ÇUKUROVA UNIVERSITY
INSTITUTE OF NATURAL AND APPLIED SCIENCES

MSc THESIS

Mohammed ZAINEL QADER

IMPROVEMENT OF HANDERE CLAY BY USING A MASTER CAST

DEPARTMENT OF GOELOGICAL ENGINEERING

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ÇUKUROVA UNIVERSITY
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We certify that the thesis titled above was reviewed and approved for the award of degree of the Master of Science by the board of jury on 10/07/2018

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ABSTRACT

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This study examines possibility of improving clayey soils in the Handere Formation exposed in the vicinity of Adana (S. Turkey), one of the largest cities in southern Turkey. The Handere Formation, from where the samples for this study are taken, is stratigraphically at the upper most part of the marine sediments of the Adana Basin. The unit is located at the northern part of Adana city. The samples were examined in the geotechnical laboratory to determine the effect of plaster mortar (Master Cast) on the geotechnical properties of the soil and its ability to improve the soil. Atterberg limits, hydrometer, specific gravity, shear box, consolidation, unconfined compressive strength tests were applied on the samples. It has been shown that the master cast used has the potential to improve soil properties geometrically and can be used as a soil stabilizer. The plasticity values of the soils were reduced by master cast addition. Besides, it was determined that the highest maximum dry unit weight and the lowest optimum moisture content were obtained by 15% master cast addition. The soil strength properties were reached to the maximum values in case of 15% master cast added into the mixtures. Besides, it is determined that the coefficient of volume compressibility (Mv) and the pre-consolidation pressure values are ideal when the master cast ratio in the mixtures are 10% and 5% respectively.

Keywords: Adana, Handere, Master Cast.

ÖZ

YÜKSEK LİSANS TEZİ

MASTER CAST MADDESİNİ KULLANARAK HANDERE FORMASYONUN KİLİ ZEMİNLERİNİN İYILEŞTIRILMESI

Mohammed ZAINEL QADER

ÇUKUROVA ÜNİVERSİTESİ
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Bu çalışmanın amacı, Türkiye'nin güneyindeki en büyük şehirlerden biri olan Adana (G. Türkiye) civarında görülen Handere Formasyonu'ndaki killi zeminleri iyileştirme olanaklarını incelemektir. Bu çalışmanın örneklerinin alındığı Handere Formasyonu, stratigrafik olarak, Adana baseninde en üst kısmında yer almaktadır. Birim, Adana şehrinin kuzey kesiminde yer almaktadır. Örnekler, sıva harcının (Master Cast) zeminin jeoteknik özellikleri üzerindeki etkisini ve zemini iyileştirilebilme özelliğini araştırmak amacıyla laboratuvarda incelenmiştir. Örnekler üzerinde Atterberg limitleri, hidrometre, özgül ağırlık, kesme kutusu, konsolidasyon, serbest basınç dayanım testleri yapılmıştır. Zeminin plastisite değerlerinin master cast ilavesi ile azaldığı görülmüştür. Karışımlarda en yüksek maksimum kuru birim hacim ağırlık ve en düşük optimum su içeriği değerlerinin %15 sıva harcı (master cast) ilavesi ile elde edildiği bulunmuştur. Zeminin dayanım özeliklerinin %15 sıva harcı ilave edildiğinde maksimum değerlere ulaştığı belirlenmiştir. Zeminin sıkışma katsayısı (Mv) değerlerinin %10 sıva harcı ilavesi ile ön konsolidasyon basıncı değerlerinin ise %5 sıva harcı eklendiği durumda en ideal değerde olduğu belirlemiştir.

Anahtar Kelimeler: Adana, Handere, Sıva Harcı.

EXTENDED SUMMARY

This study examines possibility of improving clayey soils in the Handere Formation exposed in the vicinity of Adana, one of the largest cities in southern Turkey. The Handere Formation, from which the samples for this study are taken, is stratigraphically at the upper most part of the marine sediments of the Adana Basin. The unit is located at the northern part of Adana city (S. Turkey). The clay used in this study is obtained from the Handere Formation, which is named by Schmidt (1961) and located in Adana Basin. The unit primarily consists of uncolored claystone, pebbly sandstone, sandstone, siltstone, mudstone marl, and includes gypsum lenses and clay levels of various thicknesses in places. Gravel stones seen as large scale trough cross bedded, while fine grains seen as parallel laminated. The thickness of the formation is in the range of 120 to 700 m (Yetiş and Demirkol, 1986).

The samples were examined in the geotechnical laboratory to detect the effect of plaster mortar (Master Cast@) on the geotechnical properties of the soil and its ability to improve the soil. Atterberg limits, hydrometer, specific gravity, shear box, consolidation, unconfined compressive strength tests were performed on the samples. Plaster Mortar (Master cast) is a high performance micro air entraining mortar admixture structured to enhance impermeability, workability, and freeze-thaw resistance in plaster mortars. Structure of the material is dilute solution of surface active substances with organic acid, it has specific gravity 1.8.

Collecting the soil samples from the field and transferring them to the laboratory, the necessary engineering tests were done with the help of the equipment available for use in the laboratory for the purpose of identifying the properties of the soil, and finding out the effects of the master cast used on the properties of the soil, along with the ability of master cast to improve the soil. Samples were prepared in 4 different cases pure soil and soil with Master Cast, 5%, 10%, 15%. Standard Proctor experiments were performed. Optimum Water Content is re-compacted in Proctor

mold. Unconfined compressive strength, direct shear, and Consolidation tests were carried out with the values took from Proctor mold. The American Society for Testing Materials (ASTM) procedures were employed in material analysis.

And according to the results, the plasticity limit for the pure soil was 24.05%, liquid limit was 49.7% and plasticity index was 25.05%. Soil plasticity increased from 24.05% in pure soil to 29.21% at 15% master cast. The soil liquid limit of 49.7% in pure soil decreased to 45% at 15% master cast. The plasticity index decreased from 25.05% in pure soil to 15.79% at 15 master cast %.

The Hydrometer and sieve analyses indicated the components of the pure soil as follows: clay 56,41 %, silt 37,49%, sand 6.1%. The specific gravity for soil was 2.718.

According to compaction results, water content for the pure soil was 24.2% and the dry density was 1.455 g/cm^3. According to compaction results water content for plaster mortar treated soil was 21.4% and dry density was 1.54 g/cm^3 at 15%. Optimum moisture content of 24.2% for pure soil decreased to 21.4% for 15% material mixture. The dry soil density of 1.455 g/cm^3 in pure soil increased to 1.54 g/cm^3 at 15% of material mixture.

The unconfined compressive strength of 10.27 kg/cm ^{2} in pure soil increased to 23.03 kg/cm^2 with 15% material. The shear strength of 1.09 kg/cm^2 for the pure soil increased to 3.35 kg/cm^2 at 15% material mixture.

From the consolidation test, the percentage of soil voids decreased from 0.8723 in pure soil to 0.6878 at 15% material. The (Mv) value changed with addition of material and decreased under higher loads and with high percentages of material clearly. Finally the results show that the material can be used to improve soils. The plasticity values of the soils were reduced by master cast addition. Besides, it was determined that the highest maximum dry unit weight and the lowest optimum moisture content were obtained by 15% master cast addition. The soil strength properties were reached to the maximum values in case of 15% master cast added into the mixtures. Besides, it is determined that the coefficient of volume compressibility (Mv) and the pre-consolidation pressure values are ideal when the master cast ratio in the mixtures are 10% and 5% respectively.

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CONTENTS **PAGE**

LIST OF FIGURES **PAGE**

LIST OF TABLES **PAGE**

LIST OF SYMBOLS

SYMBOLS	Description
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing Materials
LL	Liquid Limits
PL	Plasticity limit
PI	Plasticity index
SBR	Styrene butadiene rubber
P	Percentage of soil in suspension
GS	Specific gravity of soil particles
V	Volume os suspension
MS	Dry mass of soil
RC	Corrected hydrometer reading
D	Diameter of soil particle
L	This distance is known as effective depth. Distance from the surface of the suspension to the level at which the density of the suspension is being measured
T	Time at which the hydrometer reading was taken
K	Constant depending on temperature and specific gravity of the soil
Gt	Specific gravity
PS	The density of the soil solids
Pwt	The density of water at the test temperature
Mpwt	Mass of the pycnometer and water at the test temperature
MS	Mass of the oven-dried soil solids
Mpwst	Mass of water, pycnometer and soil solids at the test temperature
CV	Coefficient of Consolidation
T1	A dimensionless time factor for 90 % consolidation

H	Length of the drainage path
ho	Original height
Δh	The settlement at 90% consolidation
t	Time corresponding to the particular degree of consolidation (s or min) at t90
Ø	Angle of internal friction
C	Cohesion
τ	Nominal shear stress
F	Shear force
Ao	Initial area of the sample
dr	Displacement rate
df	Estimated horizontal displacement at failure
tf	Total estimate elapsed time to failure
t50	Time required for the sample to achieve 50 percent consolidation under the specified normal stress
t90	Time required for the sample to achieve 90 percent consolidation under the specified normal stress
qu	Unconfined compressive stress
P	Given applied load
A	Corrected area of the sample
Ao	Initial area of the sample
ϵ	Axial strain for the given load
ΔL	Length change of sample as read from deformation indicator
Lo	Initial length of sample
K	Coefficient of permeability
a	Area of the burette
LC	Length of soil column
AC	Area of the soil column
hS	Initial height of water

hF	Final height of water
t1	Time required to get head drop of Δh

1. INTRODUCTION

The soil, where the buildings build in and human beings live on, is one of the most impressive natural phenomena on earth. Soils are formed as a result of the disintegration of the rocks, because of either physical or chemical factors (Rashed, 2016). To use this material, one must combine engineering, environmental, geology, chemistry and physics. Having different physical properties, scientific researches need to study soils in order to identify their properties and the behavior to reduce/elucidate problems (Teymur, 2013). Naturally, the soils have different types, such as soft and weak with low bearing capacity, and medium stiff with good bearing capacity, and very stiff with high bearing capacity.

When building structures, tunnels, bridges, it is required to study in detail to find out the quality of the soil and extent of cohesion. Some soils do not have the ability to withstand the loads as a result of the weak properties, so these soils need to improve their properties in order to be able to bear buildings and structure built above it. Many material are used to improve soils such as lime, cement, natural fibers, bitumen, fly ash, and some chemical materials like magnesium chloride.

Human beings use many convenience methods to improve soil and modify their engineering properties. Good knowledge of the properties of soil improvement techniques has developed especially during the last 80 years that includes compaction, mixing, injection etc.

In nature, not all soils are completely stable in order to provide adequate and necessary support for foundations and buildings that built over it. Soil improvement techniques are used frequently in countries that are exposed to earthquakes or shortage of lands. So these countries try to improve their soils. The soil improvement techniques are used to reduce the risks and improve the weak soil, therefore, they evolved over the years, mostly by right and wrong attempts. One of matters that should be take into consideration is the cost, technical feasibility, environmental effects.

After identifying the area where the soil is needed to be improved and to reduce the risks that are faced when building, one of the soil improvement techniques is used for the purpose of achieving the following points.

- Reduce settlement.
- Reduce swelling and shrinkage.
- Increase shear strength.
- Reduce permeability.
- Reduce compressibility.
- Increase bearing capacity.
- Increase safety factor against possible slide.
- Reduce liquefaction risk (seismic areas).

In this study, the clayey soils in the Handere Formation exposed in the vicinity of Adana, one of the largest cities in southern Turkey were examined. The study area is shown in Fig 1.1. The Handere formation, is stratigraphically at the upper most part of the marine sediments of the Adana Basin. The unit is outcropped at the northern part of the Adana city (S Turkey) Fig 1.2.

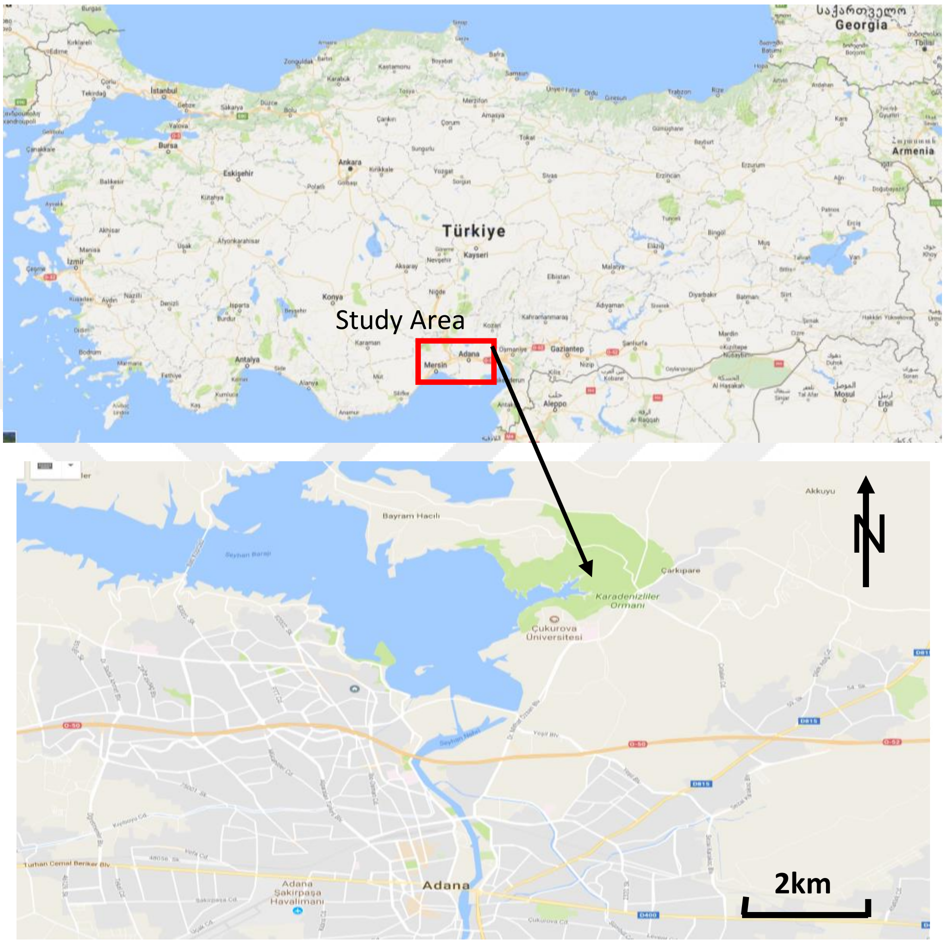

Figure 1.1. Location map of study area

The Adana basin is consist of Oligocene to Recent marine and terrestrial sediments and limestones. The basin, from bottom to top, consists of Oligocene aged Gildirli formation, formed in a river setting (Fig 1.2). The Early Miocene aged Karaisalı formation, which is formed in a shelf setting, overlays the Gildirli formation (Faranda et al., 2013) (Fig 1.2).

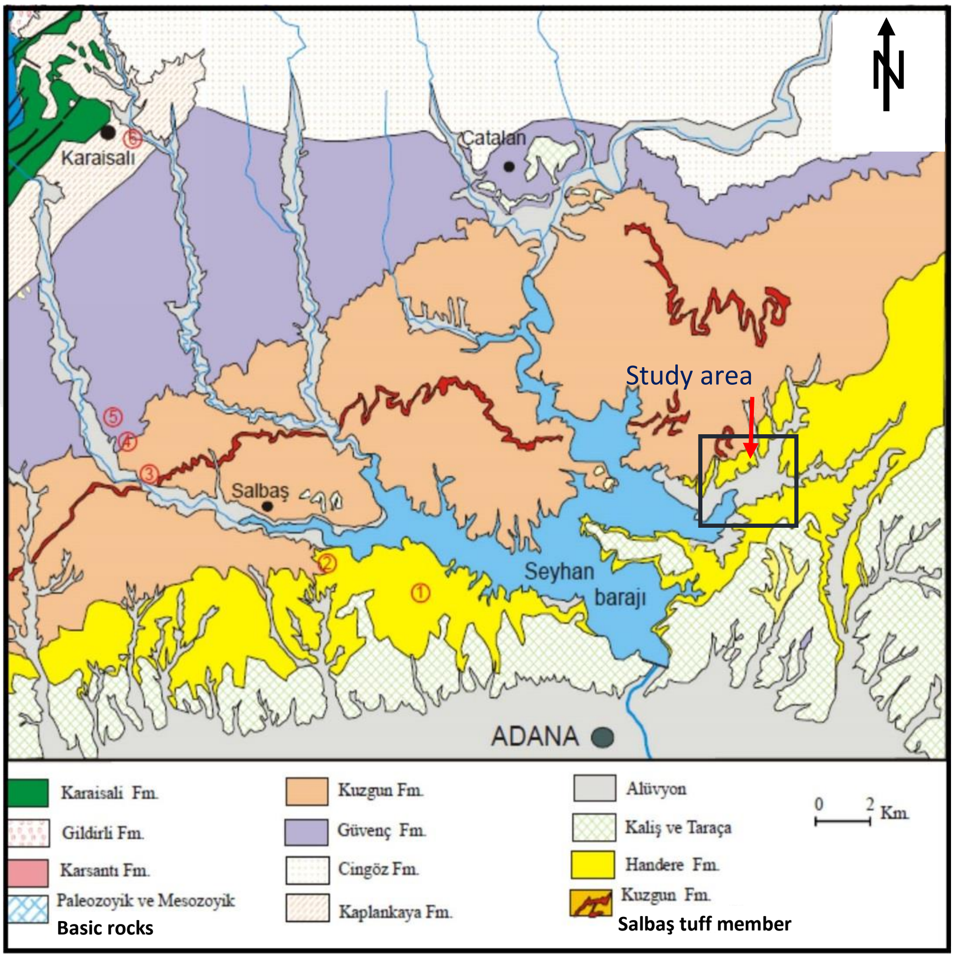

Figure 1.2. Geological map of the Adana Basin, [modified after Yetiş& Dermikol (1986), Unlügenç (1993), and Yetiş et al.(1995)].

The Handere formation is conventionally known as resting canorously on the marine deposits of the Kuzgun Formation (Gürbüz and Kelling 1993; Yetiş et al. 1995; Nazik 2004; Darbaş ve Nazik 2010; Faranda et al., 2013). Recent studies show that the Handere formation has an unconformable contact with the Kuzgun formation (Cosentino et al., 2010a, b; Cipollari et al., 2012). The conventionally accepted stratigraphy of the unit refers the gypsum member of the Handere formation is located at the top of formation, however recent studies indicate that these gypsum levels are located at the bottom of the unit and has a contact with the Kuzgun formation (Cosentino et al., 2010a,b; Cipollari et al., 2012).

The Handere formation is exposed in a large area at the northern part of the Adana city a promising candidate for new building structures. Therefore, the geotechnical properties of the unit have been investigated in detail. The clayey soils and the gypsum levels within the Handere formation are not eligible for building structure, however currently many buildings and infrastructures were already built. The current structures and the next generation building and infrastructures were under different types of risk if these clayey soils and gypsum levels were not treated.

In this study, the soil samples were taken from different parts of the Handere Formation and the change in the same samples after adding (Master Cast©) have been investigated. In the first stage, the geotechnical properties of the natural samples were determined to understand the possible risks. In the second stage, master cast has been added into the samples to improve the geotechnical properties. The input ratio of the cement liquid were tested in three different percentages; 5% 10% 15%. If the quality increase is not as accepted, the ratio of the cement liquid is increased gradually, until achieving an enough soil quality improvement.

The aim of this study is possibility of improving soil in the selected area in the city of Adana using Master cast by testing the engineering properties of the soil before and after adding the material and evaluating its effects on the soil properties through laboratory tests.

1.1. Soil Stabilization Methods

There are many traditional ways of stabilizing soil that is used in buildings and constructions, such as roads and yards. A kind of soil stabilization method was used for the first time to build a road in the United States in Johnsonville, Colorado in 1935 (Lambe, 1951).

This technique was also used during World War II throughout the years 1939-1945 (Woods, 1960; Das, 2004). The purposes of using different soil improvement methods are:

- Increase soil resistance,
- Reduce soil permeability,
- Reduce swelling,
- Increase durability,
- Reduce soil compressibility,

There are various methods used to achieve these targets, including chemical stabilization and physical fixation, and the preference of any method of soil stabilization depends on the soil itself (Ingles and Matcalf, 1972). There are different methods of soil stabilization, including:

- Mechanical stabilization
- Cement stabilization
- Bitumen stabilization
- Fly ash stabilization
- Jet grouting

There are other methods used to stabilize soil, such as the use of electricity or freezing soil used in cold areas.

1.1.1. Mechanical Stabilization or Compaction

Mechanical compaction is one of the traditional compaction methods, which has long been known, and one of the most economical methods regarding the construction of various facilities, such as dams, roads and airports. The process of compacting the soil is a process of increasing the density of the soil by expelling the air from its gaps, which leads to the reduction of the size of these gaps (Lambe, 1951).

In the mechanical stabilization of soft soils, soil strength and resistance is achieved by cohesion in soil (clay and silt), either in coarse soil such as sand, which have the advantages of internal friction forces.

1.1.1.a. Laboratory Compaction

When some amount of water added to dry soil, soil grains get coated with a thin layer of water. By increasing the amount of water added to the soil, the thickness of the water membrane that encapsulates the soil granules increases. This process is called lubrication, and it has a significant impact on the compaction soil soft grain. The addition of a limited amount of water to dry soil is an easy method and makes it compact. To a certain degree of humidity, the water expels the air from the gaps among soil granules and replaces it (Lambe, 1951).

The very first person who applied this method was the American engineer R.R. Proctor (1933). By referring to R.R. Proctor, therefore, this standard method of the test process called the Proctor standard test, ASTM D-698. This test is done by laying the soil model into a standard cylindrical mold and dropping a standard hammer, which is free of soil, from a limited height.

This test was timely, but after the developments in soil compacting equipment in the field and to increase capacity, the soil model has increased the weight of the hammer and its free-fall distance. This new test is called Modified Proctor Test, (ASTM D-698), (Lambe, 1951).

1.1.1. b. Field Compaction

According to the laboratory information mentioned previously, the site's requirement is to obtain 90-95% of the maximum dry density (MDD) extracted in the laboratory examination, where the soil is placed in the site in the form of layers and anchored by field surveying machines that are divided into the following types:

- Smooth wheel rollers,
- Rubber tired rollers,
- Sheep foot rollers figure (2.1)

There are other types of field surveying machines used for different purposes as well. After calculating the MDD of laboratory tests, and the field density of field tests, excretion percentage is extracted from the following equation:

Figure 1.3. Field compaction

$$\text{Compaction ratio (\%)} = \frac{\text{Field Density}}{\text{Labrotory Density}} \qquad \text{(Eq1.1)}$$

1.1.1.c. Vibratory Compaction

Conventional surface modulation methods result in a shallow depth of no more than 2 m, and there are two ways for achieving large depth probes.

A. Vibroflotation

It is a technique or modulation method used for non-cohesive soil or granular soils, and is more effective for loose sand, especially those underground water level (Brown, 1977).

B. Pounding

First used in the United States, this method aims the increment of the density of the incoherent soil. This method is the process of dropping a specific weight from a specific height to obtain the density required for the soil disintegrated. It has been used in many sites successfully. The degree of vibration may affect neighboring sites when using this technique (Lakas, 1980).

1.2. Chemical stabilization

Chemical stabilization is one of the traditional methods used to stabilize the soil, and as discussed below, it has several types depending on the type of the additive used:

1.2.1. Lime Stabilization

Soil improvement using lime depends mainly on the interaction between the soil and lime components, where they form a homogeneous mixture, and as a result, improve engineering properties (Ingles and Matcalf, 1972) such as:

- Increasing shear strength,
- Increasing weather resistance,

- Decreasing the amount of swelling and plasticity index,
- Decreasing the water content and increasing compaction properties.

The lime material used in this process is of three types:

- Quick Lime (Cao)
- Hydrated lime $Ca(OH)_2$
- Lime slurry

Hydrated lime is the most widely used soil stabilizer because quick lime environmentally hazardous and difficult to be dealt with, and lime surly is not recommended for its economic cost.

1.2.1. a. Soil and Lime Reaction

- Ion Exchange and Flocculation - agglomeration

Laboratory tests indicate that the inflorescences interact with the medium-soft granules of the soil and the interaction results in a decrease in plasticity, and an increase in workability and resistance (Little, 1995). In general, the ion exchange process takes place, when blending lime with the soil in the presence of water directly, between the ions of the lyotropic series ($Na^{+}<K^{+}<<Mg^{+2}<Ca^{+2}$). High-ions replace low-ions or large ions replace small ones, which are equivalent (Little, 1999). This chemical reaction causes a change in soil tissue and soil properties due to the cation exchange. As the free calcium ions in the lime exchange are adsorbed by the cations of the clay, leading to a decrease in the thickness of the water layer spread, the granules of the soil approach each other, and the exchange of ions leads to:

- Increase in operability,
- Improvements in compression,
- Decrease in swelling and plasticity (Mallela et al., 2004; Littile, 1999).
- Increase internal friction and shear resistance,
- Decrease in water volume in soil.

- **Pozzolanic Reaction**

Lime being connected with the soil, the pH of the pore water is increased in soil granules, reaching the level of 4.12, which corresponds to the ideal value of added lime. And this also leads to the melting of amorphous silica and alumina in the soil, thus becoming ready to interact with the calcium ion from the naphtha to form water calcium silicate or water calcium aluminate (Eades and Grim, 1960).

The following equations show how this interaction occurs (Mallela et al., 2004), and how it becomes a bonding material:

$(OH)_2 \longrightarrow Ca^{+2} + 2(OH)^{-1}$

$Ca^{+2} + OH^{-1} + Sio_2$ (soluble clay silica) $\longrightarrow$ Calcium_Silicate_Hydrate (CSH)

$Ca^{+2} + OH^{-1} + Al_2O_3$ (soluble clay alumina) $\longrightarrow$ Calcium_Aluminates_ Hydrate (CAH)

The persistence of these reactions depends on the characteristics of the natural soil. The resistance increases and depends on alumina and silica dissolved from the clay itself, and it has been shown that the lime is affected by the montmorillonite of kaolinite minerals. This is due to the large ion exchange capacity of montmorillonite mineral.

- **Lime carbonation**

This process yields the reaction of calcium with CO_2 in the atmosphere with the presence of water and the reaction results in insoluble calcium carbonate ($CaCO_3$).

Carbonation can be minimized in a laboratory during maturity, as the model is wrapped and isolated from the air. In the practical field, however, the potential impact of this process is greater than the potential benefit of soil stabilization. Some factors affecting soil lime stabilization.

There are many factors that influence the completion and quality of the installation and these factors are:

- **pH factor**

Soil with more than 7.0 pH value is a soil with a potentially good interaction with lime (Thompson, 1966).

- **Quantity of mud and type of clay minerals**

The amount of clay present has a clear effect on soil-lime stabilization, since it contains silica and alumina. However, one of the clay species, the soil containing montmorillonite, has a greater effect in the chemical reaction than the soil containing kaolinite.

- **Organic Matter Content**

The fixation is affected by the soil containing 1-2% organic matter, since organic materials prevent resistance from the pozzolanic reaction. Organic matter obstruct the melting of silica and alumina in the soil, or the bonding of silica and alumina with lime added to the soil (Sabbagh, 1973).

- **Curing**

The heat of ripening and ripening time has a significant effect on soil fixation. It is observed that, the resistance increases by increasing temperature of ripening and ripening time (Das, 2004). The rate of increase in resistance also depends on a number of factors including:

- Type of soil,
- Type of lime,
- Compaction type.

1.2.1. b. Properties of Soil Stabilized by Lime

One can observe two phases as lime is being added to the soft-grained soil. In the first phase, some properties of the soil, such as plasticity, workability, and volumetric change, are improved. In the second phase (Diamond and Kinitter, 1965), properties of soil stabilized by lime are:

Plastic properties: Adding lime to the soil induces the improvement or change of plasticity properties, decrease in plasticity index and liquid limit, increment of the shrinkage limit and plastic limit as explained by (Das, 2004), and then an overall enhancement in soil properties.

Granular gradient: Adding lime to the mud soil induces a change in the gradient due to the phenomenon of flocculation, changing the soil into sand and silt (Eades and Grim, 1966).

Volume change: It was found that adding lime to the soil reduces swelling and shrinkage in the soil (Diamond and Knitter, 1965).

Compaction properties: Soil properties are affected by maximum dry density and optimum moisture content by adding lime to the soil (Diamond and Knitter, 1965).

Swelling: Swelling is the increase in the volume of soil when exposed to water. Lime reduces bloating in the soil through the exchange of ion between the cation in the soil and the calcium of the lime (Lambe, 1962).

Wetting and Drying: The amount of lime and soil affect the results of the wetting and drying test. Similarly, the late maturation time increases the value of resistance in the wetting and drying test (Meteous, 1964).

Strength: Resistance generally increases if the soil granules are smooth and the lime content is perfect. The increase in resistance varies conditional to the type of soil, the quantity of lime added, the moisture content, and the time and heat of ripening (Meteous, 1964).

Compaction Effort: Resistance increases in case of the modified Proctor is used instead of standard Proctor by the ages of 7 or 28 days respectively (Meteous, 1964).

1.2.1. c. Mixture Design and Strength Characteristics

When using lime to improve soil, the goal of the mixture design is to create the ideal lime content, which gives resistance to the soil. On the other hand, it is known that the requirements for increasing the resistance vary from one project to another. In general, there are many methods or procedures employed in this type of installation as discussed below:

- The Thompson procedure and the Eades and Gieman procedure

Calculate optimum lime content by measuring the pH ratio of the lime-soil mixture after preparing the mixture for approximately one hour. The lowest proportion of the lime used to give a pH value of 12.4. This ratio is the ratio of optimum lime content. In the Illinois soil, it was found that the lime content of the pH test in this way gives the same content as the lime, which gives the highest compression resistance (Thompson, 1966).

- Louisiana procedure

In this procedure, the design of the ratio of the lime depends on the least optimum lime content. It gives the compressive strength to 50 psi for sub-base soil and 110 psi for base soil, with the condition that the models are 6-inch-diameter by 8-inch-high in the unconfined compressive strength (Thompson, 1966).

- Illinois procedure

The optimum lime content is the percentage that gives the highest resistance required. Hence, additional increases in the ratio of lime do not give additional resistance to the mixture over the optimum ratio. The soil-lime mixture gives resistance to 100 psi for sub-base and 150 psi for soil base, when samples in the test are 2-inch-diameter and 4-inch-high (Thompson, 1966).

- Texas procedure

In this method, the percentage is chosen based on the plasticity index and the passing percentage of sieve number 40 by a chart as in Figure 2.2. And for optimum lime content the chart (Geiman, 2005).

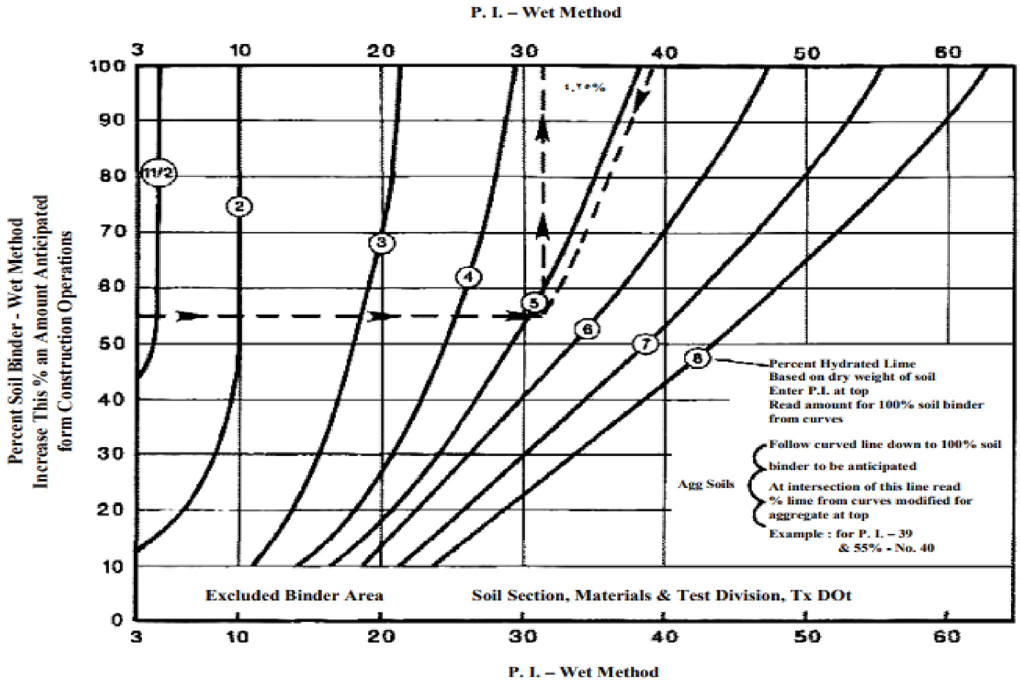

Figure 1.4. The method of choosing optimal lime content in road engineering in texas (U.S.A) (Geiman, 2005).

1.2.2. Soil Improvement Using Cement

Soil improvement by using cement is one of the traditional methods. This method is practiced by mixing Portland cement with dust and a specific amount of water, which maintains high unit weight and protects the mixture against moisture loss during processing (Woods, 1960).

The improvement of soil by Portland cement is widely used in some constructions, such as roads and airports. Because of the composition of strong cement compounds, the characteristics of this material is different from the characteristics of soil or ordinary cement (Herzong, 1963).

1.2.2. a. Engineering Properties of Cement and Soil

- Atterberg limits

The Atterberg limits is used to classify cohesive and non-cohesive soil. The plasticity index is very important to illustrate the behavior of soft materials, especially clay. Cement addition reduces the shrinkage and swelling of the soil. Croft (1967) examined many soils and noted the effectiveness of adding cement to mud with different maturation periods on some soil properties. The reaction of cement with clay reduces the liquid limit and the plastic limit and increases more than 40%, and the limit of liquidity is limited to less than 40 (Woods, 1960).

- Unconfined compressive strength

It was used extensively as an indicator of the interactions in the soil-water-cement mix, as well as the setting time and rates of hardening (Woods, 1960). In general, the resistance increases with the cement content, and varies with the soil type used. Similarly, increasing resistance gradually with increased ripening, and heat of ripening, excess drying, and immersion reduce resistance, especially in mud soils (Ingles and Matcalf, 1972).

1.2.2. b. Soil-Cement Reaction

Portland cement is generally composed of the following main compounds:

($3CaO.SiO_2$) Tri-calcium silicate (C_3S)
($2CaO.SiO_2$) Di-calcium silicate (C_2S)
($3CaO.Al_2O_3$) Tri-calcium aluminate (C_3A)
($4CaO.Al_2O_3$-Fe_2O_3) Tetracalcium aluminoferrite (C_4AF)

Composition with water and decomposition under the following equations (Skoter, 1963; Al Naqshabandi, 1990).

$3CaO.SiO \xrightarrow{H_2O} 2CaO.Sio_2.XH_2O + Ca(OH)_2$

Moderate (Microcrystalline)

$2CaO.SiO_2 \xrightarrow{H_2O} 2CaO.SiO_2.XH_2O$

Slow (Microcrystalline)

$3CaO.Al_2\,O_3 \xrightarrow{H_2O} 3CaO.Al_2O_3\,.6H_2O$

Immediate Crystalline

$3CaO.Al_2O_3 \xrightarrow{H_2O} 2CaO.Al_2O_3.3CaSO4.3H_2O$

Moderate Crystalline

$4CaO.Al_2O_3.Fe_2O_3 \xrightarrow{H_2O} 3CaO.Al_2O_3.6H_2O$+undetermined

1.2.3. Bituminous Stabilization

The soil is bonded with a high-resistance mixture because it binds the soil granules with a bonding material. This mixture shows impermeable behavior throughout this layer, binding the soil components together to give good cohesion to the uncooperative granule (Punmia et al., 1994).

1.2.3. a. Types of Bituminous Materials

Bituminous is a semi-solid or solid bond of black or brown color, and when heated, it becomes liquid and dissolves in carbon tetrachloride (CCL) and carbon dioxide (CS2).

- Natural asphalt

This appears in the nature in lakes or in rocks or veins. It can be found in the nature individually or mixed with metal materials, hence its strength varies from steel to weak.

- Manufactured Asphalt

This type of kerosene is produced by filtering crude oil. It is divided into three types (Young et al.1998): This type of asphalt comes with specific quality and strength and can be used directly. For tiling work, it is used hotly.

- Asphalt Cutback

This type of asphalt is obtained by mixing asphalt cement with hydrocarbon solvents, such as kerosene. Kerosene and fat are used in the fixation soil (Young et al., 1998).

- Water action proofing

It is more important and gets either blocked by gaps or narrowed by channels flow along the soil surface treated, which is called the plug theory, or by packaging the soil granules, which is called Member Theory.

- Cementing action

Here the resistance increases with a continuous casing of coriander material around the soil granules to give it a cohesive resistance (Flatherty, 1983).

1.2.4. Fly Ash Stabilization

Fly ash is a method of improving soils, which is used as an alternative to cement and lime. Fly ash is obtained through burnt coal, as in the United States of America, and it is obtained from the combustion of coal used for electric power production, as it accumulates in ponds or chimneys of garbage.

Some states in the United States of America began using fly ash up to 15%, such as Ohio where cement was replaced by fly ash, as it was found to improve soil properties and give it good cohesion strength and resistance to factors (Erdal, 2001), as fly ash has the potential to improve engineering properties. The main benefits that fly ash can provide are:

- Preservation of the environment: Fly ash is more beneficial as it can be better recycled compared to other remains in the environment. Low cost: Fly ash costs lower compared to cement or other soil fixing materials.
- Improves the properties of engineering soils as well as the tolerance of soil, similar to the cement of the grit, which affects the properties of the soil.

Fly ash using method:

Fly ash is used by forming a mixture of water and fly ash, as the moisture content affects the strength of the mixture. The optimum moisture content of the clay soil ranges from 4% to 8%. As for granular soil, the optimum moisture content is 3% or less. Therefore, moisture control is very important when mixing.

1.2.5. Grouting

It is one of the traditional methods used to improve soil performance geometrically. In addition, the aerosol increases the resistance of the soil and reduces the settlement, permeability, and porosity (Benzekriand and Marchand, 1978). Some of these solutions penetrate sub-surface formations more easily than others, depending on the degree of viscosity and the rate of chemical reaction or hardening.

Some of the above materials are gelatinized in seconds, whereas some others take hours to harden, and the penetration distance is not only associated with the pressure applied, but is more closely related to the size and permeability of the soil particles.

It allows long distances to penetrate more easily than clay soil, injection with cement and chemicals is relatively expensive, so a careful assessment of the economic growth is required (Al Muhaidib, 2004).

2. PREVIOUS STUDIES

Prabakar et al (2003) studied the improvement of engineering properties of fly ash and its contribution to ground. In order to understand the c-φ increasing behavior of the soil, they conducted a series of experiments by adding fly ash at various rates and they mixed the fly ash with soil to improve the soil properties. They carried out experiments with each sample, which had specific gravity, and compaction behavior, shear resistance and deviator stress, and they tested these samples without fly ash and with fly ash added from 9% to 46%. As a result of the experiments, they found that the compressibility of the ground with the contribution of fly ash decreased with respect to the low specific unit gravity and the weight per unit of volume. The decrease in compressibility was between 15-20%. The void ratio and porosity values varied according to the addition of fly ash to the ground. By adding about 46% of fly ash, the void ratio increased by 25%. The shear stress increased non-linearly by adding ash to the ash-soil mixture. Cohesion could also increase with the addition of ash. Reduction in swelling of the ground was detected with the increase of ash addition on swollen soils. The maximum cohesion value was 0.39kg / cm2 with the addition of ash to loose soil which was a mixture of organic sand and clay, and was 0.66kg / cm2 on clayey soils. Cohesion also increased linearly with ash increase. As a result, transport capacity increased by the addition of fly ash (Prabakar et al., 2003).

Şenol and Edil (2004) investigated the results of research on the stabilization of soils containing soft and partially organic material with very low load carrying capacity to increase the carrying capacity by using fly ash in road construction. They chose the various rates of mixtures to determine the thickness of the layer to be stabilized and the optimum soil-fly ash-water mixture. They found that C type fly ash stabilization using two types of ground greatly improved the engineering properties of the soft ground under the road and increased the soil strength. The CBR results of the blend samples obtained in the laboratory were at least ten times greater than those of the original samples. This result quickly provided an important data to practical use for the next step, land study. Floor stabilization with fly ash was a very sensitive work. In addition, another important result of this study was that the values obtained from the laboratory experiments and gave information about the soil strength were greater than the values obtained from the field. Land values were low about 9-6% of laboratory values. The reason for this was that the samples prepared in the laboratory environment reflected the ideal mixing conditions and were more homogeneous compared to the field environment. For this reason, it is appropriate to use a security number when it is necessary to switch from laboratory values to land values.

Çokça and Toktaş (2002) investigated the stabilization of a dispersive soil with C type fly ash. They added different ratios of (0%, 3%, 5%, 7%, 10%, and 13%), C type fly ash to the dispersive soil samples and examined the index, strength and consolidation properties of fly ash addition. The experiment results showed that the fly ash addition generally increased the strength of the sample and reduced its compressibility. In addition, with the addition of fly ash, the ground changed from the dispersive state to the non-dispersive state. Due to the low specific gravity of the Soma fly ash, as the amount of fly ash in the samples increased, the specific gravity dropped. As the amount of fly ash in the samples increased, the compressibility of the samples decreased, the optimum water content increased, and the maximum dry unit volume weight decreased. As the amount of fly ash in the sample increased by 7%, the nonconfined compression resistance of the sample increased, while the contribution of more fly ash led to a decrease in nonconfined compression resistance. With the addition of 13% fly ash, the sample changed from the dispersive state to the non-dispersive state.

Yoon et al. (2003) conducted a study on carrying capacity and seating of reinforced soils using old truck tire. They carried out plate loading experiments in the laboratory because they thought that it would be beneficial to use waste tires on sand ground. They carried out these experiments by considering factors such as relative tightness, sitting depth, number of reinforcement layers, type and size of tire. As a result, they found that the carrying capacity of the loose sand reinforced with old tires increased twice and the seating decreased by about 70%. In addition, the seating of tight sands reduced by 34%.

Kaniraj and Havanagi (1999) conducted a study on fly ash stabilization with cement. In this study, they mixed Rajghat fly ash in Delhi, India and Baumineral fly ash near Bochum, Germany with suitable soils. They mixed Yamuna sand and silt with Rajghat fly ash and Rhine sand with Baumineral fly ash. They added cement varying from 3% to 9% to the soil-fly ash mixture. As a result, they found that the increase in nonconfined compression value and secant modulus was hyperbolic. They also found that the increase in these values was due to the addition of admixture, and these values increased with the increase in cement, but decreased with the increase in fly ash. It is known that cement has more effect than ash. The water content varies depending on the amount of cement in the soil-ash mixture and the curing time. In other words, the water content decreases with increasing cement. It can be said that the amount of cement is more effective than curing time.

Kalinski and Hippley (2005) investigated the effect of water and cement content on Portland cement and fly ash. They performed Proctor and modified Proctor experiments to find the water content and carried out nonconfined compression tests to measure the strength. Prepared specimens were tested after curing for 30, 60, and 90 days. As a result, they obtained the desired results with the compaction of the F class fly ash and Portland cement. The results showed that; stabilization with cement and ash was influenced by the cement and water content and also the energy of the compaction played a major role. By knowing these parameters, nonconfined compression strength became predictable with CQA test.

While the optimum water content was 20-30%, the nonconfined compression resistance was 1.1-5.5kPa.

Tüdeş (1996) determined the physical properties of the soils by routine experiments and mineralogical properties by DTA (Differantial Thermal Analysis) and XRD (X-ray) experiments by selecting three of the soils of Eastern Black Sea Region for the stabilization of soils with lime and cement admixture. The selected soils were compressed at different rates with standard energy with cement and lime additives and shear resistance parameters were obtained. In the case of no additive, the shear resistance parameters of the soil compressed with the same energy were determined and the improvements obtained were compared. The obtained results showed that lime and cement had positive effects on the stabilization of the soil, especially that the additive ratios between 5% and 15% contained the optimum admixture and that the increase of the additive ratio did not increase the stabilization linearly in every material (Tüdeş, 1996).

Kavak and Bilgen (2005) evaluated the use of blast furnace slag (BFS) on the road infrastructure, especially with the purpose of strengthening the clayey soil. They investigated the effects of BFS on clayey soil by adding blast furnace slag (BFS) and lime at varying rates to the samples prepared with bentonite clay under laboratory conditions. For the non-confined compression tests, bentonite lime and slag were mixed at different rates by weight, and the optimum water content for each mixture was first determined, and the new mixtures prepared in these water contents were compacted at compaction tool in three layers in accordance with the road construction standards so that each layer had 25 strokes. As a result of the study, they observed that adding BFS to bentonite in size used as cement admixture without sieving did not make any significant change in the values of the bentonite clay nonconfined compression . It was clearly understood that when the slag was sieved down to a certain size (<150 μm) and when it was mixed with lime at different rates, it reacted with the bentonite clay. In the result graphs of the proctor tests, it was seen that the use of the slag in size used as the cement admixture more than 10% by weight caused a decrease in the optimum water content of the bentonite clay and an increase in the dry unit volume weights. In addition, if the sifted slag at 150 microns was added with lime to bentonite, the optimum water content did not change, the dry unit volume weights decreased, and the proctor curve became smooth. When the bentonite clay was mixed with 5% slag and 7,5% lime, the nonconfined compression value increased about 25 times from 273 kPa to 6690 kPa after 28 days. The unit deformations that occurred at fracture moment fell from 10-11% to 1%. Thus, the ground became a rigid structure and the modulus of elasticity increased.

Abdul Wahhab (1996), tried to prevent damage to the water by mixing asphalt with lime and cement in soil stabilization, and they indicated that cement was more effective than lime in comparison with the experiments using 2% to 4% cement and lime. They used two kinds of asphalt and stated that cement was more effective when emulsion type asphalt was used; lime and cement had the same effect when using asphalt.

Tunç (2002) in order to improve the properties of the ground, investigated some materials contributed to chemical cementation of the soil by chemical reaction with the soil. Chemical additives, organic materials and fibers, and inorganic materials and polymers such as phosphoric acid, phosphate, and calcium sulfate (jibs), sodium hydroxide (caustic soda), and aluminum salts used for this purpose. These were selected depending on the acidic and alkaline properties of the ground and were added to the soil at certain rates to provide more stability. The chemical substances to be used for the soil stabilization should be selected according to the type of ground and desired characteristics and should be used after proved.

3. MATERIALS AND METHOD

3.1. Materials

3.1.1. Handere Clay

The clay used in this study is obtained from the Handere Formation, which is named by Schmidt (1961) and located in Adana Basin. The unit primarily consists of uncolored claystone, pebbly sandstone, sandstone, siltstone, mudstone marl, and includes gypsum lenses and clay levels of various thicknesses in places. Gravel stones seen as large-scale trough cross bedded, while fine grains seen as parallel laminated. The thickness of the formation is in the range of 120 to 700 m (Yetiş and Demirkol, 1986).

3.1.2. Plaster Mortar (Master cast@)

Master cast@ is a high performance micro air entraining mortar admixture structured to enhance impermeability, workability, and freeze-thaw resistance in plaster mortars. Structure of the material is dilute solution of surface active substances with organic acid, Specific gravity 1.8.

This material is used in the following applications:

- In inner-outer spaces for vertical applications,
- In plaster mortars to improve impermeability,
- In brick and stone coating mortars
- Improving workability

The features and benefits of Master cast:

Has homogenous air entraining feature, reduces segregation and efflorescence effects, that can be seen on mortars without admixture, enhances neatness and workability features in mortars and enhances strength to freeze-thaw cycle and lowers costs (economical).

3.1. Methods

Collecting the soil samples from the field and transferring them to the laboratory, the necessary engineering tests were done with the help of the equipment available for use in the laboratory for the purpose of identifying the properties of the soil, and finding out the effects of the master cast used on the properties of the soil, along with the ability of master cast to improve the soil. Samples were prepared in 4 different cases pure soil and soil with Master Cast, 5%, 10%, 15%. Standard Proctor experiments were performed. Optimum Water Content is re-compacted in Proctor mold. Unconfined compressive strength, direct shear and Consolidation tests were carried out with the values took from Proctor mold. The American Society for Testing Materials (ASTM) procedures were employed in material analysis and the following tests were done:

- Atterberg limits (liquid limit, plastic limit, plasticity index) **(**ASTM D 4318_2004)
- Specific gravity (ASTM D 854-00, 2003)
- Hydrometer and sieve analyses (ASTM D 698-00, 2009)
- Standard proctor test (ASTM D 698-00, 2009)
- Unconfined compressive strength test (ASTM D 2166, 2009)
- Shear box test (ASTM D 3080, 2003)
- Consolidation test (ASTM D 2435, 2009)

3.2.1. Standard Proctor Test

This test is performed according to American standard for testing material (ASTM D 698-00, 2009). This test is used to find optimum moisture content and the maximum dry density. To construct airports, highway bridges and other structures, it is often required to compact soil to enhance its strength. In 1933 Proctor created a laboratory compaction test procedure in order to calculate the maximum dry unit weight of compaction of soils, which can be used for specification of field compaction.

Figure 3.1. Compaction mold and hammer

3.2.2. Atterberg Limits

This test is performed according to American standard for testing material (ASTM D 4318_2004). This test is used to find liquid limit and plastic limit and plasticity index. This test need to the following Equipment's is shown in Figure 3.2. This test is performed according to ASTM D 4318 to find out the plastic and liquid limits of a fine grained soil.

The liquid limit (LL) is known as the moisture content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus processed at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling (ASTM, 2004).

Albert Atterberg who was a Swedish soil scientist had first described seven “limits of consistency” to classify fine-grained soils, but only two of the limits, the liquid and plastic limits, are now applied in the current engineering practice (AASHTO). A third limit, which is the shrinkage limit, is also sometimes used. The Atterberg limits are defined according to the moisture content of the soil. The plastic limit is the moisture content that specifies where the soil changes from a semi-solid to a plastic (flexible) state. The liquid limit is the moisture content that specifies where the soil changes from a plastic to a viscous fluid state.

A wide diversity of soil engineering properties are correlated to the liquid and plastic limits, and these Atterberg limits are also employed for classifying a fine-grained soil as stated by the Unified Soil Classification system or AASHTO system

Figure 3.2. Casagrande tool

3.2.3. Hydrometer and Sieve Analyses

The hydrometer test is conducted to determine the grain size distribution for the fraction of soil this test performed according to (ASTM D 698-00, 2009). The hydrometer test is executed to find out the grain size distribution for the fraction of soil which is smaller than No. 10 (2.00 mm) sieve. Soaking the soil sample in a dispersing agent and rapidly stirring to neutralize the charges between the soil particles, fine soil grains are diffused

The test uses a type 152H hydrometer calibrated in order to give the mass of solids with specific gravity similar to 2.65 in suspension and the settling velocity of the dispersed soil grains. The soil grain diameter (D) (mm) is calculated by the application of Stokes' Law which is a theoretical equation for the terminal settling velocity of spheres in a fluid, as follows (ASTM D 698-00, 2009):

Figure 3.3. Tools used in hydrometer test

3.2.4. Specific Gravity Test

This test is performed according to (ASTM D 854-00, 2003). This test is used to find the specific gravity of soil. Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature.

Significance: The specific gravity of a soil is used in the phase relationship of air, water, and solids in a given volume of the soil.

Figure 3.4. Pycnometer flask

3.2.5. Unconfined Compressive Strength Test

This test performed according to (ASTM D 2166, 2009). This test is used to find the Unconfined Compressive Strength. The test procedure Measure the Diameter and the Height of the specimen as well as gauge length Lo. These values will be used to calculate stresses and strains from measured deformation data, After testing each model the peak load can be observed, sample's failure mode Appears after the scan as the figure 3.7.

Figure 3.5. Failure mode of the sample

3.2, 6. Shear Box Test

This test is performed according to (ASTM D 3080, 2003). The shear box machine in Figure 3.5. The purpose of the direct shear test is to quantify the soil cohesion and friction angle and to determine the shear behaviors of the soil sample. To prepare the soil sample for the direct shear test: firstly, the bottom halves of the apparatus, the motor drive is set up followed by lodging the plate and stone into place to make the soil sample. After that, different dimensions of the soil such as height, weight and compacted weight are measured. The soil sample is aligned under the normal load cell. After the soil sample being lodged into place, dimensions are adjusted and the soil sample box is secured. Then, different readings apparatus, for example, swing hanger, vertical dial gauge, and horizontal dial gauge are placed. Then, after resetting the load and taking out 2 logging pin, which is a very important step in order to prevent measuring the shear strength of fixing pin instead of the soil sample, it is ready to start the test and start readings. All the processes are to be repeated for different loadings.

Figure 3.6.a. Shear box machine, b. Shear box apparatus

3.2.7. Consolidation

Consolidation test is performed according to (ASTM D 2435, 2009). The consolidation tool as shown in Figure 3.6. The consolidation test is performed to detect the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to various vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. It is useful in detecting the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil. Also, the achieved data can be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil (ASTM D 2435, 2009).

Figure 3.7. The parts of consolidation tool.

4. RESULTS AND DISCUSSION

The test results obtained from the soil samples taken in the field and the soil – master cast mixtures, and their engineering properties will discussed in this chapter.

4.1. Atterberg Limits

This test is one of the significant tests to find the liquid limit, plastic limit and the plasticity index of the soil.

The results obtained are shown in the figures (4.1-4.4). Decreases in liquid limit were observed from 49.07 in pure soil to 49.0,46, and 45 in soils with 5%, 10%, and 15% material, respectively.

The plastic limit test was conducted in the laboratory. The results are presented in the figure (4.5). Increases in the amount of plastic limit were observed by the addition of the material. Where the pure soil plastic limit was 24.05, it was 25.3 for the soil with 5% of material, 27.08 for the soil with 10% of material, and 29.21 for the soil with 15% of material.

Plasticity index of the soil was found after the identification of the liquid limit and plastic limit, as shown in the figures (4.1-4.4). The decreases in the plasticity indices were observed from 25.05 in pure soil to 23.7 in the soil with 5% material, 19.62 in the soil with 10% material, and 15.79 in the soil with 15% material.

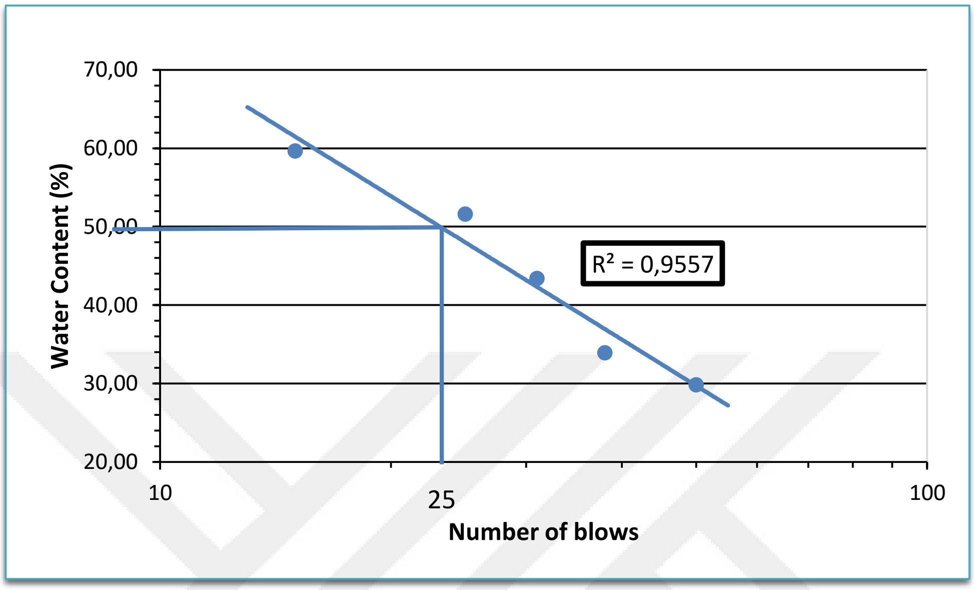

Figure 4.1. Liquid limit for soil in pure soil

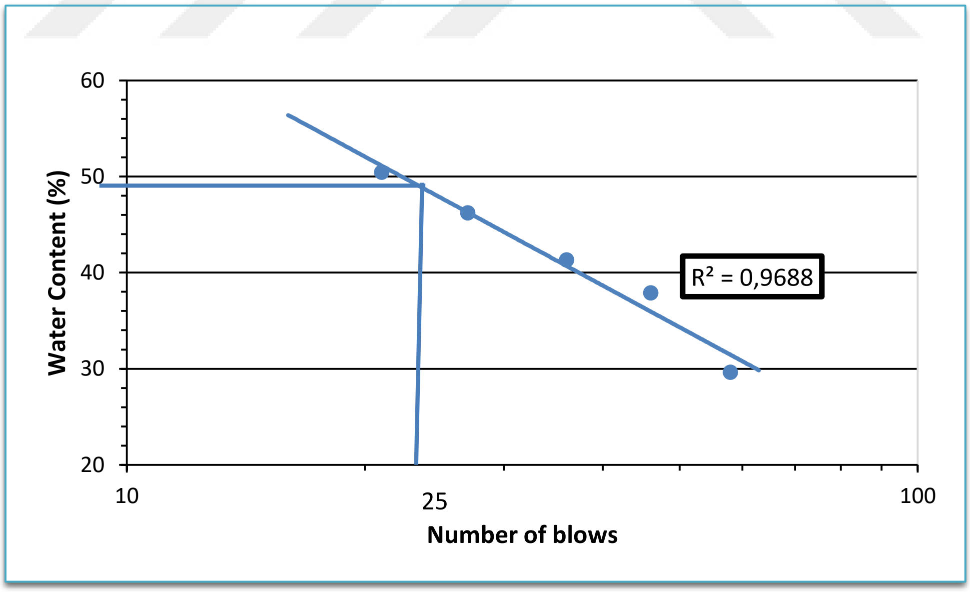

Figure 4.2. Liquid limit for soil with 5% percentage of material

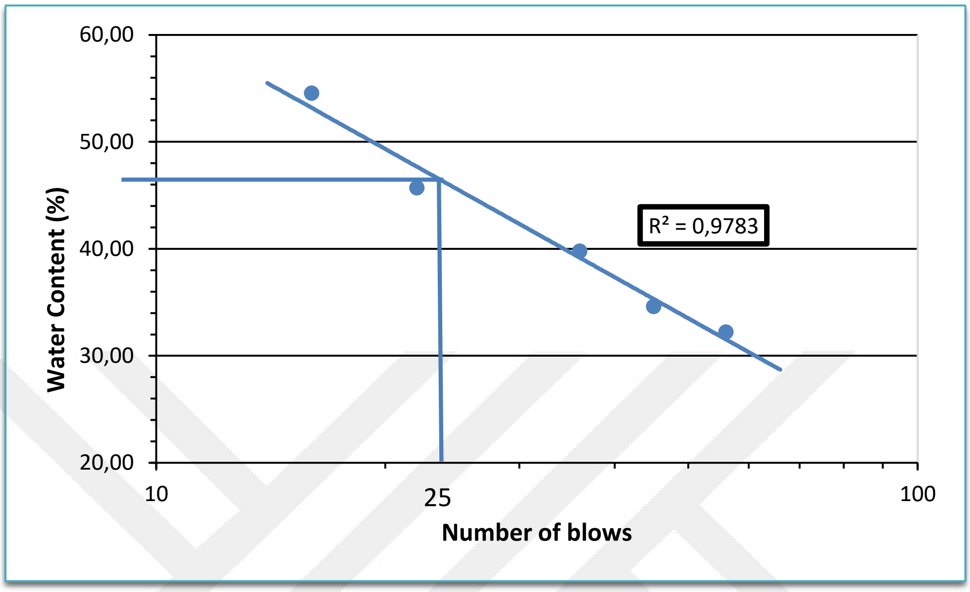

Figure 4.3. Liquid limit for soil with 10% percentage of material

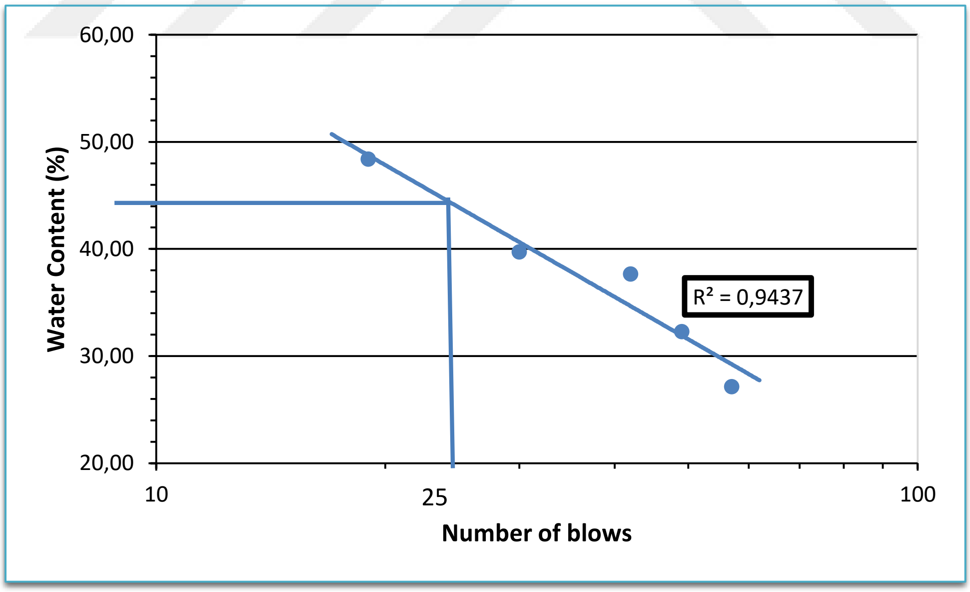

Figure 4.4. Liquid limit for soil with 15% of material

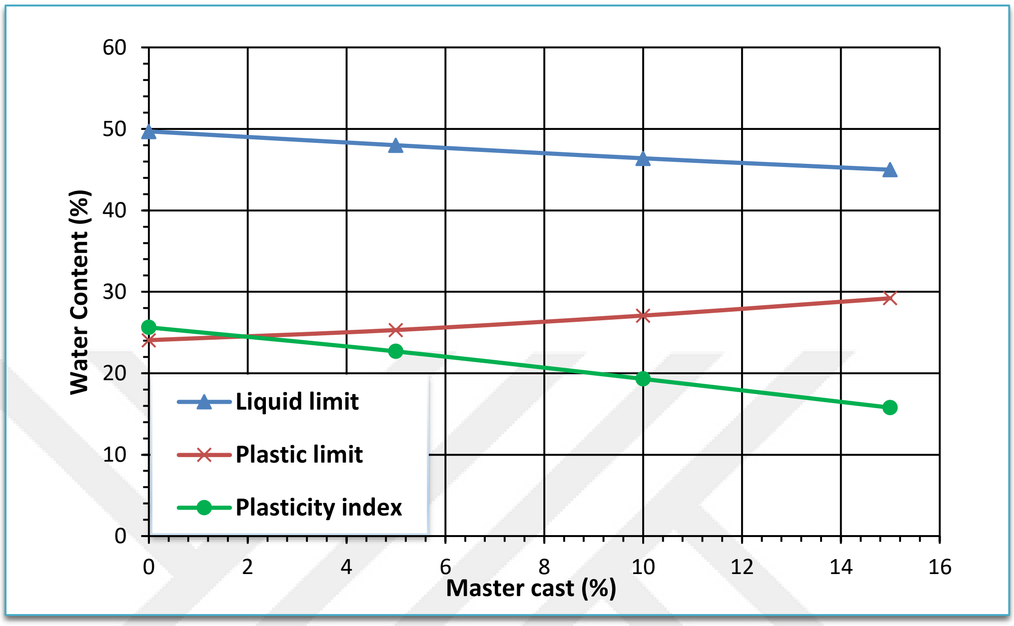

Figure 4.5. Changes of liquid limit (LL), plastic limit (PL), plasticity index (PI)with respect to percent master cast.

4.2. Specific Gravity

This test was conducted in three trials to determine the specific gravity for soil before the hydrometer test. The first trial value was 2.70535 and the second trial was 2.37207 and the last trial was 2.712283. The average values was then 2.712283 (Table 4.1). The value 2.71828 is then used in the hydrometer analysis.

Table 4. 1. Specific Gravity of the samples

Mass of soil	$M_{pws,t}$	M_{pws}	Gt	k	G
35.12	375.95	353.8	2.707787	0.9991	2.70535
35.15	375.82	353.5	2.739673	0.9991	2.737207
35.21	375.64	353.4	2.714726	0.9991	2.712283
Average					2,711828

4.3. Hydrometer and Sieve Analyses Test

This test was carried out in three trials in the laboratory to learn the percentages for each component in the Handere Formation soil before the classification of the soil. According to the test results, the percentage for each of the components was 56.41 % for clay, 37.49% for silt, 6.1% for sand, and 0 for gravel. Figures (4.6- 4.8) demonstrate the categorization of the soil by type and content, where the percentages of clay, sand, silt, and gravel is found in the expanse shown below. The soil named and classified according to USCS classification system after this test as shown Table 4.2.

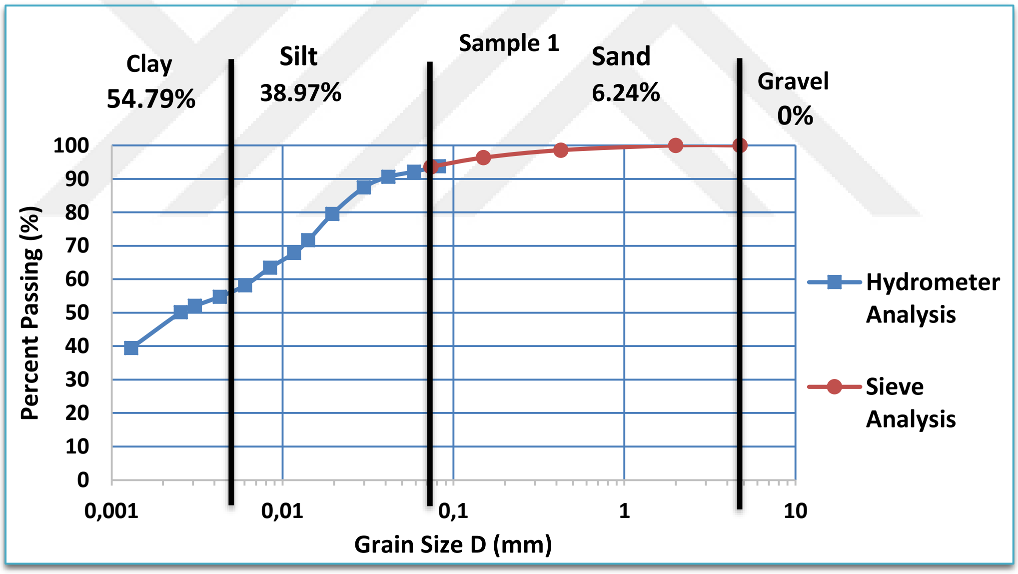

Figure 4.6. First trial of hydrometer test

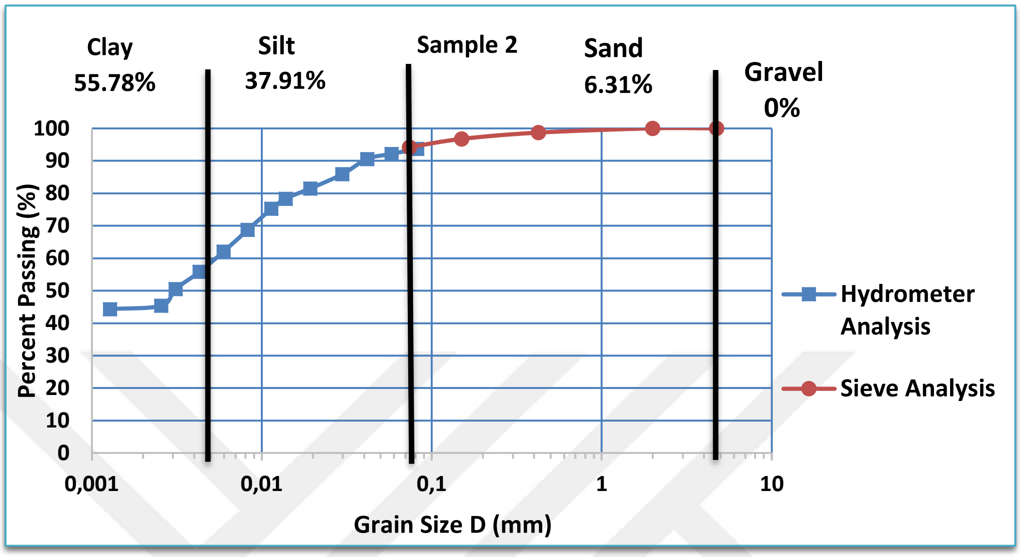

Figure 4.7. Second trial of hydrometer test

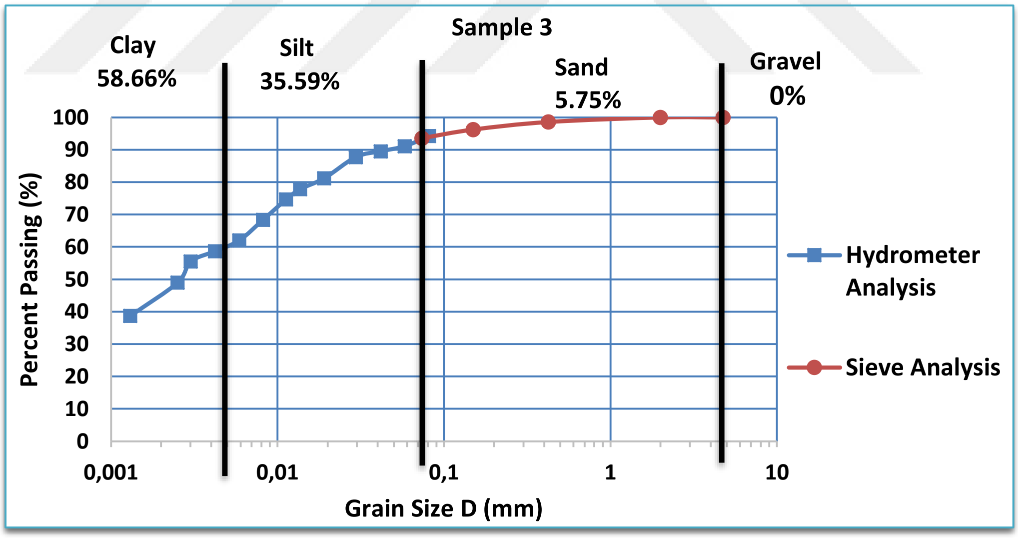

Figure 4.8. Third trial of hydrometer test

Table 4.2. Soil classification and soil name according to USCS classification system

No	Percentage of Material %	Plastic Limit %	Liquid Limit %	Plasticity Index %	Classification	Name of Soil
1	0	24.05	49.7	25.05	CL	Lean clay
2	5	25.30	49.0	23.7	CL	Lean clay
3	10	27.08	46.7	19.62	CL	Lean clay
4	15	29.21	45.0	15.79	CL	Lean clay

4.4. Compaction Test

This study aimed to find out the optimum moisture content and maximum dry density values before and after the addition of the master cast. The results of the proctor test exhibited in Figure 4.9 show a difference in optimum water content and maximum dry density before and after the addition of the substance in different percentages (5%, 10%, 15%, respectively). Figure 4.9 demonstrates that water content optimized by the effect of the material used.

According to compaction results, optimum water content for pure soil was 24.2% and dry density for pure soil was 1.455 g/cm^3. The compaction results show that, the optimum water content for plaster mortar treated soil was 22.5% for soil with 5% of material and the dry density was 1.465 g/cm^3, the optimum water content was 21.5% for soil with 10% of material and the dry density was 1,5 g/cm^3. The optimum water content decreased to 21.4% for soil with 15% of material and the dry density increased to 1.56 g/cm^3 for soil with 15% of material.

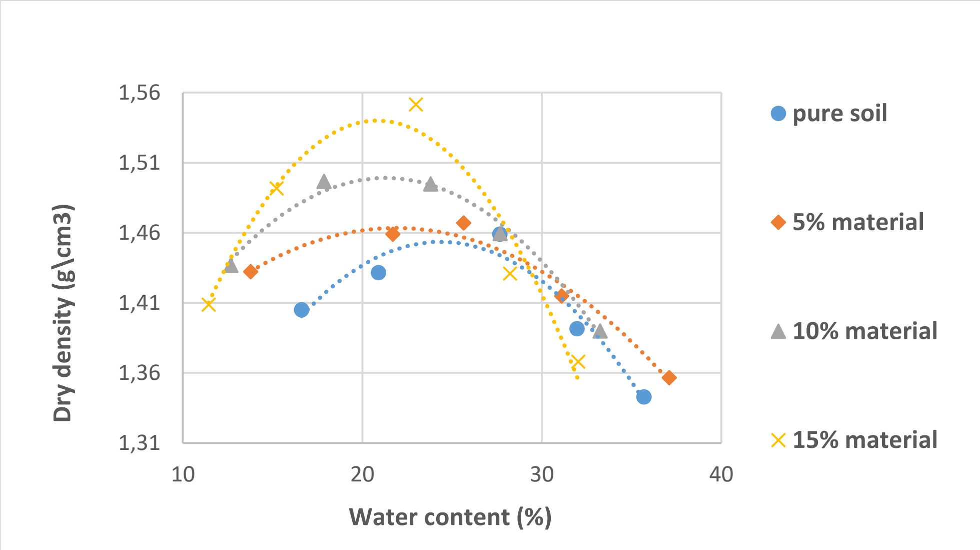

Figure 4.9 Compaction tests results

4.5. Unconfined Compressive Strength

At the beginning, the test was conducted on three specimens in the natural state of the soil, and next, the test was carried out after adding different percentages of master cast material (5%, 10%, 15% , respectively). The effect of the material is explained in the figure 4.10 for each percentage and natural weight. It was found, as shown in the figures 4.11- 4.13, that adding the material to the soil leads to a difference in the results of the test. The increases in the unconfined compressive strength started of 10.27 kg\cm^2 for pure soil to 13.8 kg\cm^2 in soil with 5% of material and 20.38 kg\cm^2 soil with 10% and 23.04 kg\cm^2 with 15% of material respectively (Table 4.3).

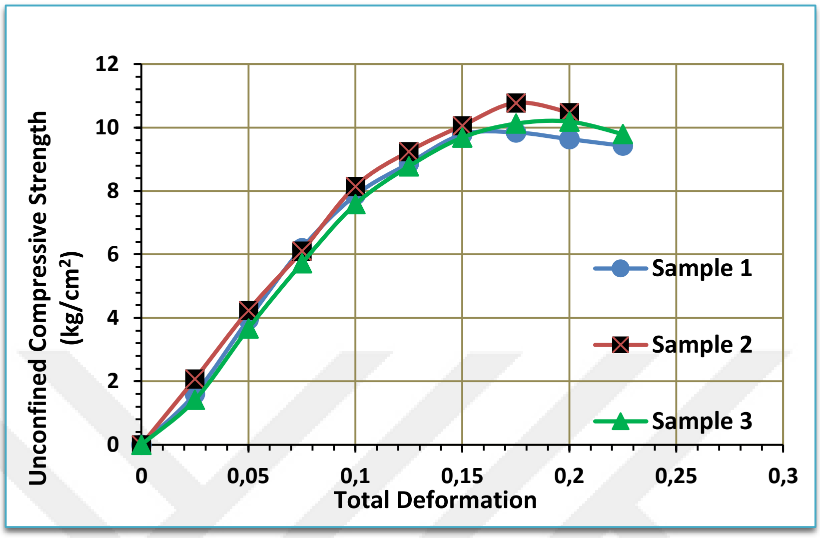

Figure 4.10. Unconfined compressive strength for natural soil

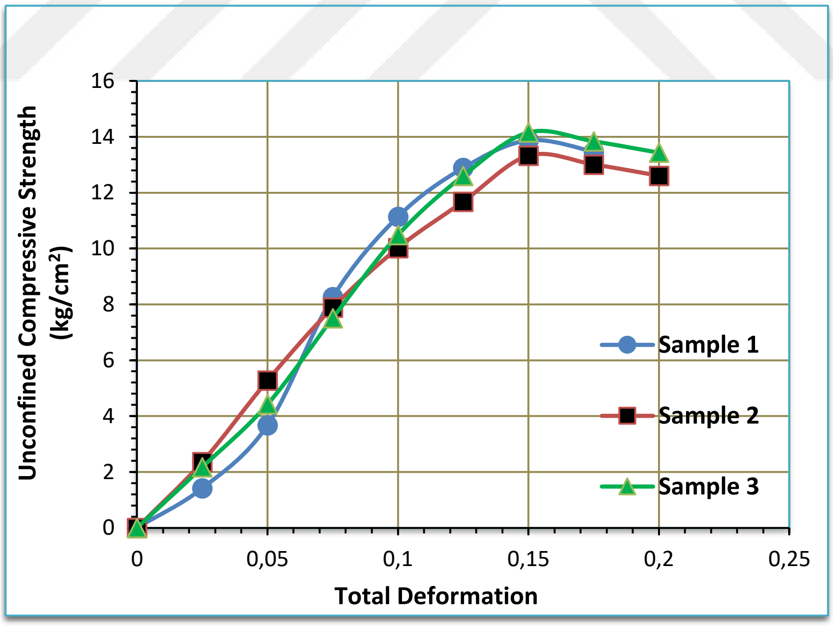

Figure 4.11. Unconfined compressive strength test for soil with (5%) percentage of material.

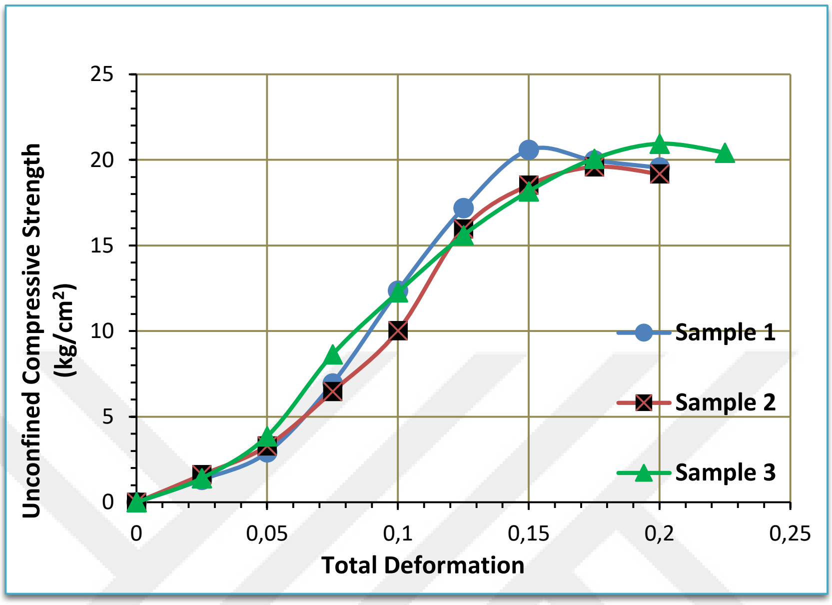

Figure 4.12. Unconfined compressive strength for soil with 10 % percentage of material

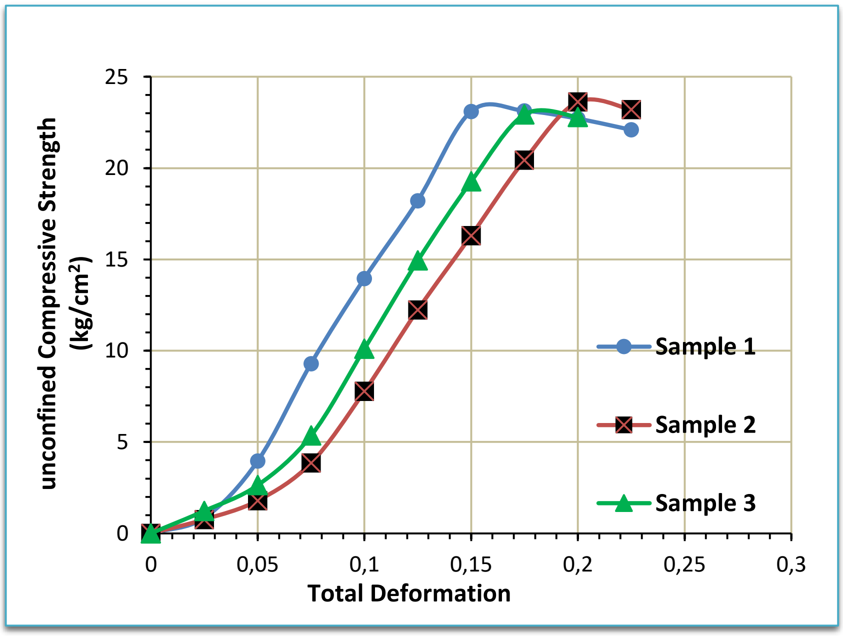

Figure 4.13. Unconfined compressive strength for soil with 15 percentage of

Material

Table 4.3. Shown the average unconfined compressive strength for natural soil and the soil with the testing Material

1 Day Drying			
No.	**% Master cast material**	**Unconfined Compressive Strength (kg/cm^2)**	**Average Unconfined Compressive Strength (kg/cm^2)**
1	0%	9.85	10.27
		10.78	
		10.19	
2	5%	13.80	13.78
		13.32	
		14.16	
3	10%	20.58	20.37
		19.60	
		20.94	
4	15%	22.70	23.04
		23.63	
		22.80	

4.6. Direct Shear Test

The shear strength of the soil was tested by using direct shear box in the laboratory. This test is carried out as follows: First, the natural soil was sheared at different loads (7, 10, 13, 16 kg). The soil is tested after the addition of structural plaster mortar, the material used in the research, to determine the effect of the shearing on the soil, where the material was added at different percentages (5%, 10%, 15%).

The soil was tested on the same loads used in natural soils and according to ASTM and the results are presented in the figures 4.15_4.18. The shear strength capacity increases parallel to an increase in the amount of the material added.

The shear stress values under 16 kg load were 2.25 kg\cm^2 for pure soil and 2.63 kg\cm^2 for soil with 5% of material and 3.15 kg\cm^2 for soil with 10% of material and 3.36 kg\cm^2 for soil with 15% of material. These results show that the shear strength increases as the percent mortar increases.

The R^2 value was 0.9727 in the natural soil; 0.9849 in the soil with 5% of plaster mortar; 0.9836 in the soil with 10% plaster mortar and 0.9938 with 15% plaster mortar. The difference in the results stems from the effect of plaster mortar on the shear strength of the soil as shown in the figure (4.14)

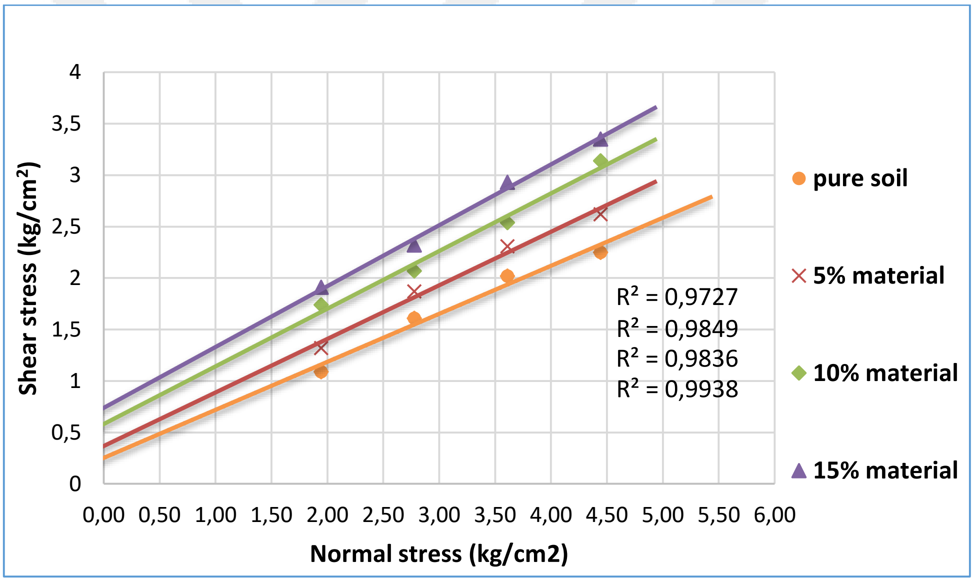

Figure 4.14. Failure envelopes for soils before and after adding the material

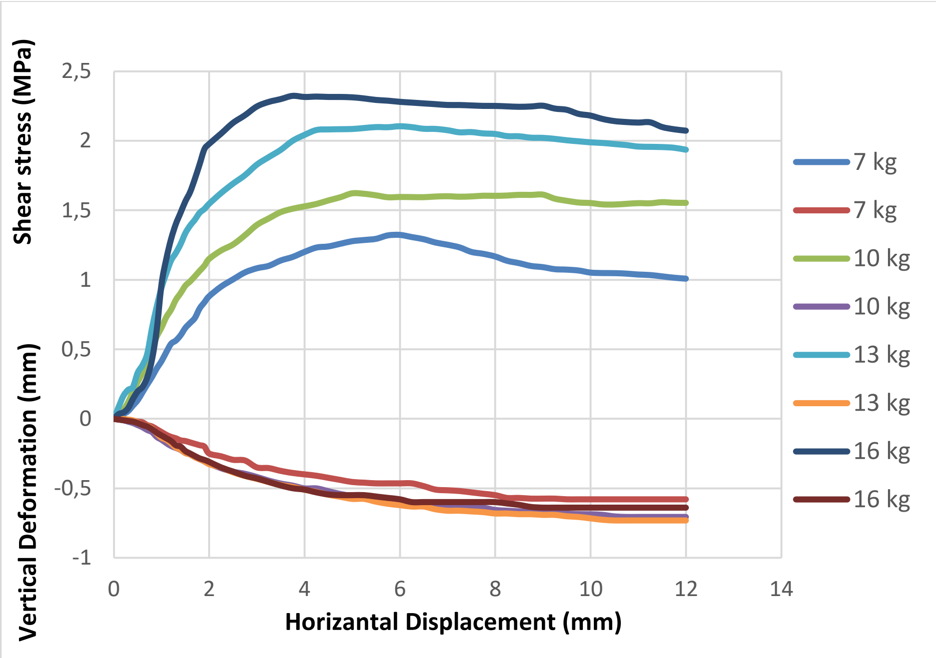

Figure 4.15. Shear stress for pure soil

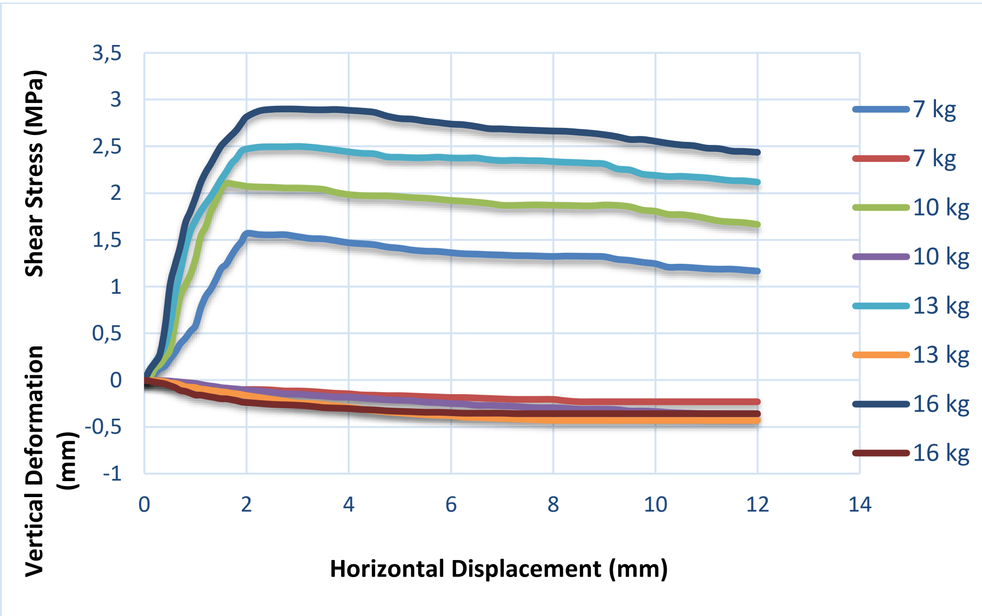

Figure 4.16. Shear stress for soil with 5% of material

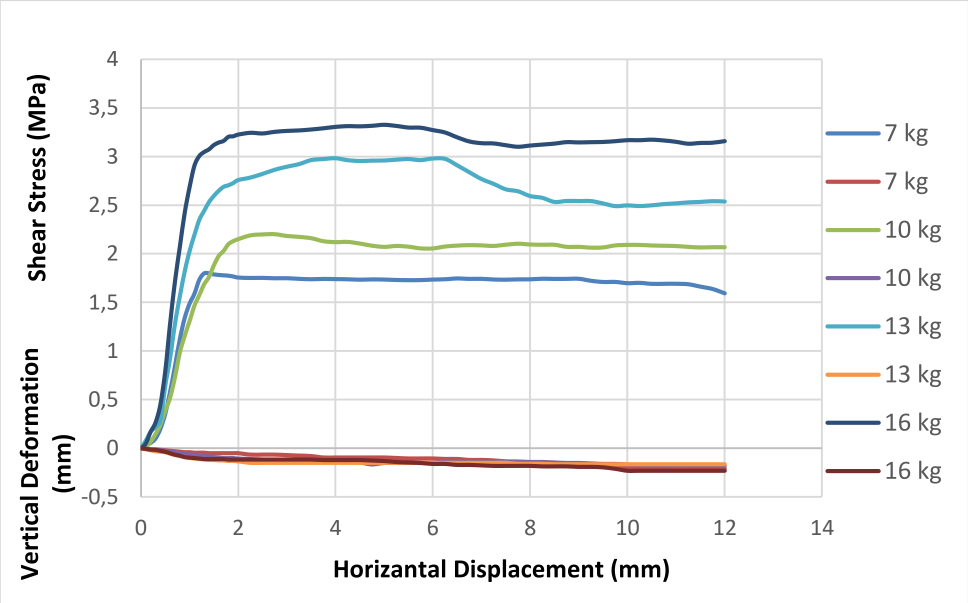

Figure 4.17. Shear stress for soil with 10% of material

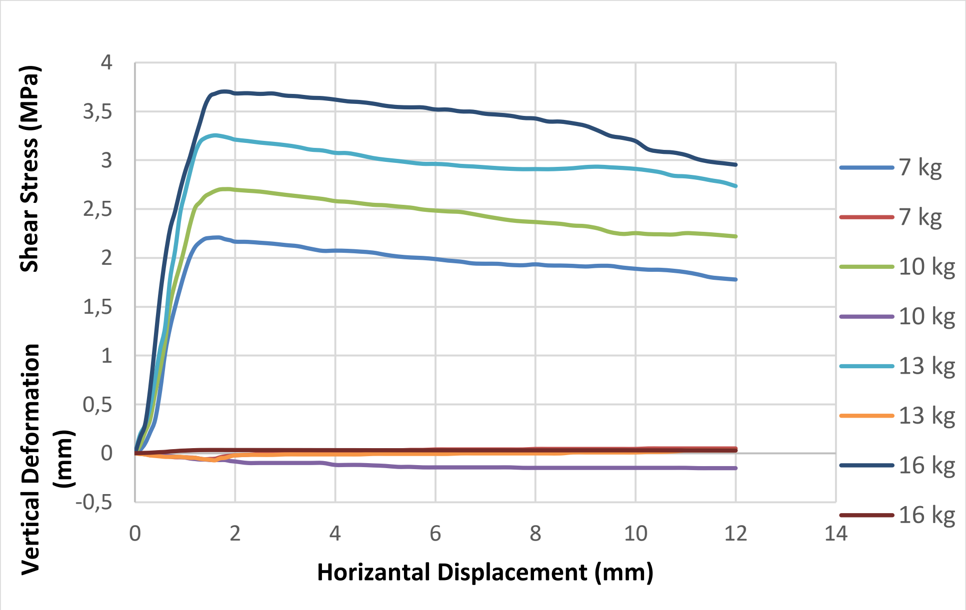

Figure 4.18. Shear stress for soil with 15% of material

4.7. Consolidation Test

In the beginning, the natural soil was tested in different loads. Next, the soil consolidations are tested in different percentages of master cast material (5%, 10%, 15%). The loading program was chosen as 0,25, 0,50, 0,75, 1,0, 1,5, 2,0, 3,0, 4.0, 5,0, 6,0, 8,0, 10, kg in order to obtain a more detailed curve with a distict break.

The results are exhibited in the figures below. As can be seen, the coefficient of volume of compressibility (Mv) decreases as shown in Figure 4,19. It gradually decreases in the soil with 5% by a small percentage, but with the increase to 10% of material, Mv decreases sharply.

Then, while it stays about the same for higher loading increments (1.5-10 kg for example), it slightly decreases for the load increment of 1-1.5 kg and it slightly increase for the load increment of 0.25-1.0 kg.

The analysis of the results also, show that pre-consolidation stress was 1.4 for natural soil, 1.8 for soil with 5%, 1.6 for soil with 10%, and 1.4 for soil with 15% material. The void ratios decreased from 0.8723 in pure soil to 0.6878 at 15% material. The pre-consolidation stress results shown in figures 4.20- 4.24.

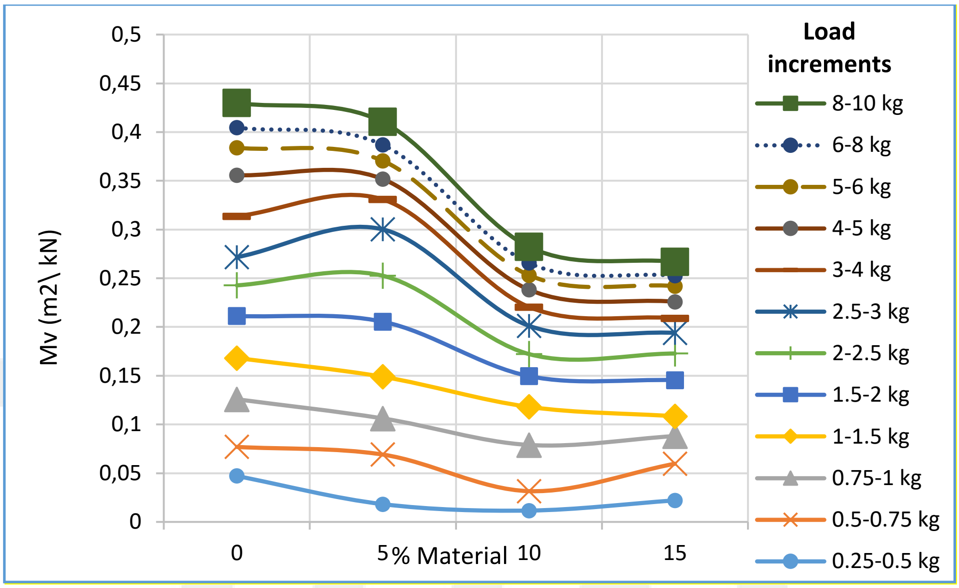

Figure 4.19. The coefficient of volume of compressibility (Mv) change under Different loads with respect to % material.

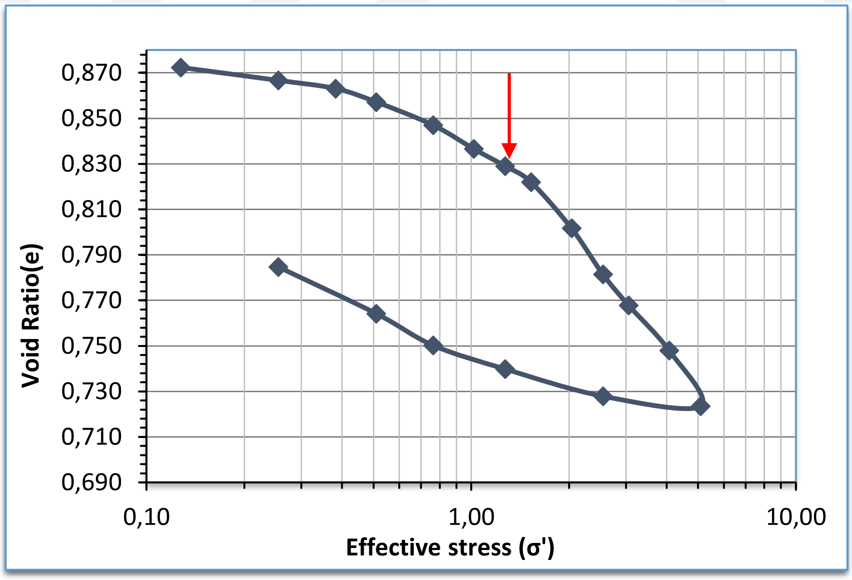

Figure 4.20. Void ratio verses effective stress curve for pure soil (the red arrow Shows the pre-consolidation pressure).

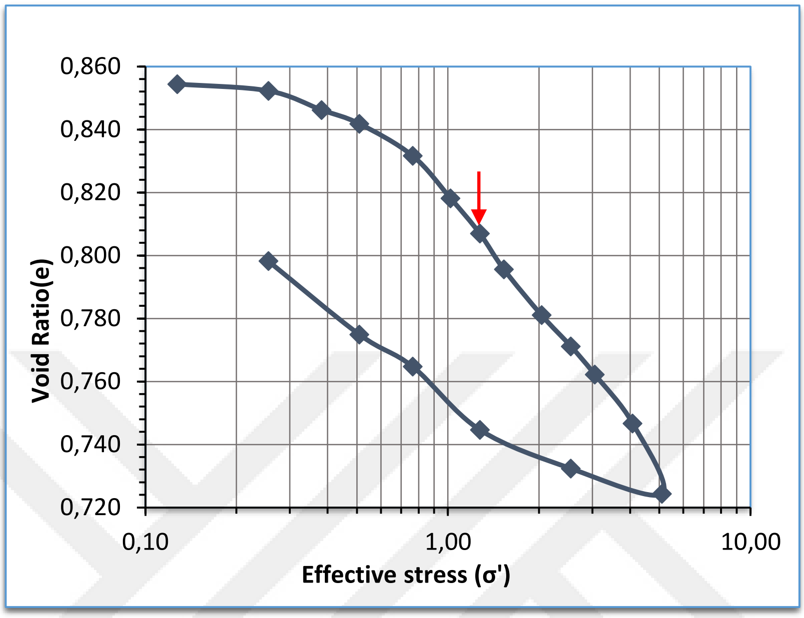

Figure 4.21. Void ratio verses effective stress curve for soil with 5% (the red arrow Shows the pre-consolidation pressure).

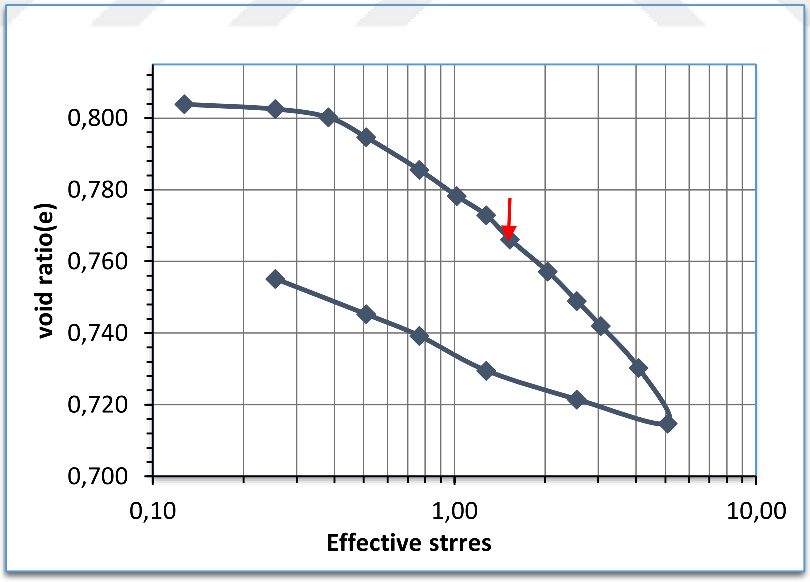

Figure 4.22. Void ratio verses effective stress curve for soil with 10% (the red Arrow shows the pre-consolidation pressure).

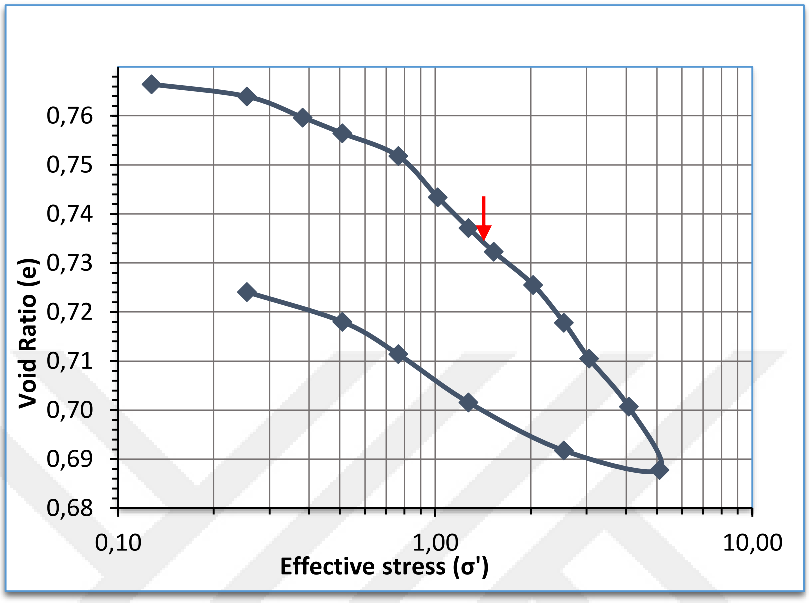

Figure 4.23. Void ratio verses effective stress curve for soil with 15% (the red Arrow shows the pre-consolidation pressure).

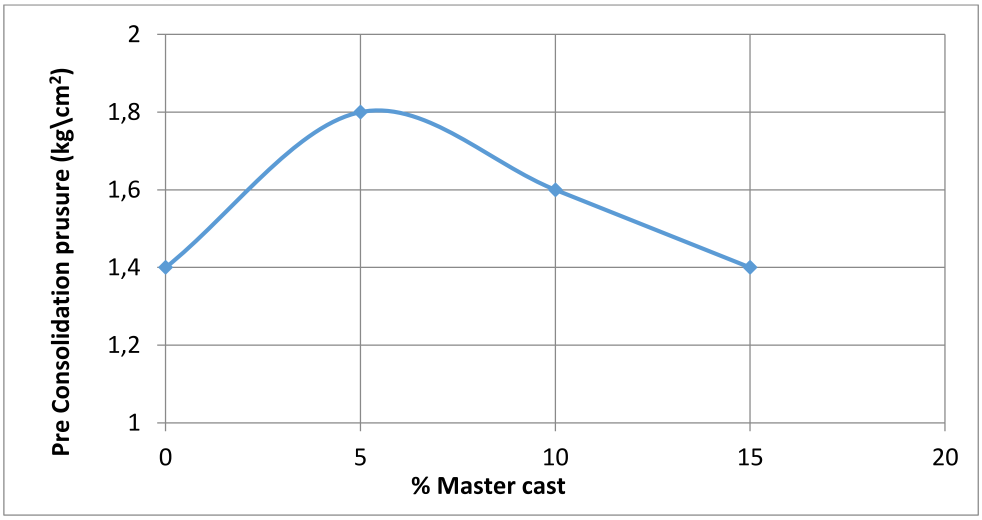

Figure 4.24. Pre consolidation stress change with respect to % material

5. CONCLUSION AND RECOMMENDATIONS

5.1. Conclusion

The plasticity limit for pure soil was 24.05%, liquid limit was 49.7% and plasticity index was 25.05%. Soil plasticity increased from 24.05% in pure soil to 29.21% at 15% added material. The liquid limit of 49.7% in pure soil decreased to 45% at 15% material. The plasticity index decreased from 25.05% in pure soil to 15.79% at 15% material.

The Hydrometer and sieve analyses found the components of pure soil as follows: clay 56.41%, silt 37,49%, sand 6.1%. According to soil classification system, USCS soil was CL type and the name lean clay soil. The specific gravity for soil was 2.7.

According to the compaction test results, the optimum water content for pure soil was 24.2% and the dry density was 1.455 g/cm^3. Also, the results show that the optimum water content for plaster mortar treated soil was 21.4% and the dry density was 1.54 g/cm^3 at 15%. The optimum moisture content decreased from 24.2% in pure soil to 21.4% at 15% added material. The dry soil density increased from 1.46 g/cm^3 in pure soil to 1.54 g/cm^3 at 15% added material.

The unconfined compressive strength increased from 10.27 kg/cm^2 in pure soil to 23.03 kg/cm^2 with 15% material. The shear strength of pure soil increased from 1.09 kg/cm^2 to 3.35 kg/cm^2 at 15% material.

According to the consolidation test, the void ratios decreased from 0.8723 in pure soil to 0.6878 at 15% material. The coefficient of volume of compressibility (Mv) decreases as % material increases. The decrease is higher in higher load increments, then the lower ones. The plasticity values of the soils were reduced by master cast addition. Besides, it was determined that the highest maximum dry unit weight and the lowest optimum moisture content were obtained by 15% master cast addition. The soil strength properties were reached to the maximum values in case of 15% master cast added into the mixtures. Besides, it is determined that the coefficient of volume compressibility (Mv) and the pre-consolidation pressure values are ideal when the master cast ratio in the mixtures are 10% and 5% respectively.

5.2. Recommendations

Recommend experimenting with other cement additives for soil improvement. Conducting experiments on Master cast added soil with different percentages.

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