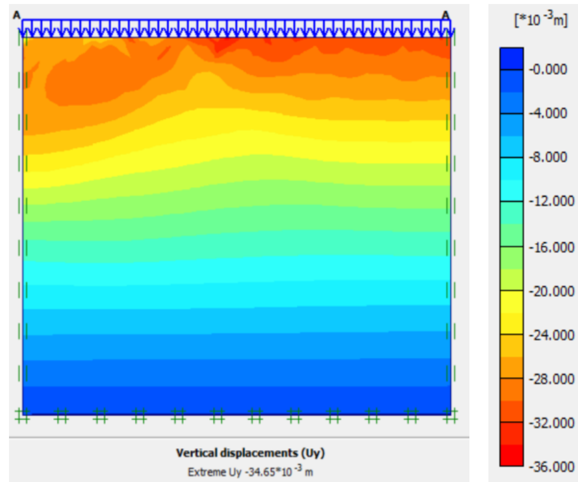


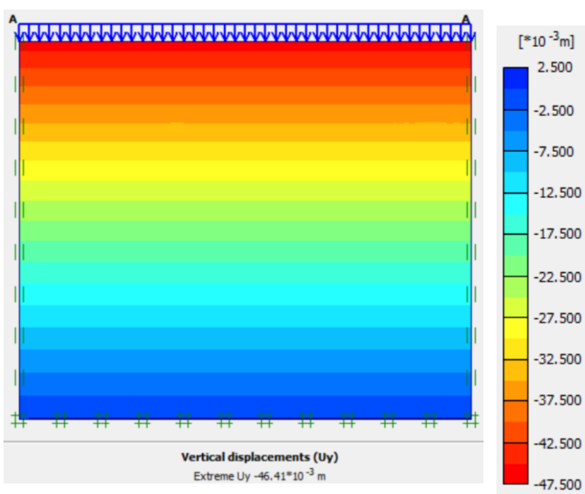
Figure C.4: 15% of cement stabilized soil of effective stress.



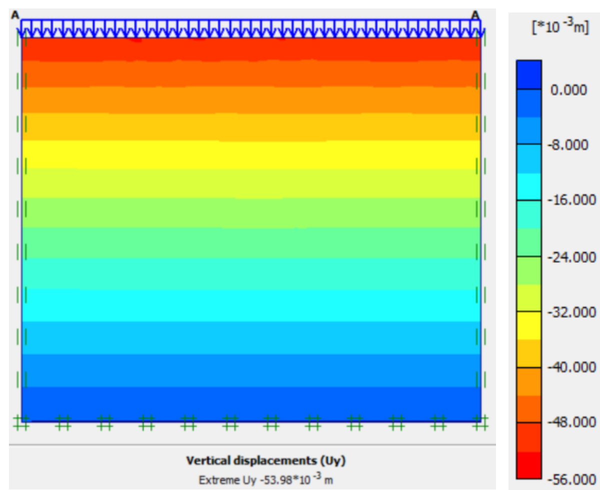
(a)



(b)



(c)



(d)

Figure C.5: vertical displacement which is used for validation.

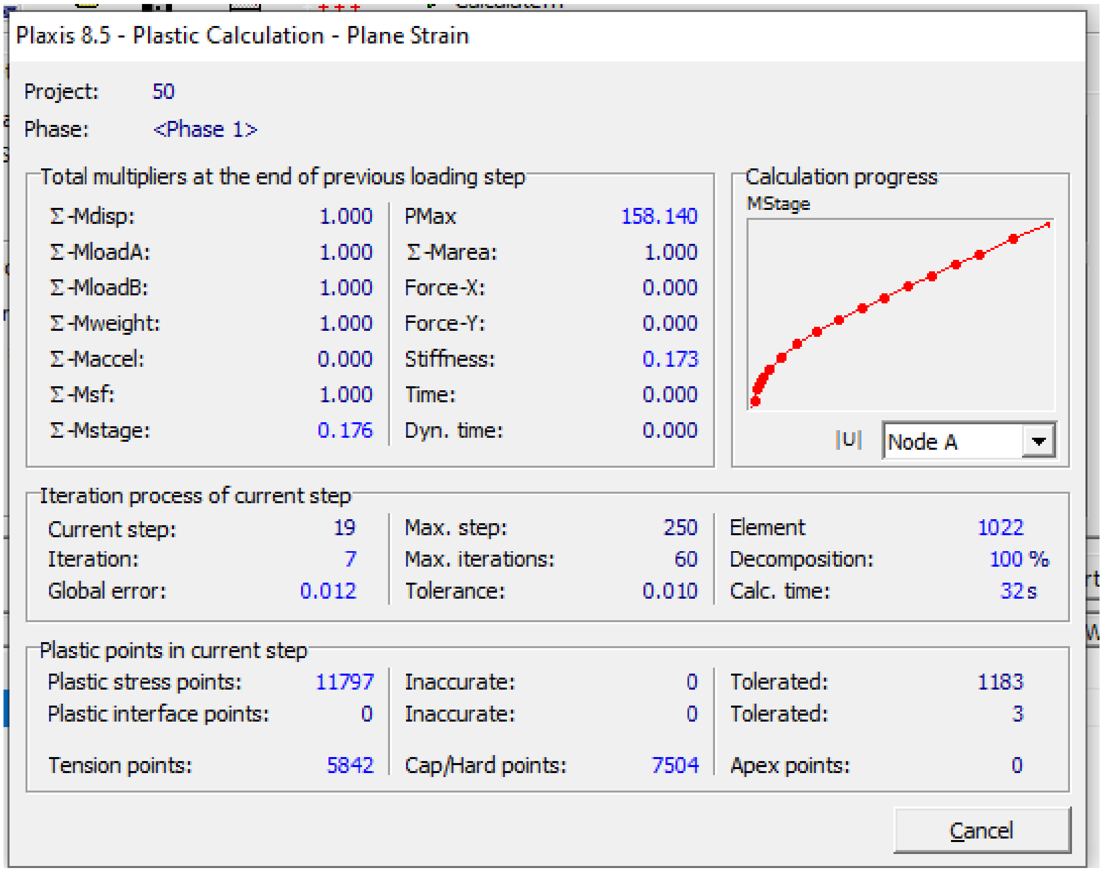


Figure C.6: Calculation progress of plaxis software.

APPENDIX D

Photo captured during laboratory test


POWDER