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THE EFFECT OF THE USAGE OF SAWDUST AND QUICKLIME ON SHEAR STRENGTH BEHAVIOUR OF CLAYEY SILT SOIL

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A thesis submitted by Omar Hamdi JASIM in partial fulfillment of the requirements for the degree of **MASTER OF SCIENCE** is approved by the committee on 21.04.2016 in Department of Civil Engineering, Geotechnical Engineering Program.

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Omar Hamdi JASIM

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LIST OF SYMBOLS

C-X	Halogen group
μm	Micro meter
c	Cohesion of the soil
τ_f	Shear strength at failure
$\Delta\sigma$	Deviatoric stress
σ_1	Major principle stress
σ_3	Minor principle stress (confining pressure)
C	Carbon
τ	Shear strength
P	Uniform vertical normal load
ε_f	Strain at failure
ε	Strain
σ	Normal Stress
qu	Unconfined compressive strength
φ	Angle of internal friction
G_s	Specific Gravity

LIST OF ABBREVIATIONS

ACI	American Concrete Institute
CBR	California Bearing Ratio
CD	Consolidated Drained
CU	Consolidated Undrained
FT-IR	Fourier Transform Infrared Spectroscopy
Imm	Immediate
KN	Kilo Newton
L	Lime
LFA	Lime-Fly Ash
LT	Long Term
LL	Liquid limit
MDD	Maximum dry density
MH	High plasticity silt
OMC	Optimum moisture content
PL	Plastic limit
PI	Plasticity index
RHA	Rice husk ash
SD	Sawdust
SEM	Scanning electron microscopy
SCM	Supplementary cementitious materials
UCS	Unconfined compressive strength
UU	Unconsolidated Undrained
USCS	Unified Soil Classification System

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ABSTRACT

THE EFFECT OF THE USAGE OF SAWDUST AND QUICKLIME ON SHEAR STRENGTH BEHAVIOR OF A CLAYEY SILT SOIL

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MSc. Thesis

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Soil improvement with some additives is a common technique to improve the properties of soils such as shear strength, bearing capacity etc. and to reduce settlement and lateral and vertical deformations under transmitted loads from the structures. Recently a lot of researchers have conducted the studies about the interaction between the natural and environmentally friend’s materials with the soil. However, in the literature, there is no investigation about the effect of sawdust and quick lime on the shear strength parameters of a grinded clayey silt soil passed from sieve No 40. Sawdust and lime were mixed by (1, 2, 3 and 5) % from dry weight of soil, separately, with a soil was given from the district of Büyükçekmece in Istanbul city. The soil was compacted with and without the additives and the consistency limits were studied for both cases. The unconfined compression (UCS) and Unconsolidated Undrained Triaxial tests (UU) were conducted on the samples to see the immediate and long-term effects of sawdust and lime on the shear strength.

The results show that the addition of sawdust and lime decrease liquid limits and plasticity indexes for the soil. The quick lime behaves unlike sawdust, it increases the plastic limits of the studied soil. The maximum dry unit weight decreased with increasing content of additives. The sawdust decreases the optimum water content while lime increases it during compaction tests. The study concluded that the addition of sawdust up to 5% decreased the liquid limit, plastic limit and the plasticity index by 13.01%, 11.63% and 15% respectively. The addition of lime up to 5% decreased the liquid limits and the

plasticity index by 14.52% and 52% respectively, but it increased the plastic limits by 11.63%.

For the shear strength, 5% was the optimum lime content in the immediate and long term while the optimum changed from 3% in the immediate to 2% in the long term for the case of sawdust.

Key words: Undrained Shear Strength, Sawdust, quick lime, Triaxial Test, Unconfined Compression Test, Consistency Limits, Compaction Test.

ÖZET

TALAŞ VE SÖNMEMİŞ KİREÇ KULLANIMININ KİLLİ SİLT BİR ZEMİNİN KAYMA DAYANIM DAVRANIŞI ÜZERİNE ETKİSİ

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Zeminlerin bazı katkılarla iyileştirilmesi zeminlerin kayma mukavemeti, taşaıma gücü gibi özelliklerinin iyileştirilmesi ve oturmaların ve yatay ve düşey deformasyonların yapılardan iletilen yükler altında azaltılması için yaygın bir tekniktir. Literatürde talaş ve sönmemiş kirecin 40 nolu elekten elenmiş bir zeminin kayma mukavemeti parametreleri üzerine etkisi ile ilgili bir çalışma bulunmamaktadır.Talaş ve sönmemiş kireç Büyükçekmece ilçesinden getirilen zemine kuru ağırlığın %1,2,3 ve 5 oranlarında ayrı ayrı karıştırılmıştır. Zemin katkılı ve katkısız olarak kompakte edilmiş ve kıvam limitleri her iki durum içinde araştırılmıştır. Numunler üzerinde talaşın ve sönmemiş kirecin kısa süreli ve uzun süreli etkilerini görmek için serbest basınç deneyi ve UU üç eksenli basınç deneyleri yapılmıştır.

Sonuçlar talaş ve sönmemiş kireç eklemenin lilit limit ve plastisite indeksini düşürdüğünü göstermiştir. Sönmemiş kireç talaştan farklı olarak plastic limiti yükseltmiştir. Zeminin kuru birim hacim ağırlığı katkı yüzdesinin artması ile düşmüştür. Kompaksiyon deneyi sırasında talaş optimum su muhtevasını düşürürken sönmemiş kireç optimum su muhtevasını yükseltmiştir.

% 5 talaş eklenme likit limit, plastic limit ve plastisite indeksini sırasıyla %13.01, %11.63 ve %15 oranlarında azaltmıştır. % 5 sönmemiş kireç eklenmesi likit limit ve plastisite indeksini sırasıyla %14.52 ve %52 oranlarında azaltmış fakat plastic limiti %11.63 oranında arttırmıştır. Kayma dayanımı açısından, %5 sönmemiş kireç oranı kısa süreli ve uzun süreli etki için optimum oran olarak belirlenmişken talaş için kısa süreli etki

açısından %3 optimum oran iken uzun süreli etki için %2 optimum oran olarak belirlenmiştir.

Anahtar Kelimeler: Drenajsız kayma dayanımı, talaş, sönmemiş kireç, üç eksenli basınç deneyi, serbest basınç deneyi, kıvam limitleri, kompaksiyon testi

CHAPTER 1

INTRODUCTION

1.1 Literature Review

1.1.1 General Information

Soil shear strength is an important parameter in geotechnical engineering applications. The safety of any geotechnical engineering structure dependents on the shear strength of the soil beneath it [1]. The shear strength of soils is a valuable aspect in many problems such as the bearing capacity of shallow foundations and piles, also it is important for the stability of the slopes of embankments, dams, and lateral earth pressure on retaining walls [2]. Determining the shear strength could lead to the classification of the condition of a soil entity [3], and it increases the ability of engineers to imagine the critical conclusions about the overall the soil in a specific environment.

From a continuum mechanics standpoint, shear strength of common engineering materials, such as steel, is control by the molecular bonds that hold the material together. The higher the shear strength of a material, the stronger the molecular structure must be [2] However, soil shear strength works under a different set of principles. Soil is a material consist of particles, thus, shear failure occurs when the stresses between the particles are such that they either slide or roll over each other. Because of the particulate nature of soil, unlike that of a continuum, the shear strength depends on the interaction between the particles rather than the internal strength of the soil particles themselves [3].

1.1.2 Characteristics of Soil

The consideration of the geotechnical behavior of the underlying and surrounding soils must be always taken during the process of designing the infrastructures and other facilities. This is because all of the stresses and energies transferred to the ground by the soils. Unlike many of common engineering materials such as concrete and steel, the properties of soils have a high degree of variability and the soils in reality are non-homogeneous materials [4]. Because of the condition of the forming of soils, it is often kind of challenging to work with.

1.1.3 The Effect of Particles Size on the Shear Strength

The shape and the grain size distribution of soil particles have a big effect on the basic behavior of composite soils with different ranges of particle distribution was investigated by Li [5], two groups of soil were prepared for comparing the results, the first group is mixtures of fines (clay and silt) and an ideal coarse fraction (glass sand and beads), and the second one is mixtures of fines and natural coarse fraction (river sand and crushed granite gravels), direct shear box test was conducted on 34 samples and the structure of the shear surfaces, the shape of the particle, the coefficient of the particle shape of the samples, water content, and changing in volume and were examined, the results showed that the contraction and the dilation of the specimen is restrained in the shear zone, while the outer zones did not change through the test, increasing coarse fraction normally leads to increase shear strength, in addition to increasing elongation or decreasing convexity of the coarse fraction increases the angle of internal friction angle, the overall roughness of the shear surface state is negatively related to particle smoothness (convexity) and positively related to the area of the shear surface which occupied by particles with special shapes. [6] Studied the influence of particle size distribution on shear strength of accumulation soil, a series of direct shear tests and triaxial tests were carried out to understand the shear strength of the accumulation soil, results from the direct shear tests showed that the range of the internal friction angle of the soil is 33.5–54.6, and those from the triaxial tests showed that the angle is 37.2–50.7, the properties of the soil, such as median particle diameter, coefficient of uniformity, and gravel content, were used to analyze the effects, the angle of shearing resistance is generally increasing with increasing median particle diameter and gravel content and decreasing with increasing coefficient of uniformity. Alias et al [7], studied the direct shear test behavior of a granular soil passing the sieves with opening size of 2.36 mm, it was found that if the particle size increases, peak and residual shear strength increases as well.

1.1.4 Soil Stabilization by Natural Fiber

Every year, farmers harvest around 35 million tons of natural fibers from a wide range of plants and animals [8], a big part of those fiber uses in the industries, the remaining probably use as a waste materials. Natural fiber is sustainable material and suitable economically, that's why it recently wide used. According to Wikipedia [9], fibers are generally consist of two main types natural fiber; comes from animal and plant, and man-made fiber; which is the synthetic fibers and regenerated fibers. Natural fiber has a bunch of different types, usually it classifies as seed fiber, leaf fiber, bast fiber, skin fiber and finally stalk fiber. Sawdust belongs to the last type of fibers (stalk fiber). Sawdust is a name for the materials that can be produced after grinding of the wood, it is formed in a different shapes and sizes depending on the method of cutting, the amount of sawdust generated every year constitutes up to 10-13% of the total volume of wood. Very little information about the usage of sawdust as additive with soils from the engineering view is available in the literature. It can be said that nobody has not been studied the effect of sawdust on the geotechnical properties of course soils and as well the fine soil as an additives to find out how the shear strength might behave. Type of soils, length of the used fibers, its type, as well as the exceedance of water content are effected of the shear strength and the behavior of compaction parameters. Addition of the natural fibers into the soil generally increase the shear strength. Maher and Ho [10], found that unconfined strength increased because of the usage of natural fiber, same conclusion was figured out by Marandi et al [11] with the addition of palm fiber to a silty sand soil. Prabakar and Sridhar [12], also noticed the deviatory stress was improved significantly due to sisal fiber insulation. Coir fiber increased both of the unconfined strength and deviatory stress from the triaxial test with three different soil have different properties, two of them were low plasticity silt [13]. On the other hand, compaction parameters' behavior is not clear enough with the natural fiber addition. Optimum moisture content was increased with the addition of the natural fiber, this was confirmed by Maher and Ho [10] and [11]. While Prabakar and Sridhar [12] published a paper showed that optimum moisture content was decreased with insulation of sisal fiber. The maximum dry density decreased with fiber addition [11] and [12]. Sometimes maximum dry unit weight does not affected with the exceedance of the fiber, according to Maher and Ho [10]. Another study was conducted the interaction between crushed dry leaves and passed on sieves No.40 by Abdel Nafii [14], soil was mixed with the amount (5, 10, and 15) % from dry weight of soil, then, samples engineering properties of the samples were indicated, the study concluded that the presence of organic matters can cause instability in soil properties generally, so that its decrease the plasticity (more than 35%), it has been observed that high organic matters cause decrease in shear strength (more than 50% decreasing in cohesion at 15% organic content) and considerably increase in the optimum moisture content (25%) with decrease of the dry unit weight. Khan S and Khan H [15], found out that the dry density of the soil was improved by 7.8%, permeability reduced by 71.8% and shrinkage limit was increased. Further, the angle of internal friction was increased by 22.14% with adding of 12% sawdust ash and shear strength parameters were also improved significantly. Overall, the sawdust ash had positive effect on geotechnical properties of soil and it can be used as admixture in soil. Sawdust ash used as additive in the soil which used in the highway pavement, tests that were performed on 3 samples, A, B and C, compaction, consistency limits, unconfined compressive strength and finally shear strength. These tests were conducted at both row soil and stabilized states by adding 2, 4, 6, 8 and 10% of saw dust ash. The results showed that saw dust ash has improved the geotechnical properties of the soil samples, maximum dry density increases from 1403 to 1456 kg/m^3 and 1730 to 1785 kg/m^3, also optimum moisture content increases from 23.6 to 28.2% and 26.2 to 29.2%, the same behavior was observed with the unconfined compressive strength, the shear strength was increased - from 101.4 to 142.14 and 154.97, also shear strength - from 50.92 to 71.07 kN/m^2 and 77.49 to 105.99 kN/m^2 for samples A and B, respectively, it was concluded that the sawdust ash can be an effective stabilizer for lateritic soils [16]. Similarly Koteswara et al [17] used sawdust as a partial replacement of the soil, it was found that the liquid limit of the marine clay has been decreased by 15.43% on addition of 15% Sawdust and it has been further decreased by 27.50% when 4% lime is added, the plastic limit of the marine clay has been also increased by 11.50, the optimum moisture content of the marine clay has been decreased by 15.37% on addition of 15% Sawdust and it has been further decreased by 17.91% when 4% lime is added, and they found that the maximum dry density of the marine clay has been improved by 1.96% on addition of 15% sawdust and it has been improved by 1.10%

when 4% lime is added. While maximum dry density showed slight decrease due to induction of natural fiber but optimum moisture content (OMC) increased with increasing in natural fiber content was a concluded by Chegenizadeh and Nikraz [18]. Harianto et al [19] the addition of polypropylene fiber reduces optimum moisture content when maximum dry density were increased, the same researchers increased fiber content for the same soil with the same condition let to increased optimum water content followed by decrease maximum dry unit weight as shown in Table 1.1

Table 1.1 Obtained maximum dry density and optimum water content at various natural fiber contents [19]

Fiber content (%)	Optimum moisture content (%)	Max. dry unit weight (kN/m^3)
0.0	78	8.13
0.2	74	8.19
0.4	73	8.27
0.6	69.3	8.58
0.8	68.2	8.73
1.0	65	9.03
1.2	70.8	8.42

The randomly distributed polypropylene fiber has an effect on the engineering properties of the soil, Subasis et al [20], mixed the polypropylene fiber with optimum percentage of Rice husk ash (RHA) and lime, the engineering properties were determined as an optimum moisture content (OMC), maximum dry density (MDD) and Unconfined compressive strength (UCS), the results of the tests have indicated a significant improvement in these properties, for best utilization effect the optimum percentage of Rice husk ash RHA is 10%, lime is 4% and polypropylene fiber is 1.5.

Sawdust ash was used as additive, but this time for concrete as a partial replacement, it shows that the sawdust ash can behave like the pozzolanic materials in the concrete, it reduces the compressive strength of the concrete [21].

1.1.5 Cementitious Stabilization and Soil Improvement

Cementitious improvement is that kind of soil improvement which can be done by using either cement or another materials which might provide a cementitious ability (supplementary cementitious materials) SCM. SCMs are mixtures of pozzolanic materials such as lime, blast furnace slay or fly ash [22], the uses of cementitious stabilization include strengthening of existing pavements and soils, improving low quality to make suitable for sub base or base, reducing the need to increase the base thickness to achieve design strength and drying out of wet pavements [4].

Lime stabilization is one of the most effective methods to improve the geotechnical properties of soils. The basic idea behind the addition is that limited addition of lime for soils generally improve their properties for construction purposes [23]. Problematic soft soils can stabilize and modify their engineering properties by increasing the workability, decreasing plasticity index, and shrinkage limit, eliminating swelling properties, increasing California Bearing Ratio (CBR) and strength of soils as well as increasing permeability of soils [24]. With lime addition, the plasticity of Montmorillonite was reduced while the plasticity of quartz and kaolinite was increased somewhat. However, the addition of lime had little effect on the plasticity but also has a significant reduction occurred in that of the laminated clay, all materials showed an increase in their optimum water content and a decrease in their maximum dry density, during lime addition, the strength was increased somehow and Young's Modulus occurred in these materials when they have treated with the lime, the period of curing and temperature at the curing place had an important effect on the development of strength [23].

Zhang et al [25] modified high liquid limit clay by quick lime and concluded that the optimum water content of the modified clay increases nonlinearly with increasing mix-ratio of quick lime, on the other hand, the maximum dry density decreases with increasing mix-ratio of quick lime also the liquid limit and plasticity index of the modified clay decrease with increasing mix-ratio of quick lime and time, but its plastic limit increases with increasing mix-ratio of quick lime and time. Harichane et al [26], studied the effect of using lime plus natural pozzolana or a combination of both of them on the geotechnical properties of two cohesive soils, lime or natural pozzolana was added to these soils at ranges of 0-8% and 0-20%, respectively. In the same study, combinations of lime-natural pozzolana added at the same ranges, test samples were compacted and shear tests were conducted, the samples were cured for different periods which are 1, 7, 28 and 90 days then, they were tested for shear strength tests, according to the experimental results, it was seen that the combination lime natural pozzolana showed a notable improvement of the cohesion and internal friction angle with curing period and particularly at later ages for each single soil. The hydrated lime in clean sand and sand clay specimens was also studied. In order to illustrate the improvement of shear strength behavior of soil by hydrated lime addition, kaolin clay at different percentages is added in soil specimens containing a sand from mining with 5% of hydrated lime because lime has a good reaction with clayey soil. It is observed that lime increasing the strength of clayey soil due to the reaction with the clay, lime looks like to have little influence on the clean sand and soil specimens containing less than 10% of kaolin clay, it was interesting to conclude that 30% clay content in treated (5% of hydrated lime) soil specimen is needed to increase the deviatoric stress of soil as compared to row soil specimen with the same clay content. On the other hand there are a lot of researchers studied the effect of lime on the behavior of different kind of soil, it uses to reduce the brittleness of soil stabilized [27].

Majeed et al [28], studied the effect of lime produced in Karbala's factory on the geotechnical characteristics of a clayey soil, they have used different percentages of lime (2, 4, 6, 8), The unconfined compression increased, best curing time was 30 days with 6% lime content, there were also some trials to study the effect of polymers- fiber mixtures on the engineering properties of a clayey soil, nine groups of stabilized soil samples were prepared and tested with three different amounts of fiber content (i.e. 0.05%, 0.15%, 0.25% by weight of the soil) and three different amounts of lime (i.e. 2%, 5%, 8% by weight of the soil). These treated samples were tested by unconfined compression, and direct shear test, through scanning electron microscopy (SEM) analysis of the specimens after shearing, the improving mechanisms of polypropylene fiber and lime in the soil were discussed and the observed test results were explained, it was found that fiber content, lime content and curing duration had significant influence on the engineering properties of the fiber–lime treated soil, an increase in lime content resulted in an initial increase followed by a slight decrease in unconfined compressive strength, cohesion and angle of internal friction of the clayey soil; On the other hand, an increase in lime content led to a reduction of swelling and shrinkage potential. However, an increase in fiber content caused an increase in strength and shrinkage potential but brought on the reduction of swelling potential, an increase in curing duration improved the unconfined compressive strength and shear strength parameters of the stabilized soil significantly. Based on the SEM analysis, it was found that the presence of fiber contributed to physical interaction between fiber and soil whereas the use of lime produced chemical reaction between lime and soil and changed soil fabric significantly [29]. Saranya and Muttharam [30] carried out different laboratory tests such as vane shear test, unconfined compressive strength test and triaxial test to measure the angle of internal friction as well as the undrained cohesion of the soil, generally the angle of internal friction and the cohesion of the soil were increased with increasing the content of lime and curing periods. However for a specific lime content and curing time, increasing the consolidation stress increases the undrained shear strength while it decreases the internal angle of friction decreases as consolidation stress. Dafalla et al [31] investigated the variation in strength properties for the first day and seventh day curing of lime an expansive soil treated with lime from Al-Qatif in Saudi Arabia, the difference which might happen on the geotechnical characteristics due to lime usage were studied, for clays treated with 4% and 8% lime, the tendency of stress-strain relationship were noted and variations in initial tangent modulus and secant tangent modulus were also reported for different lime content and a different curing time, the results of the study is helpful in estimate the development in strength within stabilized expansive soils and help geotechnical designers evaluate the efficiency of the stabilization.

The long term eligibility of the treatment of the expansive soil using lime was studied by Khattab et al [32] all the tests were carried out on samples compacted at optimum water content and maximum dry density conditions to study the possible usage of a typical expansive soil to climatic changes for road construction, or other civil engineering projects, the treatment of the soil with 4% lime results in a total improvement of most of the geotechnical properties of the expansive clay by increasing the strength. stabilized soil using lime, by adding different percentages of limes (1%, 3%, 5% and 7%) and over different curing periods (7, 15, 30 and 45 day), according to the results of conducted tests on these stabilized soils (unconfined compression test, compaction test and consistency limits) it was concluded that 7% was the optimum lime percent and curing time is to be 45 days, there were attempts to use a combination between lime and other materials, several projects have used various Lime-Fly Ash (LFA) ratios for soil stabilization [33], Pavement engineers recognize the long term benefits of increasing the strength and durability of subgrade soil by mixing in a cementitious binder, this is carried out during reconstruction or in new construction, Beeghly **[**34], stated that stabilizing the soil can result in the reduction of pavement thickness of other layers. the researcher describes that in one case, approximately 5 inches of bituminous base course and 2 inches of granular crushed stone base was eliminated from the design, recent investigations and practice has shown that soil stabilized with lime and Class F fly ash can be economically engineered for long-term performance, the same researcher asserts that for appropriate soils, Lime-Fly Ash (LFA) can offer cost savings by decreasing material cost by up to 50% in comparison with Portland cement stabilization. [22], stated that for fly ash, a ratio of about one part of lime to two parts of fly ash by volume will produce maximum strength of the paste. Literature has also found that other state departments have also used a 1:2 proportion of LFA thus, based on literature, a ratio of 1:2 lime and fly ash will be used for the current analysis**.** Abass [35], had written a paper about an experimental program was carried out to study the effect of the engineering properties of a kaolin clayey soils when mixed with lime and Silica Fume. A series of laboratory tests have been conducted for varieties of samples (2.5,5,7.5 and 10)% for lime and (2, 4 and 6)% for silica fume. These experiments are consistency limits, specific gravity, compaction, unconfined compression test and California Bearing Ratio (CBR) test. For each test, the optimum lime content and optimum silica fume content as well as optimum mixture content (lime+silica fume) were obtained. The results imply that the optimum percentage of lime-silica fume combination was attained at a (2.5%lime+6.0%silica fume), which worked as controlling data in the study, the optimum percentage decrease the, specific gravity, liquid limit, plasticity index, and maximum dry unit weight as well. On the other hand it increased the optimum moisture content (OMC), unconfined compressive strength (UCS) and California Bearing Ratio (CBR). These results showed also, that the mixtures of Lime-silica fume stabilization at (2.5% lime+6.0% silica fume) is better than the properties which achieved by Lime alone: 2.5%lime for plasticity index, 10.0% lime for specific gravity, maximum dry density and optimum water content, 5.0% lime for unconfined compression stress and 7.5% for California Bearing Ratio, all of these results showed that the engineering properties of clayey soils can be enhanced, by mixing lime and silica fume together.

Physical characteristics of Goderville silt were tested by Okyay and Dias [36], different amounts of lime and cement were added to the soil, it compacted at the optimum water content. Stabilized samples were tested at (7, 28, 90, and 350) days after the curing to see the progress of mechanical resistance over time. Finally, numerical analysis were made on the lab results in order to understand the mechanisms of load transfer in the piled earth platforms depending on the treated soil, pile spacing and height of earth platform. Pandey et al [37], used lime, jute, fly ash and water proofing compounds for improving the physical characteristics of black cotton soil. A series of Compaction tests and California Bearing Ratio tests have been conducted as well as Atterberg Limits were carried out on soil mixed with jute fiber of various diameters (2mm to 8mm) and lengths (0.5mm to 2.0 mm) in different amounts (0.2% to 1.0%) to know the optimum amount and also with different quantity of fly ash (10, 15, 20 and 25) % and lime (1 to 5) %. It was figured out that mixing of (1%) jute fiber, (20%) fly ash and (5%) lime together in a soil give better result as compare to adding those materials individually for soil improvement, and it also reduces the cost of road near about 50-60% and increases the CBR value near about 18-20 times. Rice husk ash (RHA) an agricultural waste can be effectively used for stabilization of soils using cement or lime as additive. Rice husk ash is source of silica has numerous applications in silicon based industries. Addition of RHA to the soil in general increases optimum moisture content and reduces the maximum dry density. RHA and lime/cement improves plasticity index and swelling potential of expansive soils. Using waste tyres in geotechnical engineering applications may be feasible to consume the scrap tyres. Waste tyre can use for improvement of bearing capacity soil up to optimum rubber content and tyre waste can effectively use as soil reinforcement beneath footing, embankment and retaining wall. Findings lead to overall saving in soil material costs and recycling of tyres waste and RHA waste [38]. Silica fume increases liquid limits, plastic limits and plasticity index by 1.8 times and decreased the specific gravity in all clay samples by 4%. Silica fume increased the optimum water content and decreased the maximum dry unit weights of the samples by about 31% for all the composite samples in the same compaction effort [39]. Fly ash alone is added to the soil, it increased stiffness and peak strength of the soil whereas the ductility was reduced, the curing has no big effect on shear strength parameters because fly ash has not a cementitious property [40]. Generally specific gravity, plasticity index and linear shrinkage decrease in the mixtures of soil fly ash, samples whatever was the type of fly ash [41], values of unconfined compression strength gradually decrease of ash regardless the type, but as a long term especially at the 7th day UCS values are showing increasing more than 1st day values. The stabilization soils by cement is a technique has also been found to improve the fine soils, Yang [42], discussed geotechnical properties of soil-cement the results demonstrated that the cement content mainly effects the compressive strength of soil cement, age and moisture content also have an influence on the strength. As the cement content increased, the compressive strength of soil-cement increased, with the age the strength is growth and increase, with increase of the moisture content of the soil sample lowers quickly. The relationship between the stress and the strain is not linear, showing the elastic-plastic material properties. Besides, as increasing of compressive strength, the deformation modulus, also tensile strength and shear strength increased

1.2 Objective of the Thesis

The basic objectives of this thesis are to evaluate the effect of adding the sawdust and lime on the shear strength behavior of a clayey silt soil passed from sieve No. 40.

In addition to the effects of mentioned additives on the maximum dry density and optimum moisture content and also some geotechnical properties of a clayey silt soil including consistency limits.

1.3 Hypothesis and Layout of the Thesis

The weakness of some kinds of soils is a serious problem causes instability for the buildings and other structures above the soil. Increasing the bearing capacity will lead to increase the stability. Sawdust and lime usage improving fine soil and make them able to sustain more loads. The present thesis consists of five chapters. The first one gives a review of previous studies about shear strength of the soil besides the factors that effect on the characteristics shear strength behavior for both of sawdust and lime. Chapter two talks about the background of the study and brief explanation about the used tests

Chapter three includes the materials, test methods and test procedures used in this research. The performed, Physical and chemical tests.

Chapter four includes the results and discussion of the physical and chemical properties for prepared and treated soils. In addition this chapter includes a comparison between the results obtained using the treated and untreated samples.

Chapter five includes the conclusions drawn in this work with the recommendations for future works.

CHAPTER 2

GENERAL INFORMATION

2.1 Atterberg Limits

2.1.1 Liquid Limit:

It is the moisture content of the soil which lies between the liquid and plastic states. Generally, it defined as the minimum water content required to close a 13 mm groove by applying 25 blows. Measured by the Casagrande's apparatus. It is denoted by LL. Figure 2.1 shows the Casagrande's apparatus.

Figure 2.1 the Casagrande's apparatus. [2]

2.1.2 Plastic Limit:

It is denoted by PL, it is the moisture content lies between plastic state and semi solid state in the soil, it can be determined by rolling the soil on a non-porous surface when it start to get cracks at approximately 3mm in diameter. Figure 2.2

Figure 2.2 Soil at plastic limit [1]

2.2 Particle Size Distribution

Everywhere, soils consist of wide range of particles with different shapes and sizes, it depends on the way of the formation. Soil particles have different ranges, it might distribute from microns up to several centimeters. The gain size distribution plays an important role in the physical properties such as permeability and the density of the soil, as well it effects in the shear strength.

Two different lab experiments usually done to determine that distribution, sieve analysis which conduct for the soils which has larger than 75 μm in diameter, it can be done by passing the soil on the standard sieves, No.4,10,20,40,60 ,100 and 200. On the other hand, the soil passed from sieve No.200 consider either clay or silt soil. The hydrometer uses to know that. Soils many times classified under three categories, well graded, uniformly graded and gap graded. Well graded soils have an enough amount of particles on each single mentioned sieve. On the other hand, soils considers as uniformly graded if it has all of the sizes but there is a notable change in the amount of the soil in one or more sieves. Sometimes the gap graded soils result because of the absence of some particles, one or more of the sieves has no particles.

Figure 2.3 (A) Stack of sieves, (B) The Hydrometer [1]

2.3 Specific Gravity

It is a number given to each different soil with respect to the specific gravity of the water, in other words it defined as the ratio between mass of the soil with known volume divided by the mass of water with equal volume. The specific gravity depends on the type of the soil, general range for specific gravity of soils:

Table 2.1 typical values if specific gravity for different soils [43]

Type of the soil	Specific gravity
Sand	2.63-2.67
Silt	2.65-2.7
Clay and silty clay	2.67-2.9
Organic soil	<2.0

2.4 Compaction

Compaction is the oldest process to improve the bearing capacity of soils under foundation, and even today, it still the most economical and applicable method. The general idea behind the compaction process is to increase the density of soils. In spite of the real increase in density might be only between five to ten percent Lin [44], it increases the properties of soils such as compressibility, coefficient of permeability and the angle of internal friction depend only on the degree of compaction. In 1933, Procter gave the basic idea and explained the principles of the compaction procedure, Procter showed that reducing the void ratio could happen in different ways such as reorientation of the particles; breaking of the grains or the bonds between them, then followed by reorientation; and bending or distortion of the particles and their adsorbed layers, energy absorbed in this process is provided by the compactive effort of the compaction device. The efficiency of the applied energy depends mainly on the type of particles of which the fill is composed and on the way of the application of the energy. Increasing the water content of the soil leading to increase the cohesion, the strength becomes less, and the effect of compaction becomes more effective. If the moisture content is very high, however, the densification of cohesive soils leads to a high degree of saturation. Lambe [45] explained the engineering behavior and the structure of compacted clays from physico-chemical concepts. The electrical forces between the particles causes an attractive forces between clay particles instead of mass forces because of their small size, the negative charge of clay particles generally attract cations and polar water molecules towards them.

However, Procter's test normally used in the lab, especially standard Procter test to determine the maximum dry unit weight of a soil and optimum water content .In the standard Proctor test, a dry soil specimen is mixed with water and compacted in a cylindrical mold of volume 944 cm^3 (standard Proctor mold) by repeated blows from the mass of a hammer, 2.5 kg, falling freely from a height of 305 mm. The soil is compacted in three layers, each of which is subjected to 25 blows [1].

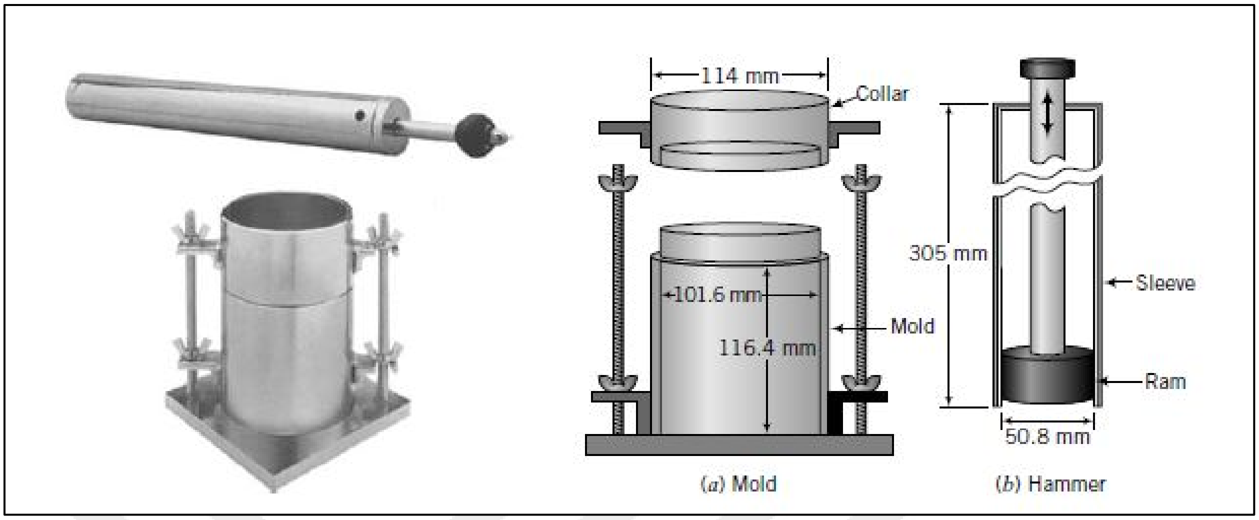

Figure 2.4 Compaction apparatus [1]

2.5 Shear Strength

Shear strength is a very important design parameter. It also known maximum stress can be sustain by any material without failure named as strength. Generally, the strength of the soil can be measured by its ability to transfer loads in form of stresses. The stability of any substructure depends mainly on the strength of the soil.

Here are examples of some types of failure that have happened due to lack of the shear strength in the soil.

Figure 2.5 Liquefaction-induced bearing capacity failure due to lack of shear strength, The City of Adapazari [46]

Figure 2.6 Taiwan landslide slope failure [47]

According to Das and Sohban [2], the failure happens in the soil if shear stresses exceed the shear strength along the failure surface. Soils consist mainly of a collection of grains, the grain slides over each other at the failure, failure can also happens due to failure of individual grains but it is a rare event.

As previously mentioned, the main elements of shear strength are cohesion (c) and internal friction angle (φ). The friction angle is also known as the angle of internal friction or angle of or the shearing resistance. This angle resulting from a relationship between the shearing resistances to the normal stress acting on the surface within a soil mass during the time of shear. The cohesion of the soil is the internal strength between the particles, it is the intermolecular force that holds together the particles within the soil mass [48]. The intermolecular force mainly divided into three groups [49]:

1. Cementations: chemical bonding caused by the presence of cementing agent, such as calcium carbonite (CaCo3) or iron oxide (Fe_2O_3). These forces also can be get from adding cementations materials such as cement, lime or other materials.
2. Electrostatic and electromagnetic attractions hold particles together, these forces are very small and probably it does not have a great effect on the shear strength.
3. Primary valence bonding (adhesion): type of bonding happens to the clays when they become over consolidated.

The cohesive strength generally increased with increasing clay content, thus high clay content soils usually tends to have high cohesive strength and have a good ability for shaping and molding. Most of the fine-grained materials are cohesive and they have a large amount of silt and/or clay. In 1776, Coulomb published the most important contribution to the problems of shear strength, he suggested the equation that combines between angle of internal friction and the cohesion of the soil,

$$\tau_f = c + \sigma_f \tan \varphi \tag{2.1}$$

σ = normal stress

τ_f =shear strength at failure

Therefore, increase of c and σ result in led to increase shear strength and finally it improves the stability. The shear strength of soil can determined by obtaining soils samples from the site, from boreholes or trial pits made by a drilling machine, or it might get from compacting the soil in the lab and getting the samples by Shelby tubes and then tested. These tests include unconsolidated undrained Triaxial and unconfined compression test.

2.5.1 Mohr-Coulomb Failure Criterion

The theory of failure which presented by Mohr and Coulomb used to find out the shear strength of soils and rocks. The theory, at the early beginning, was developed by Mohr in the 1900s to establish a criterion for determining the shear failure for such materials, where the shear failure happens when shear stresses overcome the shear strength along the failure envelope. An example of a shear strength τ to normal stress σ plane is graphically shown in Figure 2.7. The shear strength (τ_f) is a function depends on the normal stress (τ_f) on the failure plane, as shown by the following equation:

$$\tau_f = f(\sigma) \tag{2.2}$$

By getting back to the principle of strength of materials, Mohr circle normally represented by the type of failure of the specimen, where $\sigma>0$ is failure by compression normally happens, when $\sigma<0$ is tension failure occurs. The convention consists of drawing a series of Mohr circles on a ($\tau - \sigma$) plane, where each circle represents stress states at different failures under increasing levels of confining stress. The Mohr-Coulomb failure envelope is presented by drawing a tangent to the Mohr circles. The soil mass is considered stable if all the Mohr circles are found within the stable domain ($\tau < \tau_f$,) which is below the envelope. The plot in Figure 2.7 shows that the two Mohr circles touches the failure envelope at which ($\tau = \tau_f$)where failure occurs at these points.

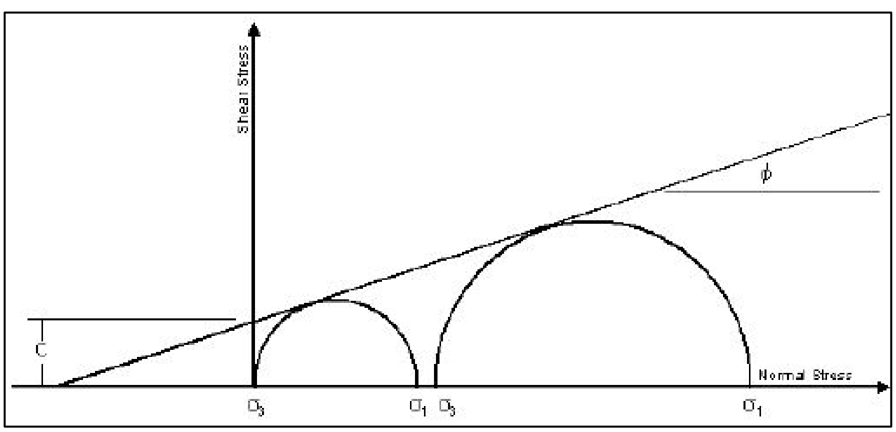

Figure 2.7 failure envelope and Mohr circles [2]

Coulomb related the shear strength parameters, c and φ, as shown in equation 1. These values are determined using the failure envelope where c and φ is the σ -intercept and the angle of the failure envelope, respectively.

Shear strength behavior depends on many factors which are [50],

- Stress history
- Water content
- Degree of saturation
- Soil compaction
- Void ratio
- Soil structures
- Rate of loading
- Drainage conditions
- Isotropic media in the soil

2.5.2 Triaxial Test Method

The machine of the triaxial shear test passed through gradual developed over years until reached to this form, the early machine were originated by Buisman 1924 and Hveem 1934, but the first device was developed was in 1930 by Casagrande, both apparently under the direction of Terzaghi. A consensus developed during the late 1930's that the triaxial device was superior to the direct shear device and that view tends to persist today. The triaxial test is one of the most effective available methods to evaluate the shear strength parameters of soils, The triaxial devices has been used for over 70 years to determine drained shear strength parameters of soils [51]. This stress can be applied in one of two ways:

1. Application of the hydraulic pressure or even the dead weights in equal increments until the specimen fails, the dial gauge measures the axial deformation of the specimen caused from the application of the load through the ram.
2. The strain controlled test can be done by Application of axial deformation at a constant rate by means of a geared or hydraulic loading press [2].

The triaxial device, can be chosen for different reasons,

1- The concentration of the stress caused by restriction of the end can influence the results. Research performed by Taylor in 1941 showed that the effect of the concentration is negligible if the ratio of length to diameter is between 1.5 and 2.5 [2].
2- It gives enough idea about the behavior strain-strain curve under different confining pressure.
3- It provides wide range of loadings and confining pressures. It provides a uniform stress distribution along the samples as well.

The triaxial test has several kinds of principle types:

1- Unconsolidated Undrained : UU
2- Consolidated Undrained : CU
3- Consolidated Drained : CD

The unconsolidated undrained (UU) triaxial test was used to validate the alternative criterion. This type of test means that the drainage of water is not permitted from the pores of the soil sample during the stressing period. This test was selected because it is considered to be faster than the other types to perform. It does not need a lot of experience to do especially for the researchers and the students. The triaxial compression testing apparatus is shown in Figure 2.8 below.

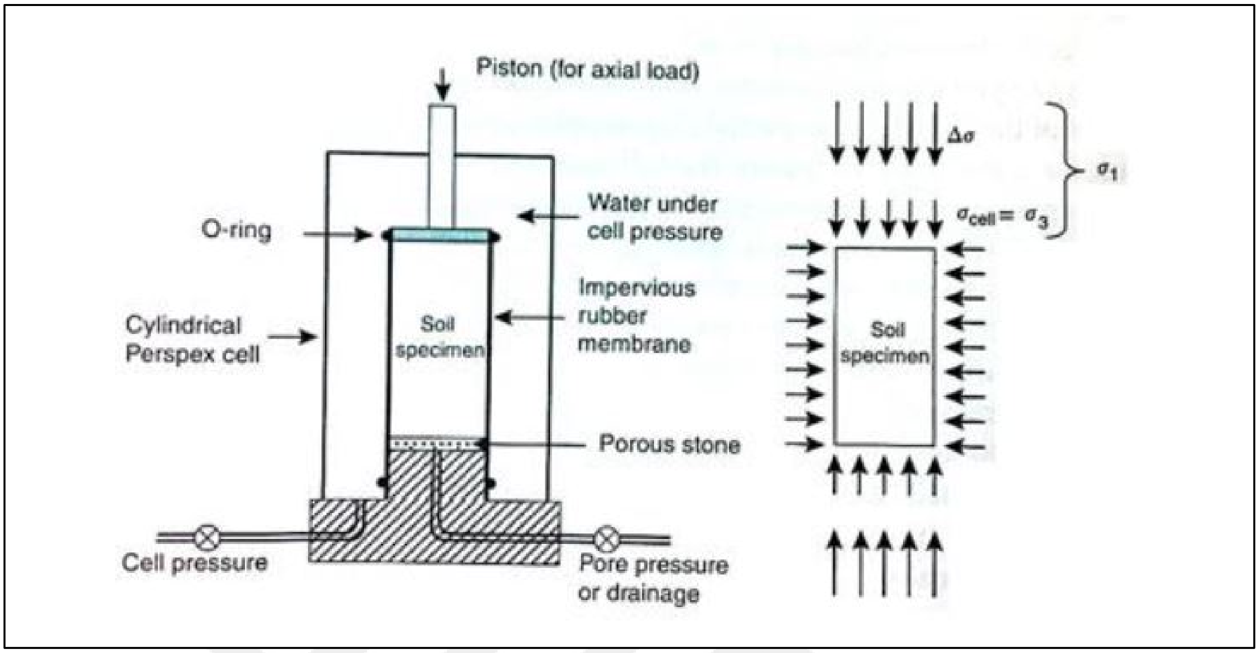

Figure 2.8 Triaxial Cell Apparatus

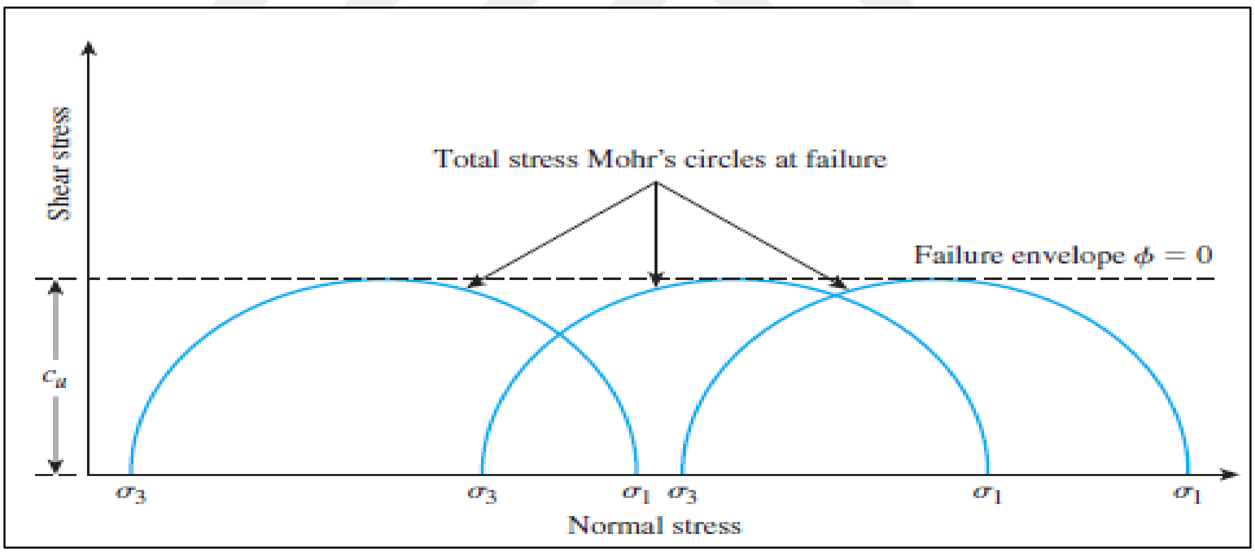

Figure 2.3 Total stress Mohr's circles and failure envelope (φ =0) obtained from unconsolidated-undrained triaxial tests on fully saturated cohesive soil [2]

This test will be carried out in accordance with ASTM D2850-03A, the procedure is to determine the shear strength of the soil specimen in the triaxial compression apparatus under a constant cell pressure, in which there is no change in the water content within the soil specimen. The diameter of the specimen about 36 mm and 76 mm (3 in.) long generally is used. The specimen is cover by a thin rubber membrane and placed inside a

Perspex cell. It is then filled with water (or fluid), the cell pressure applied to the water, in this case the water applies the confining pressure to the specimen inside of the chamber. To cause shear failure in the specimen, an axial stress must apply on the sample (sometimes called deviatoric stress $\Delta\sigma_d$) through a vertical loading ram, at constant rate of strain until the soil specimen fails. In the unconsolidated undrained triaxial test, the drainage is not allowed during the test. This process will guarantee that the specimen will not consolidate during the test, even if a large confining pressure was used. The shear strength of soil is related to the degree of confining it receives from the surrounding soil, where the triaxial test attempts to simulate these stress conditions. After that the data can collected, Mohr circles can be drawn in terms of the total stresses. This allows to draw the failure envelope, and after, c and φ can be determined.

Undergoing the triaxial test with all types of test requires skill and a clear understanding of its limitations. However, it is important that engineers are aware that leakage can often be a problem. Leakage into the specimen causes a change in the volume and resulting in changes to the pore water pressure and effective stresses.

2.5.3 The Unconfined Compression

The scientist C.J.Jenkin was probably the founder of the earliest compression test for soils in Britain, in 1940 Cooling and Golder designed a portable device with a spring with springs of different strengths, it had the same principles with the currently in use device. It was designed to conduct a quick tests in the field immediately after sampling.

The unconfined compression (UCS) test, is one of the most common strength parameters that uses to determine the UCS cohesive soils, similar to the triaxial compression test except for the lack of a confining pressure. It is performed using a soil specimen of similar size. The specimen is placed between two loading platens and then stress is applied to compress the soil. A typical machine used for this test is shown in Figure 2.11. Since there is no confining pressure and no membrane around the specimen, only cohesive soils can be used for this. During a test, a stress-strain curve will be created. It involves applying a uniform vertical normal load P, on the horizontal circular cross sections of a cylindrical specimen, where this load is increased until failure .The UCS is given by the equation the highest stress applied on this curve is defined as the unconfined compressive strength. Plotting this on a Mohr's circle diagram is shown below in Figure 2.10. The undrained shear strength of the soil is simply the unconfined compression strength divided in half. The test is quick and simples because there is no confining pressure around the samples. This test can be carry out according to ASTM D2166.

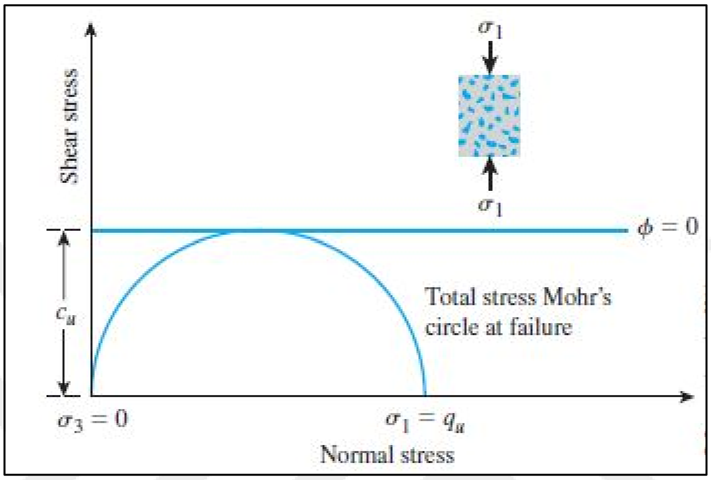

Figure 2.10 Unconfined Compression Test [2]

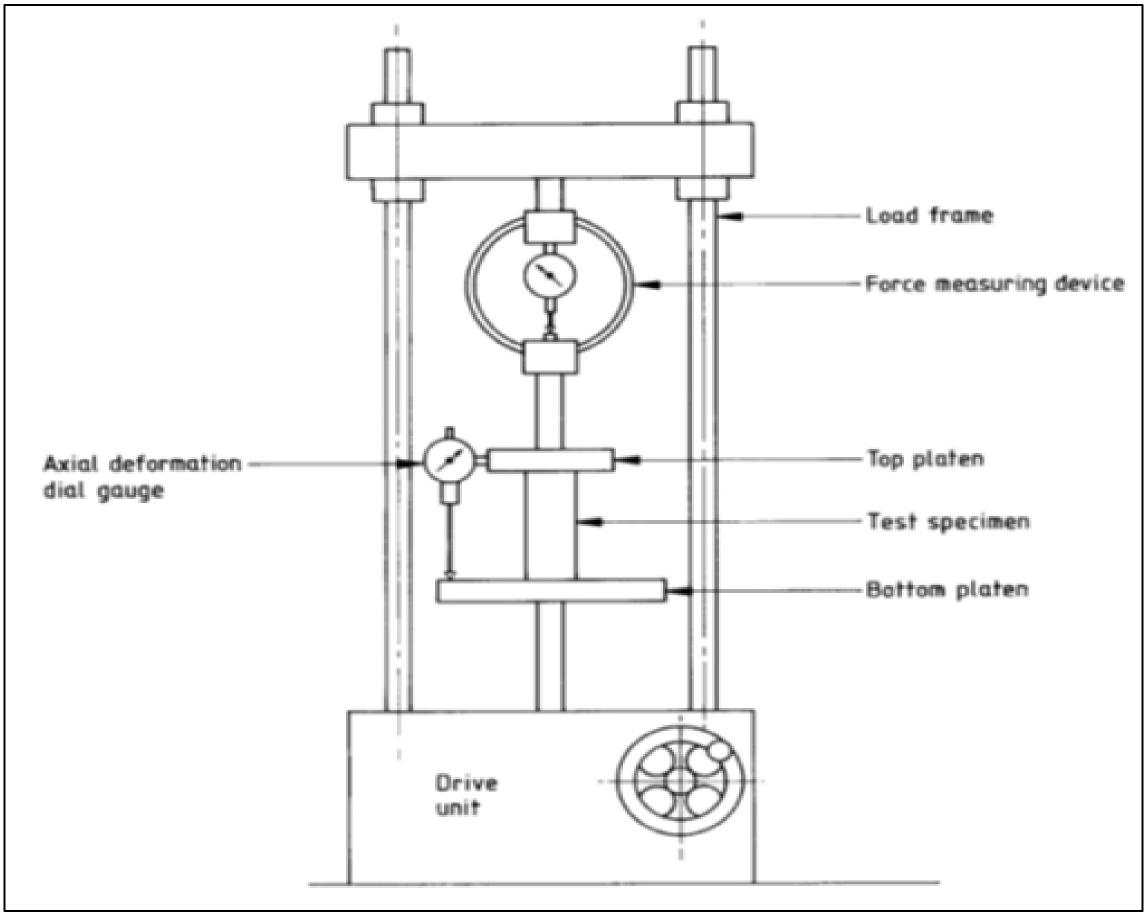

Figure 2.11 Typical Load Frame for Unconfined Compression Strength test [52]

CHAPTER 3

EXPERIMENTAL WORK

3.1 General

The laboratory experiments given in this chapter were conducted to study the effect of sawdust and lime on shear strength, consistency limits and the compaction parameters testing equipments and experimental procedure were discussed. Two series of tests were carried out, the immediate tests include shear strength tests, consistency limits and the compaction while long term tests were done on shear strength samples only. Results are an average of two samples at least in both of immediate and long term tests. Immediate tests conducted after extruding samples from the compaction mold, but long term test conducted after 90 days (2160 hours) to see a clear reaction between the soil and additives.

3.2 Materials

3.2.1 General Description for the Soil

The used soil in this thesis was brought from Büyükçekmece in Istanbul city within the coordinates 41.069987N, 28.604636E as shown in Figure 3.1, the location of the soil is surrounded by the lake of Büyükçekmece from the north as well and an area, where Fatih University lies, to the west and the south. Normally consolidated, and stiff soil at a depth of 8 m below the water level and also below the existing ground level which is the foundation level of the project conducted in the area to be sure that the soil is natural. Based on the result obtained from the basic soil tests soil was classified according to the Unified Soil Classification System as high plasticity silt (MH) and it has a plasticity index 30%. Soil properties were given in Table 3.1.

Figure 43.1 the location of used soil

3.2.1.1 Particle Size Distribution (ASTM D 422-63)

A 500 gm of the soil washed through the sieve No.200 to determine the percentage of sand-clay particles in the specimens, all of the soil have passed through the mentioned sieve. To determine the percentages of silt and clay in the soil an approximately 50 grams of dry soil passes on sieve No. 200 was treated with a dispersing agent for 24 hours. According to results of the Hydrometer analysis, about 75% of the soil is silt and the 25% remaining is clay. Figure 3.2 shows the distribution.

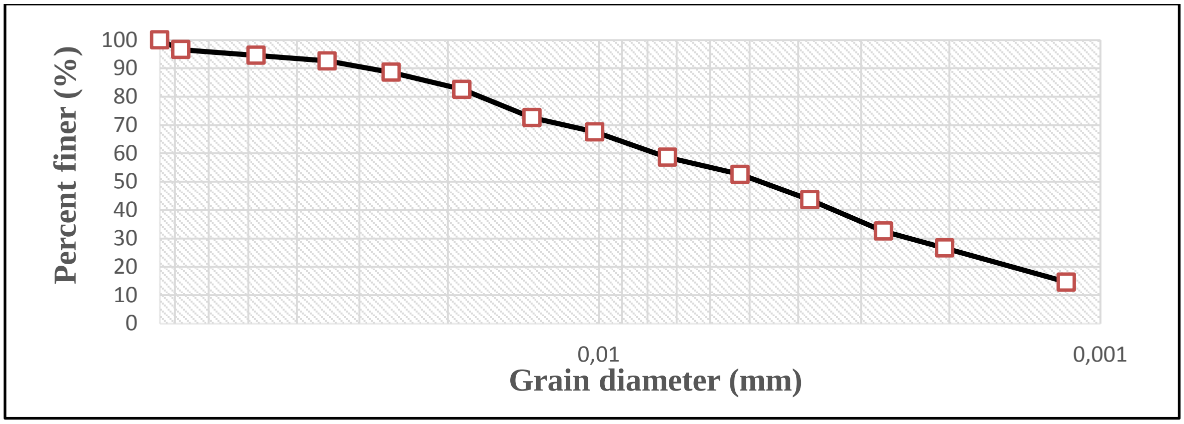

Figure 53.2 Grain size distribution curve of the clayey silt soil

3.2.1.2 Atterberg Limits (ASTM D 4318)

In order to determine the plasticity of the soil, a representative samples were subjected to Atterberg Limits. Generally, the material passing through a 475 μm (No. 40) sieve was used with An Atterberg limits device to determine the liquid limit of the soil. The plastic limit of the soil was determined by using soil passing through a 475 μm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index was then computed for the soil based on the difference between liquid and plastic limit. Obtained liquid limit and plastic limit of the row soil are 73 and 43 respectively and the plasticity index is 30. The liquid limit and plasticity index were then used to classify the soil. As well as, Atterberg Limits have conducted for the soil with different percentages of sawdust and lime respectively. Figure 3.3 shows the flow curve of liquid limit determination for the row soil.

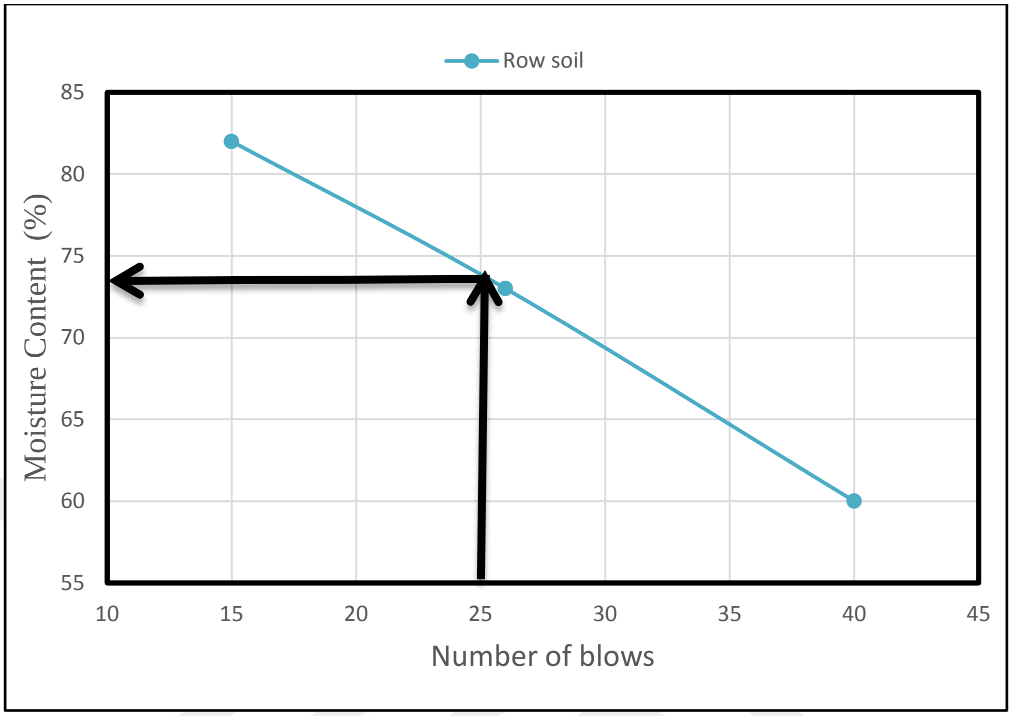

Figure 3.3 Flow curve of liquid limit determination of clayey silt soil

3.2.1.3 Classification (ASTM D 2487-00)

Soil was classified using the Unified Soil Classification System (USCS). Using the particle size distribution and the Atterberg limits, the USCS designates a two letter symbol and a group name for each soil and it founded that the soil is clayey silt soil. As clearly can be seen in figure 3.4, according to values of liquid limit and plasticity index, soil classified as high plasticity silt (MH).

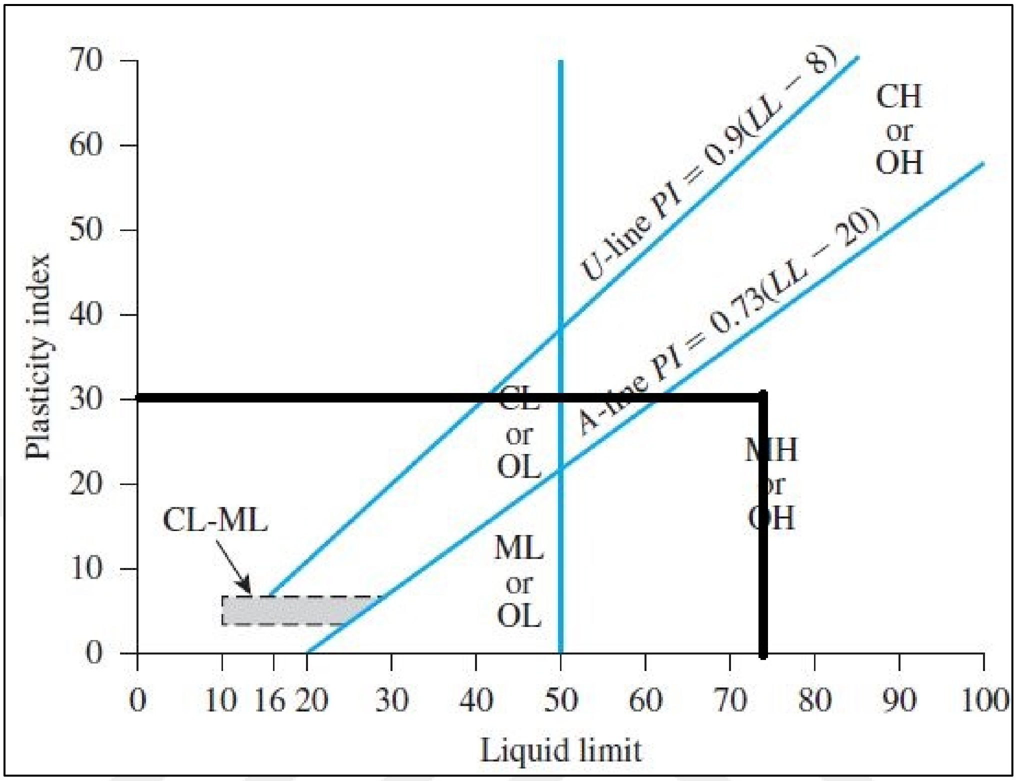

Figure 63.4 Plasticity chart of the soil [2]

3.2.1.4 Specific Gravity (ASTM D 854-02)

The value for specific gravity of the soil solids is an average of two tests, they were determined by grind the soil and pass it on sieve #40, drying the soil (oven dried) and placing a known weight of the soil in a flask, and then the volume was completed by filling the flask with a distilled water. The weight of displaced water was then calculated by comparing the weight of the soil and water in the flask with the weight of flask containing only water. The specific gravity was then calculated by dividing the weight of the dry soil by the weight of the displaced water, 2.65 was the average specific gravity of the soil.

Table 3.1 Soil properties

Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Specific Gravity, GS	Sand Content (%)	Silt Content (%)	Clay Content (%)	Classification According to Unified Classification System
73	43	30	2.65	0	75	25	MH

3.2.1.5 The Chemical Analyze of the Soil

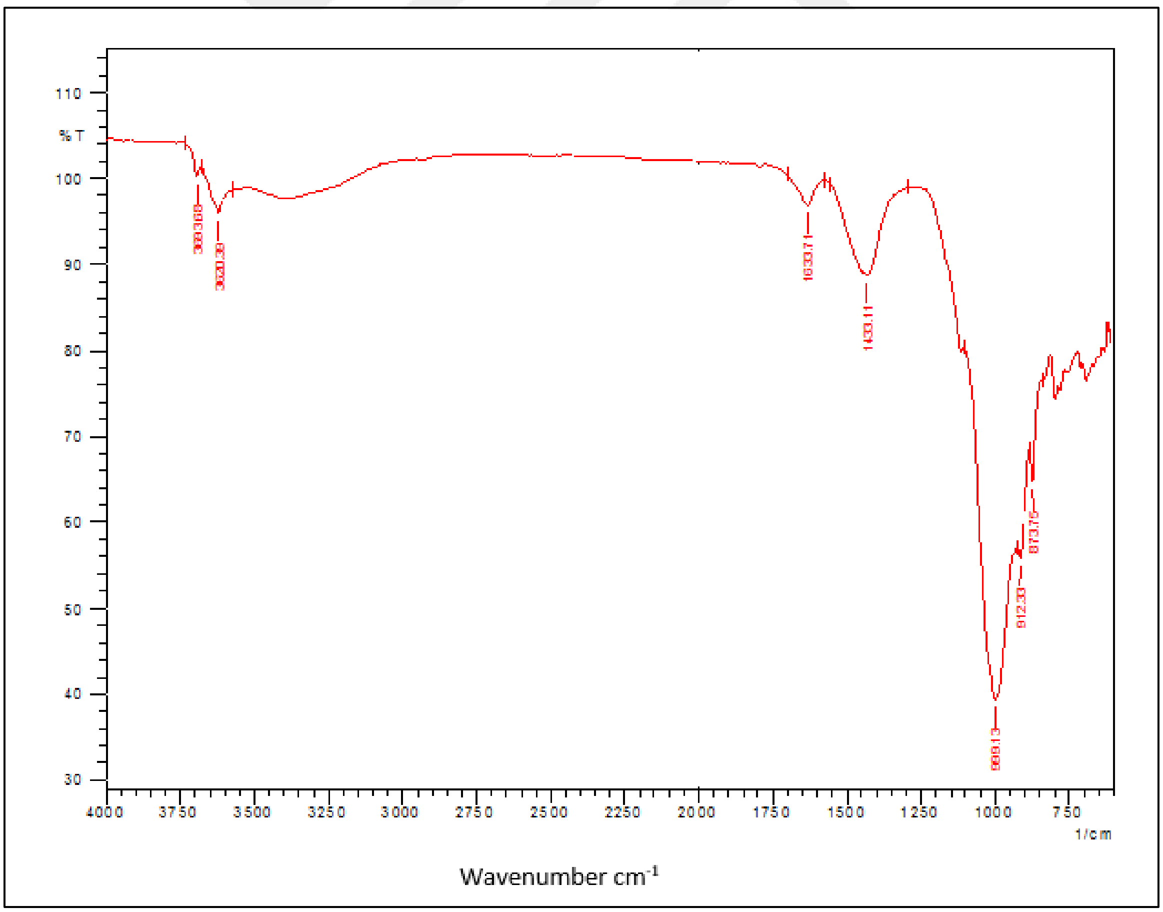

Figure 3.5 Result of FT-IR analyze of soil in ethanol solution

Vertical axis is transmittance in percent. At 3620 wavenumber, the peak indicates the H_2O group. Wavenumber of 999.13 is the Metal Salts (M-X) that may create metal bonds with OH groups. The other wavenumbers have no meaning.

3.2.2 Sawdust

Sawdust is a name can be given for the materials which can be produce after cutting the trees. It has a lot of shapes and sizes depending on the machine and method of cutting. When trees are cut about 22% of its weight convert to sawdust [18]. Sawdust used in this study was obtained from carpenter's facility of Yildiz Technical University in Esenler District of Istanbul.

Figure 3.6 picture of used sawdust

3.2.2.1 Moisture Content of the Sawdust (ASTM D 2216-98 and ASTM D 4643-00)

The oven-drying method was used to determine the moisture contents of the sawdust samples. Known weight specimens obtained from three bags of sawdust and weighed, then oven-dried at 105°C for 24 hours. Samples were reweighed again, and the difference in weight was assumed to be the weight of the evaporated water during drying process, the difference in weight was divided by the weight of the dry soil, giving the water content on a dry weight basis. The average water content of the sawdust was 11.22% at the mixing time.

3.2.2.2 Particle Size Distribution of Sawdust (ASTM D 422-63)

A 3000 gm of the used sawdust were passed through the standard sieves to analysis the grain size distribution of the sawdust, more than 50% of the used sawdust has fine particles, Figure 3.7 shows the distribution.

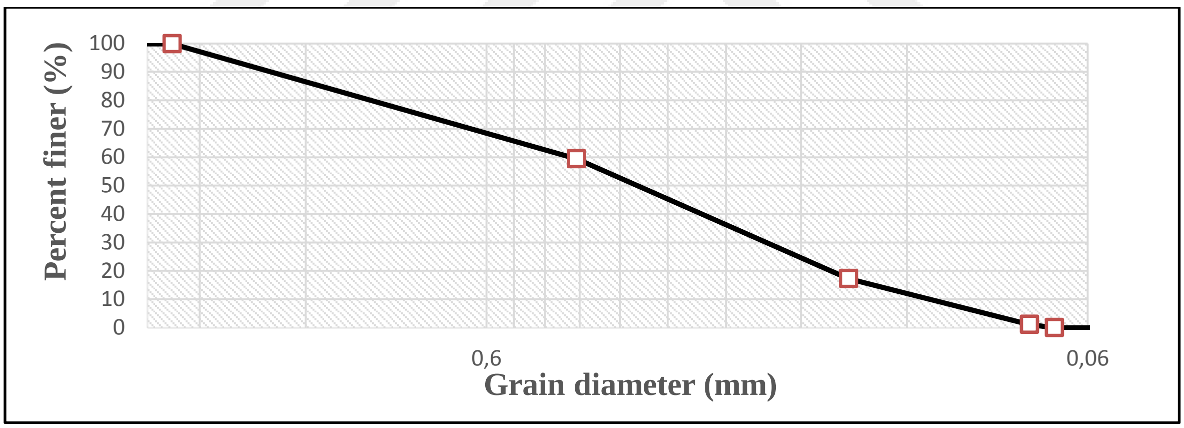

Figure 73.7 the grain size distribution of the sawdust

3.2.2.3 Specific Gravity of the Sawdust (ASTM 2487)

Same procedure was followed to find the average specific gravity of the sawdust, three samples of sawdust were taking, drying them (oven dried) and. Known weight of the samples were placed in a flask, then the volume was completed by filling the flask with a distilled water. The weight of displaced water was then calculated by comparing the weight of the sawdust and water in the flask with the weight of flask containing only water. The specific gravity was then calculated by dividing the weight of the dry sawdust by the weight of the displaced water, 1.31 was the average specific gravity of the soil.

3.2.2.4 The Chemical Analysis of Sawdust

In order to know the chemical composition of sawdust, a chemical analyze were conducted in Chemistry Department of Yildiz Technical University. Dry form of sawdust and sawdust in ethanol solution were analyzed by FT-IR (Fourier Transform Infrared Spectroscopy) method. FT-IR is a technique which is used to obtain an infrared spectrum of absorption or emission of a solid, liquid or gas. An FTIR spectrometer simultaneously collects high spectral resolution data over a wide spectral range. In infrared spectroscopy, IR radiation is passed through a sample. Some of the infrared radiation is absorbed by the sample and some of it is passed through (transmitted). The resulting spectrum represents the molecular absorption and transmission, creating a molecular fingerprint of the sample. Like a fingerprint no two unique molecular structures produce the same infrared spectrum. This makes infrared spectroscopy useful to determine chemical composition of the materials.

Figures 3.8 and 3.9 show the FT-IR results of the sawdust in dry form and in ethanol solution.

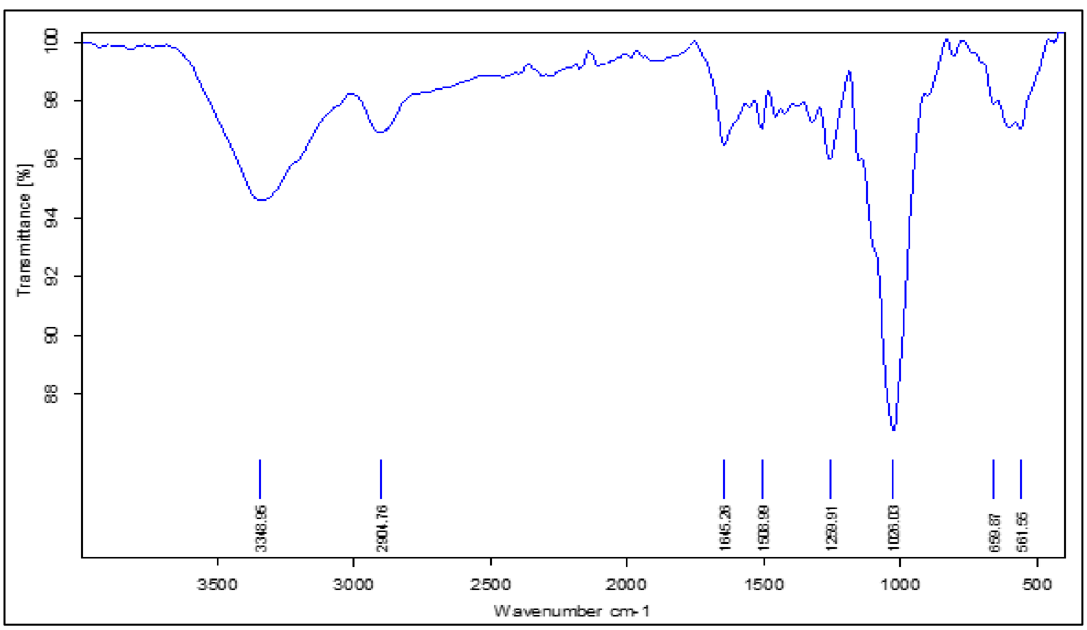

Figure 3.8 Result of FT-IR analyze of sawdust in dry form

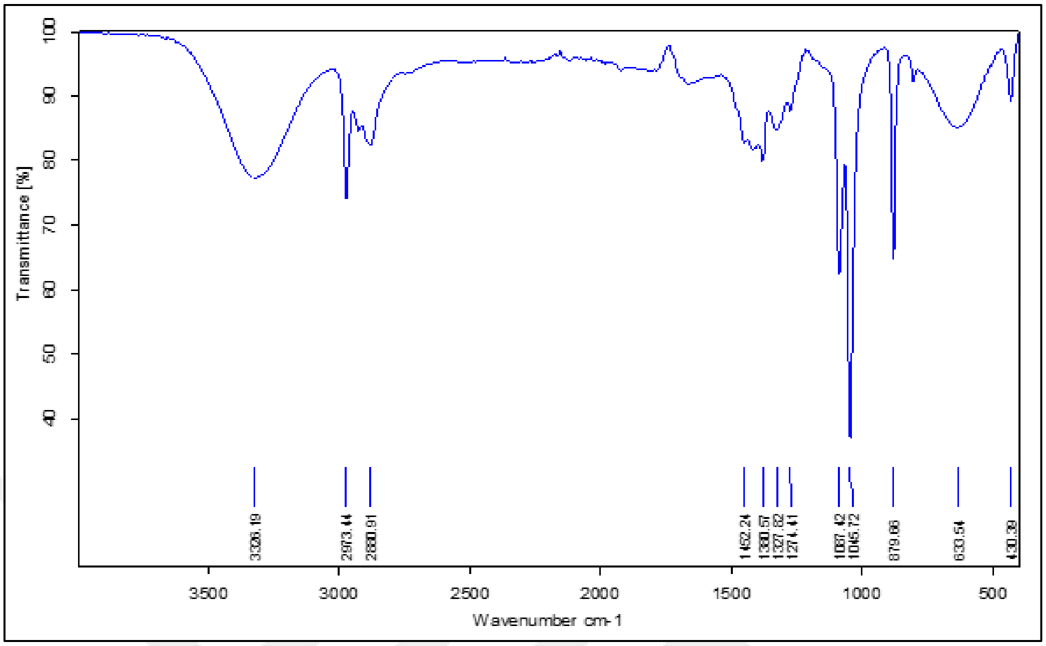

Figure 3.9 Result of FT-IR analyze of sawdust in ethanol solution

Based on the Figure 3.8 and especially Figure 3.9, it can be said that there is hydroxyl group (OH) at 3326 wavenumber that may create hydrogen and metals bonds if soil contains the metal and hydrogen. Between 2973 and 2880 wave numbers, the group is methyl. Between 1452 and 1380 wave numbers, aromatic group (C=C) was detected. Between 1087 and 1045 wave numbers, the ether group (C-O-C) was detected. Ether group is very functional to create the metal bonds. At 879 wave number, halogen group (C-X) was detected, which can create metal bonds. These groups are important in time effect of sawdust.

3.2.3 Lime

The used lime in this thesis were purchased locally as 1x5 kg bags

3.2.3.1 Types of Lime

Many kinds of lime are available in the markets, generally lime classified under two famous types; quicklime (calcium oxide) and hydrated lime (calcium hydroxide).

Calcium Oxide

Calcium oxide (CaO) normally called quicklime or burnt lime, it produces from the thermal decomposition of limestone. Calcium oxide obtained by heating calcium carbonate ($CaCo_3$) with 1250˚ producing calcium oxide. As showing in the equation:

$$CaCo3 \xrightarrow[1250^{\circ}]{\Delta} Cao + Co2$$

The produced calcium oxide is unstable product, it spontaneously reacts with the Carbon dioxide from the air –in case of the long term storage- producing calcium carbonate.

Calcium Hydroxide

Calcium hydroxide sometimes called slaked lime or hydrated lime, its chemical formula $Ca(OH)_2$ shaped in a fine powder form. Calcium hydroxide can be prepared by the hydration process which can be done by mixing the calcium oxide with the water, as shown in the following equation:

$$CaCo_3+H_2O \rightarrow Ca(OH)_2 + Heat$$

A significant amount of heat is released by the process. Unlike Calcium oxide, Calcium hydroxide has lower lime content of lime.

3.2.3.2 Advantages of Lime Usage

The stabilization by lime has a lot of different advantages construction cost can be minimized with better strength as comparing with other different additives, it also improving the strength, reducing the swelling, resist the damage effect due to the moisture, improving resilient properties, and also improving the resistance against fractures, fatigue and permanent [53].

3.3 Soil-Additive Mixture Preparation

The soil passed through sieve No.40 was kept for oven drying at 105°C then it was mixed with the sawdust and the lime with different percentages (1, 2, 3 and 5) % respectively from the dry weight of the soil. The sawdust and the lime were mixed with the soil according to optimum water content for each single mentioned percent. All mixing were done manually, and proper care was taken to prepare homogeneous mixtures at each stage of mixing.

3.4 Preparation of Test Samples

In order to find the effect of sawdust and the lime on a clayey silt soil, it is necessary to fix some of the variables of the row soil to use them as a controlling data, four percentages of sawdust and lime were used, the soil-additives mixture was compacted according to the corresponding optimum moisture content. Each prepared soil sample is mixed thoroughly with the required amount of water, and then compacted. Samples extruded by a hydraulic jack from mold compacted using standard proctor compaction test. Specimens were obtained by exerting a Shelby tube (40mm) in diameter, of cleaned and oiled walls to the samples, the samples length is (100mm).

3.5 Long Term Tests

To investigate the possible chemical reaction, and the interaction between the sawdust and the lime from the other side, and to see the clear effect of the mentioned additives over time on the soil, soil samples were kept in the desiccator, they well covered with polyethylene to make sure that samples do not loss too much of its water content. As shown in Figure 3.10.

Figure 3.10 Keeping soil samples for the long term tests

3.6 The Test Methods

3.6.1 General

Physical and chemical tests have been conducted. The physical tests involve classification, compaction, and shear tests. The chemical test involved the chemical composition of the row soil. The flow chart of the test program is shown in Figure 3.11

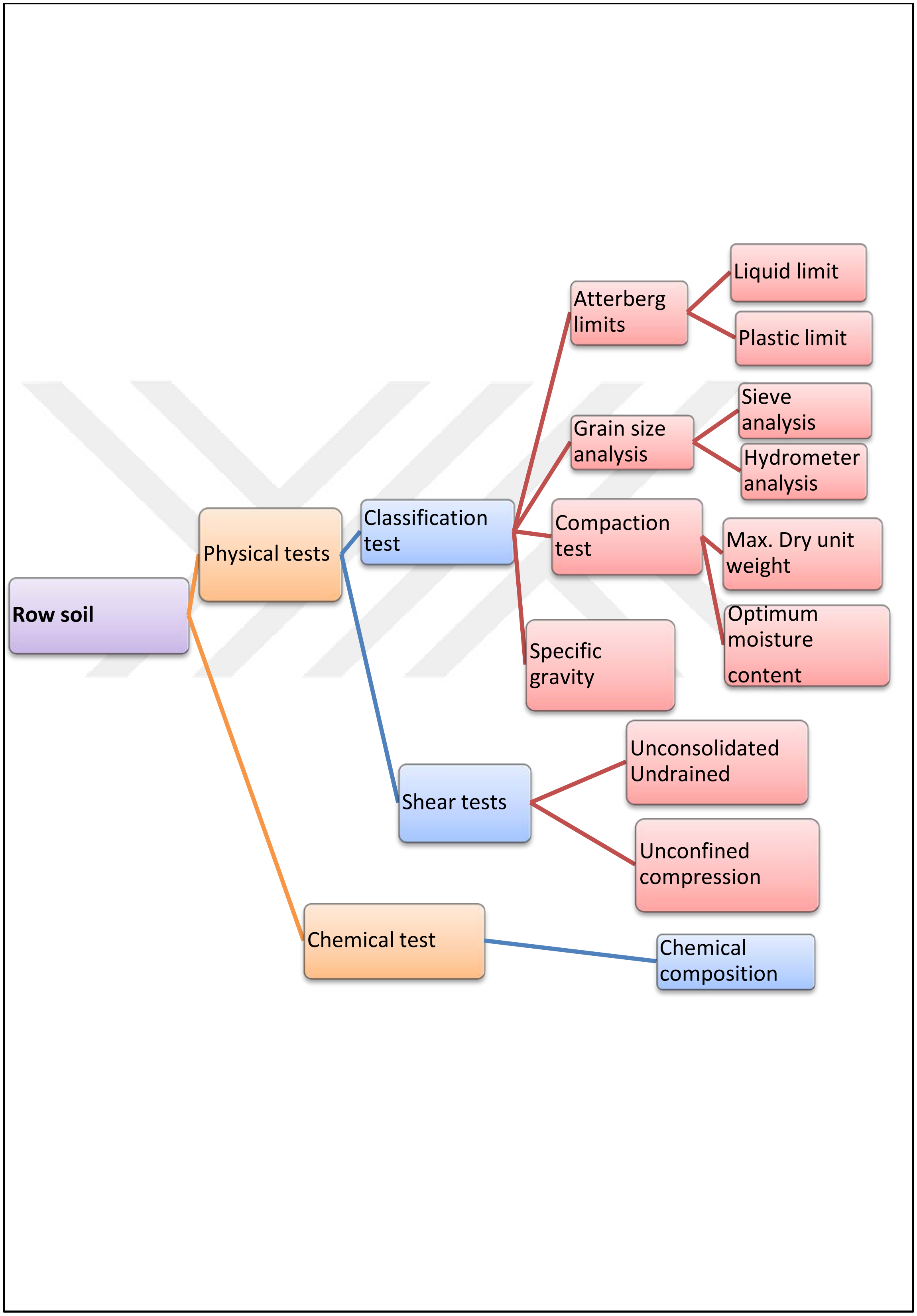

Figure 3.11 Flow chart of the test program

3.6.2 Compaction Test (ASTM D698)

The "Ordinary" compaction test (2.5 kg rammer method) was applied to determine the maximum dry density and the optimum moisture content of the soil. Several samples of the same soil with the same compaction energy, but at different water content, are compacted according to the compaction test specifications. The used soil were passed on sieve No.40 instead of the sieve No. 4.

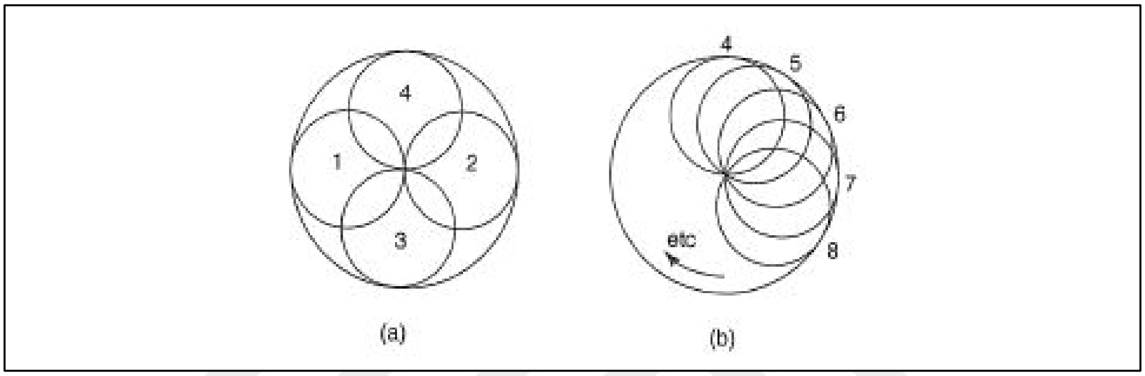

Figure 3.12 Sequence of blows using hand rammer [50]

The total or wet density and the actual water content of each compacted sample are measured. Compacted sample were measured. The soil mixtures, with and without additives, were mixed together carefully. The first group of compaction tests were done to determine the compaction properties of the untreated soils as a control data.

Secondly, tests were carried out to determine the proctor compaction properties of the treated soils with varying amounts of sawdust and lime.

Figure 3.13 shows the dry unit weight-water content relations of the row soil. The maximum dry unit weight and the optimum moisture content for prepared soil are 12.35 kN/m^3 and 40% respectively.

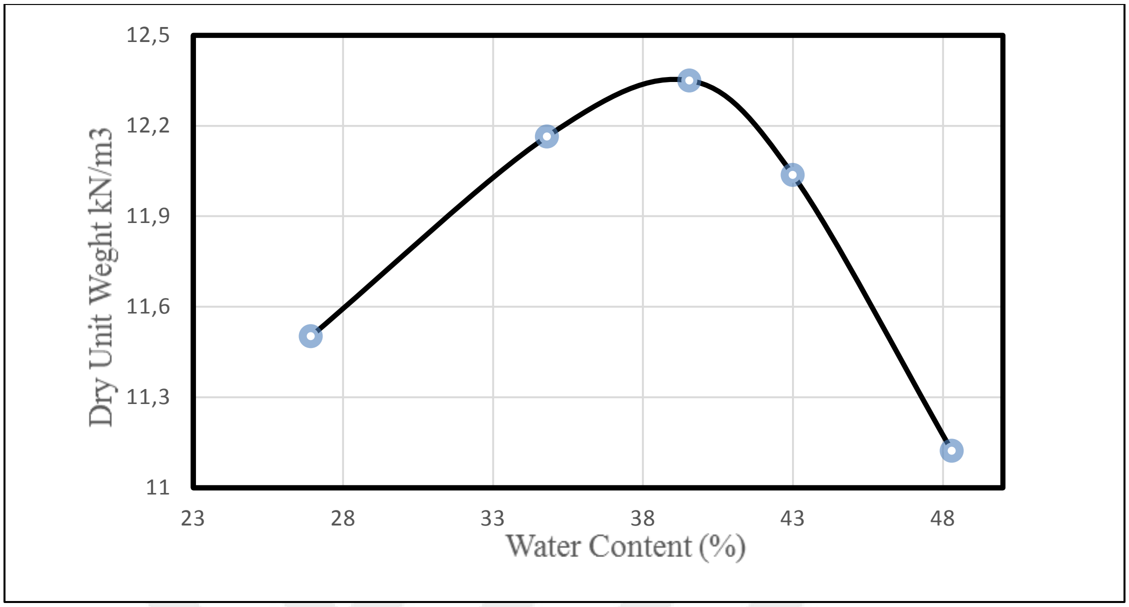

Figure 3.13 Standard Proctor compaction test results for clayey silt soiL

3.6.3 Unconfined Compression test (ASTM D2166)

The unconfined compression device used in the thesis manufactured by ELE International, Loveland, CO. it was capable of testing specimens with diameters as large as 40 mm, the load frame used in this study was an ELE Digital Load Frame capable of delivering a maximum axial load of 50 kN with programmable displacement rates between 0.00001 and 9.99999 mm/min.

The Procedure of The Test

Unconfined compressive strength tests on compacted specimens were conducted. The test conducted by using cylindrical samples with 3.5-4.0cm in diameter with 7.3-8 cm in height as a dimension to satisfy the requirements of ASTM D2166, Length=(2)-(2.5)Diameter.

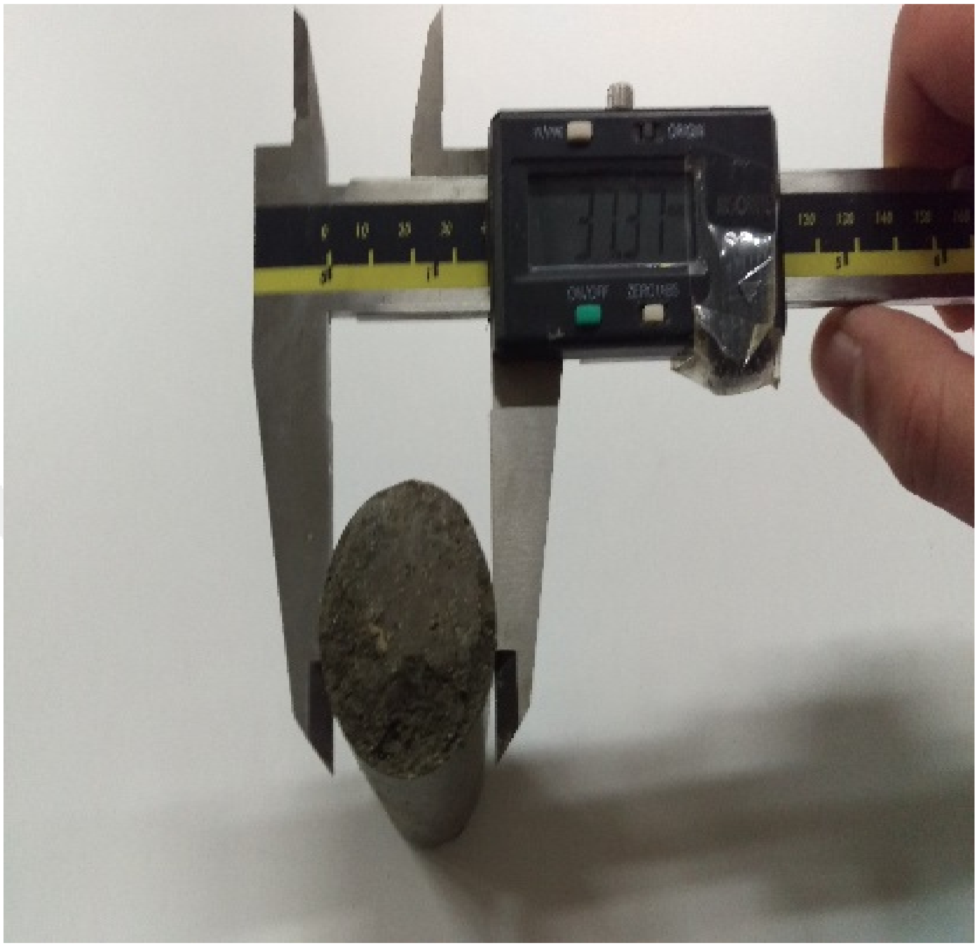

Figure 3.14 The picture shows measuring the diameter of the sample

The diameter of the sample was measured from the upper edge, lower edge and the middle of the sample, the average diameter was chosen during the calculation. Same procedure was applied in case of the length, the used length was the average of three lengths. Figures 3.14 and 3.15.

Figure 3.15 The picture shows measuring the height of the sample

Then, samples were placed in the device between two plates to make sure that the pressure is equally distributed as shown in figure

Figure 3.16 The picture shows the used unconfined compression device

3.6.4 Unconsolidated Undrained Triaxial Test

The equipment used in the thesis manufactured by ELE International, Loveland, CO. The triaxial apparatus was capable of testing triaxial specimens with diameters as large as 70 mm with applied confining pressures up to 1,700 kPa. A height to diameter ratio of two was used for all specimens, as prescribed by ASTM D 4767. The pressure control panel boards used in this study were an ELE pressure panel, Control Panel capable of delivering pressures up to 1,700 kPa. The load frame used in this study was an ELE Digital Tritest 50 Load Frame capable of delivering a maximum axial load of 50 kN with programmable displacement rates between 0.00001 and 9.99999 mm/min.

The Procedure of the Test

1- Sample was cut by knife to get the standard height, is equal to two times the diameter, then it was placed on the base of the cell between a porous paper and a porous stone at each end.

Figure 3.17 Soil sample placed on the base of the cell

The sample is covered with the membrane with two or more O-rings are snapped onto the top cap and the base, to secure the membrane on the sample.

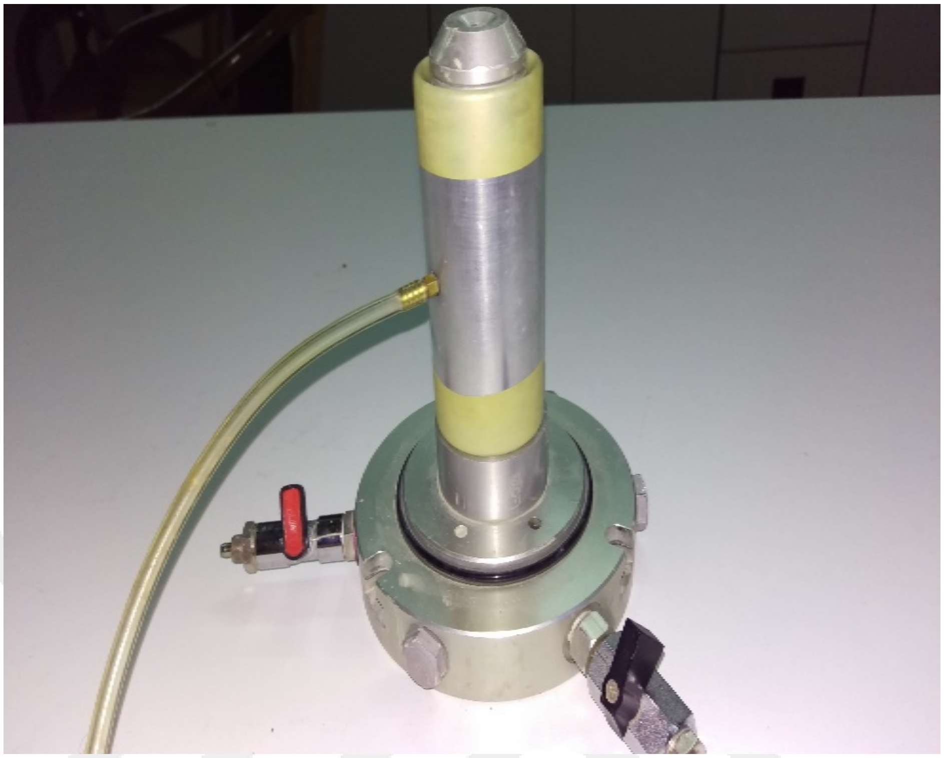

Figure 3.18 The sample is covered with the membrane

2- Pyrex cover is placed over the base to complete the cell.

4- The cell is filled with the water to make sure that the confining pressure is uniformly distributed alone the sample.

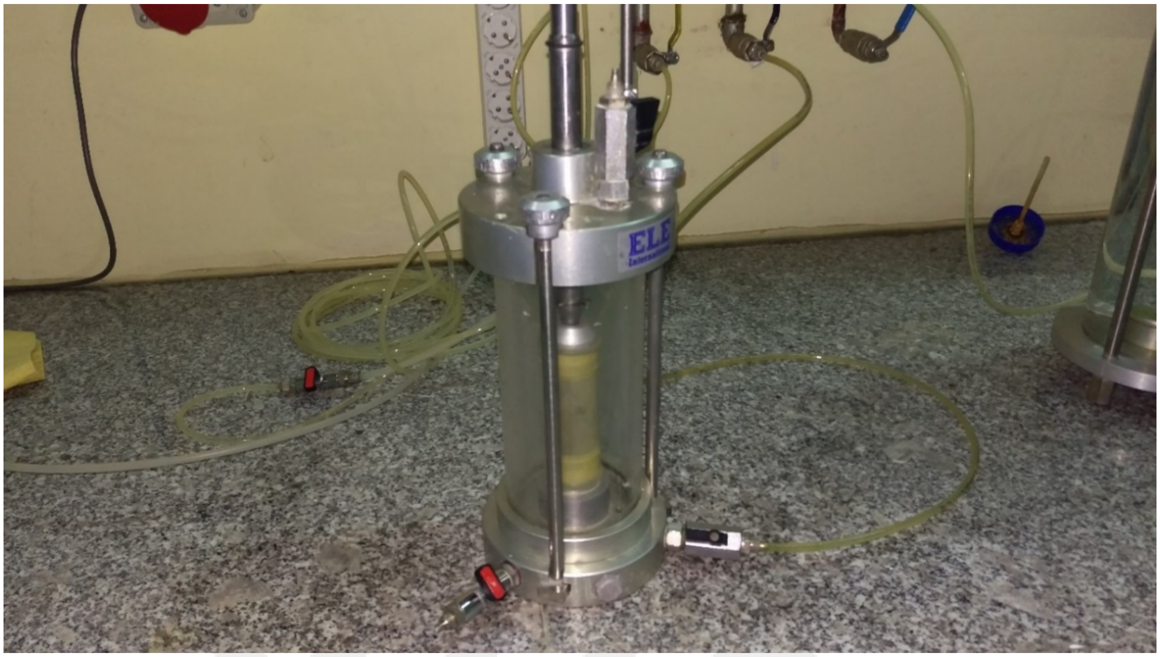

Figure 3.19 The cell is filled with the water

5- Cell is placed in the testing machine and in the frame and correcting the dial gauges to start the test.

Figure 3.20 The picture shows the used triaxial device

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Consistency Limits

4.1.1 Consistency Limits of Soil-Sawdust Mixtures

The liquid and plastic limits tests were carried out with different percentages of soil–sawdust. The effects of sawdust content on liquid limit, plastic limit and plasticity index for the sawdust- soil samples shown in Table 4.1 and are plotted in Figure 4.1.

Table 4.1 The variation of consistency limits with different sawdust content.

Soil type / Property	Row soil	1% sawdust	2% sawdust	3% sawdust	5% sawdust
L.L	73	71.5	70	66	63.5
P.L	43	42	40.7	39	38
P.I	30	29.5	29.3	27	25.5

The liquid limit value decreased by 13.01% with increasing sawdust content up to 5%. These results sounds logical because sawdust increases the exceedance of cohesionless materials in the soil and works to reduce the cohesion of the soil. However, the plastic limit decreased by 11.63% with increasing sawdust content up to 5% for the same mentioned reason. Figures 4.2 and 4.3 show the variations of liquid limits and plastic limits with different sawdust percentages as a result, the plasticity index as well decreased by 15% with addition of same mentioned percentages.

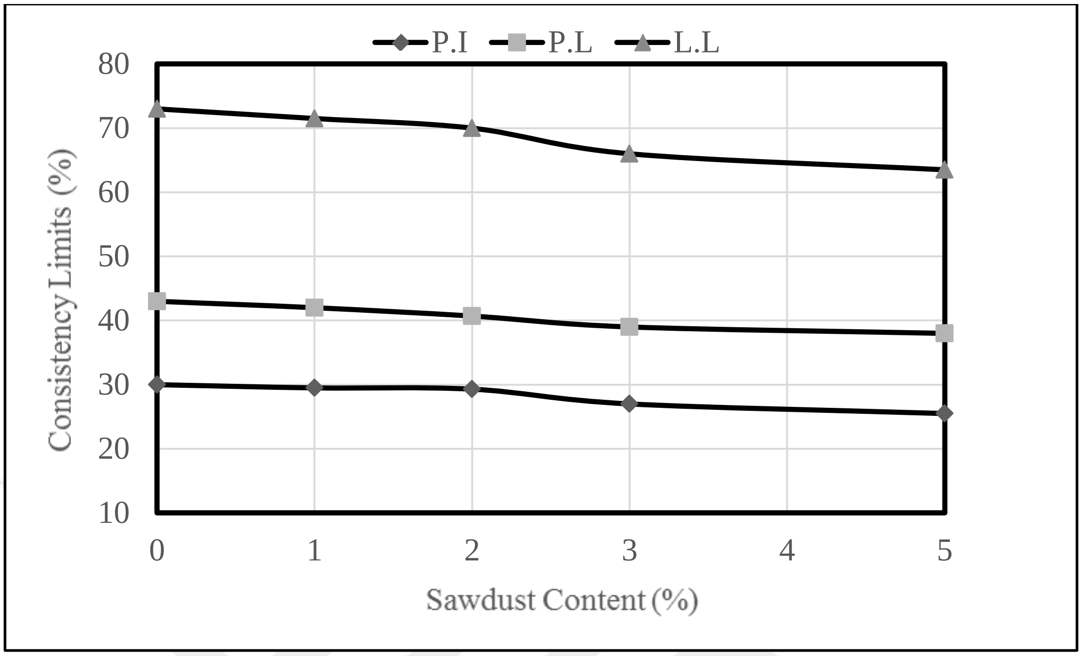

Figure 4.1 The effect of sawdust on the consistency limits

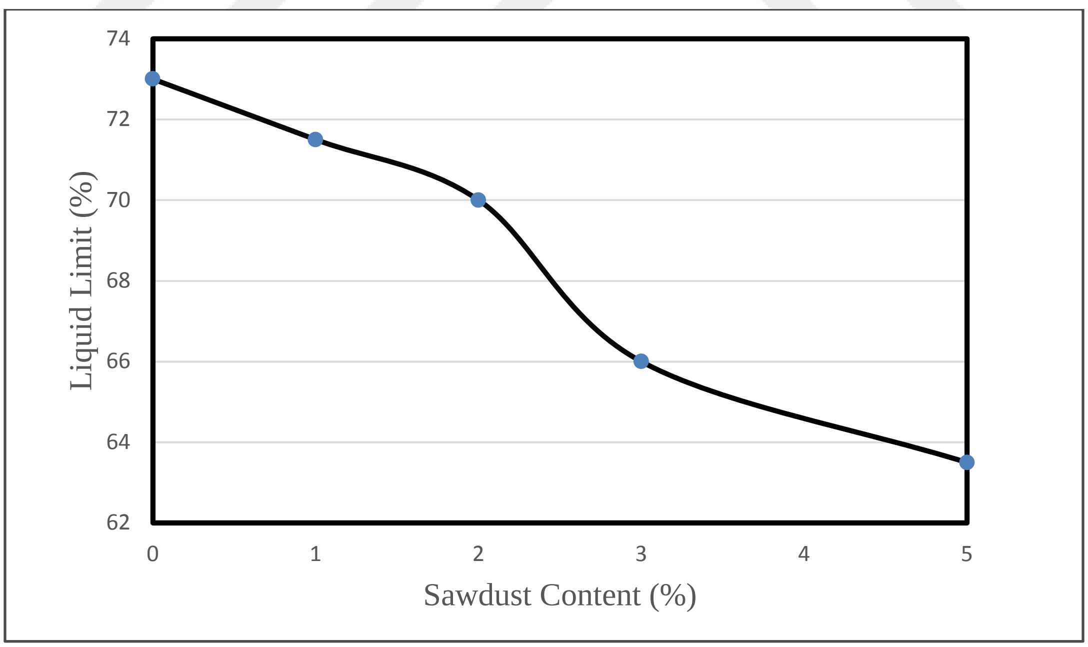

Figure 4.2 The effect of sawdust on the liquid limits

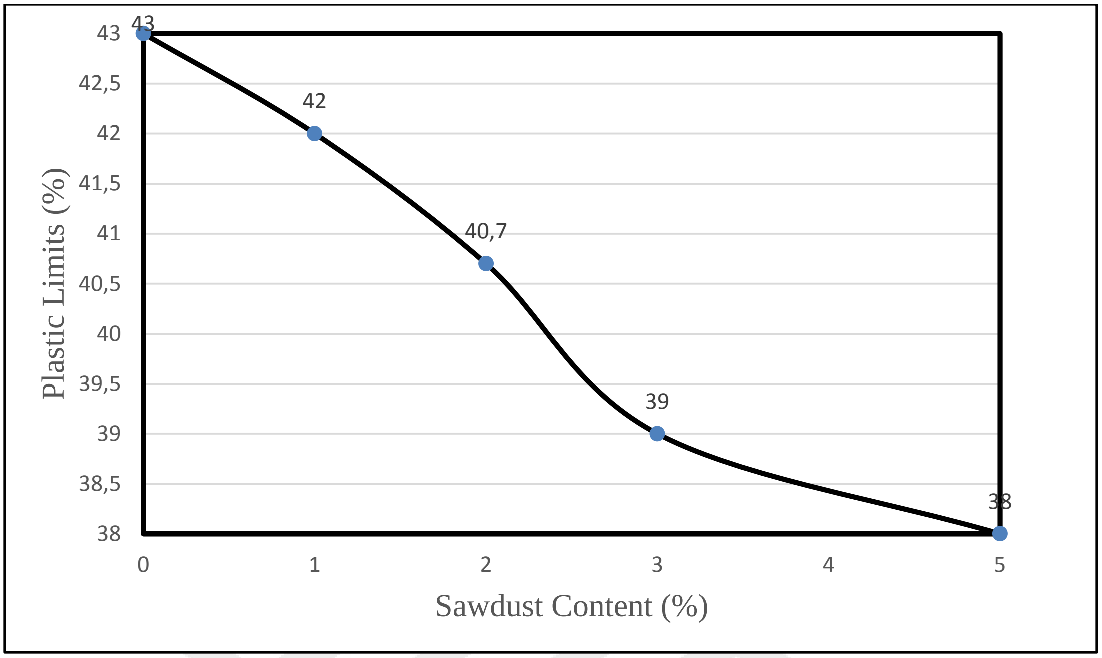

Figure 4.3 The effect of sawdust on the plastic limit

4.1.2 Consistency Limits of Soil-Lime Mixtures

Consistency limits were conducted with different percentages of lime on a soil passed on sieve No. 40, results were plotted in Figure 4.4 and showed in Table 4.2, lime addition decreases liquid limit of the soil Figure 4.5, but it increases the plastic limit as shown in Figure 4.6. Similar behavior was found by [23] and [26].

According to Bell, in most cases the addition of lime has more or less immediate effect on the plasticity of clayey soils. Increasing calcium ions which resulted from the lime addition, usually lead to reduce the plasticity of the soil [23]. The clay particles forming aggregates experiencing the flocculation, the aggregates changes the behavior of the soil like particles of silt, to meet changes in clay behavior, very little amounts of lime are required, it depends on the amount and type of clay minerals presented in soil varies from 1% to 3% [23]. The tests concluded that the addition of lime up to 5% decreased the liquid limits and the plasticity index by 14.52% and 52% respectively, but it increased the plastic limits by 11.63% because increasing lime content means increasing the contact between soil particles, Figure 4.7.

Table 4.2 The variation of consistency limits with different lime content.

Soil type	Row soil	1% sawdust	2% sawdust	3% sawdust	5% sawdust
L.L	73	70.5	69	65.3	62.4
P.L	43	44.4	46	47	48
P.I	30	26.1	23	18.8	14.4

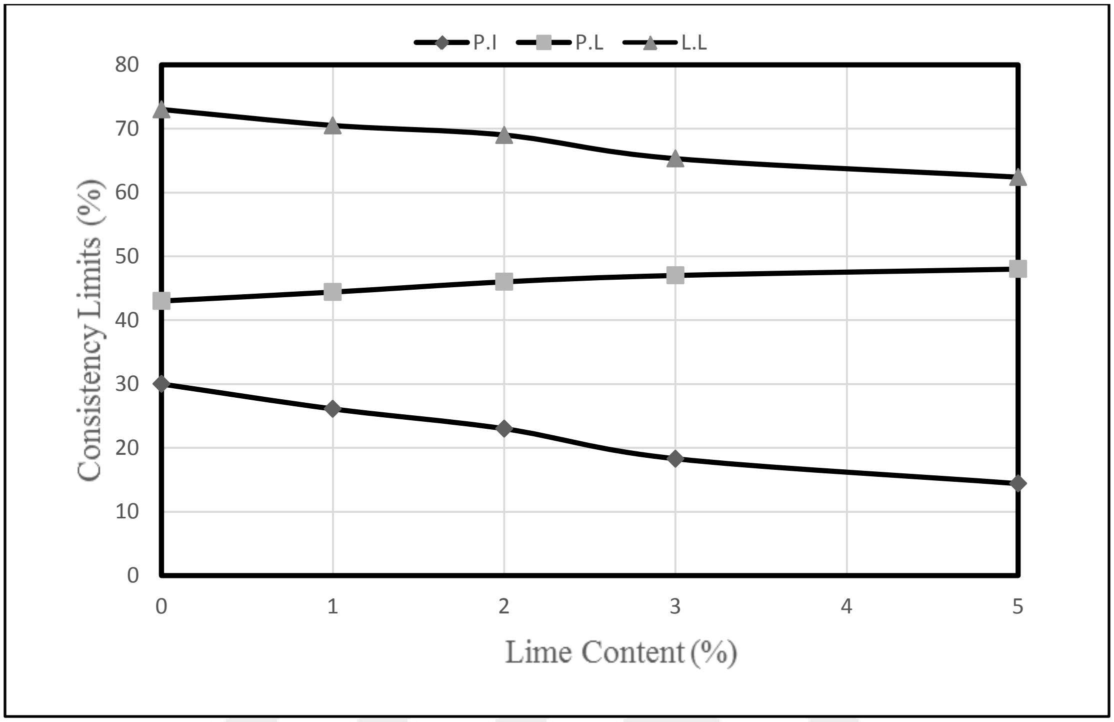

Figure 4.4 The effect of lime on the consistency limits

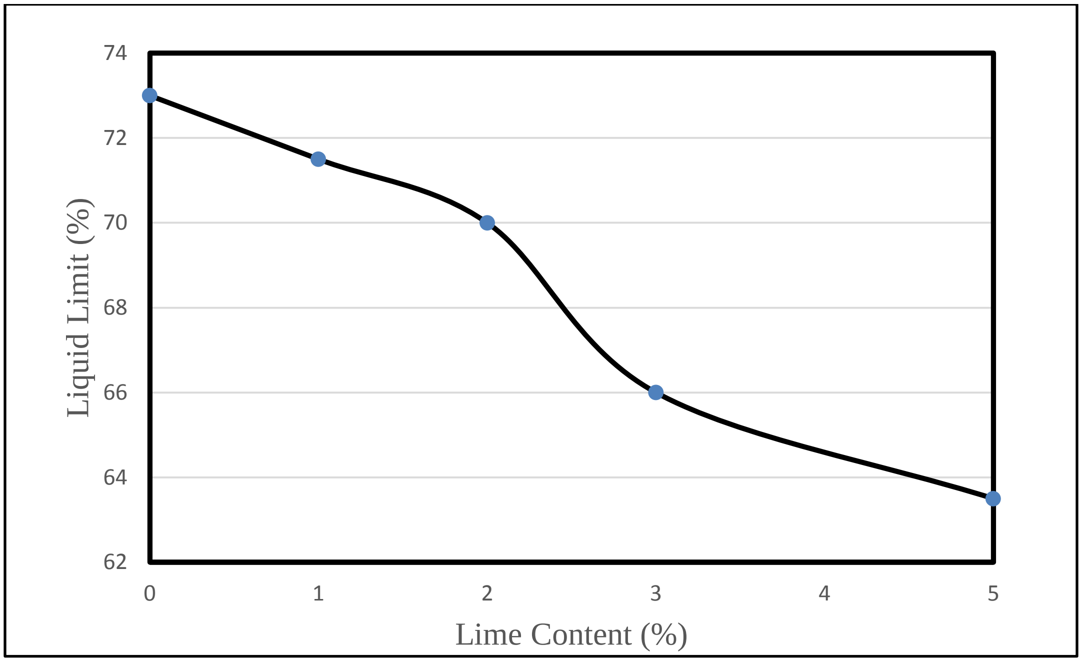

Figure 4.5 The effect of lime on the liquid limits

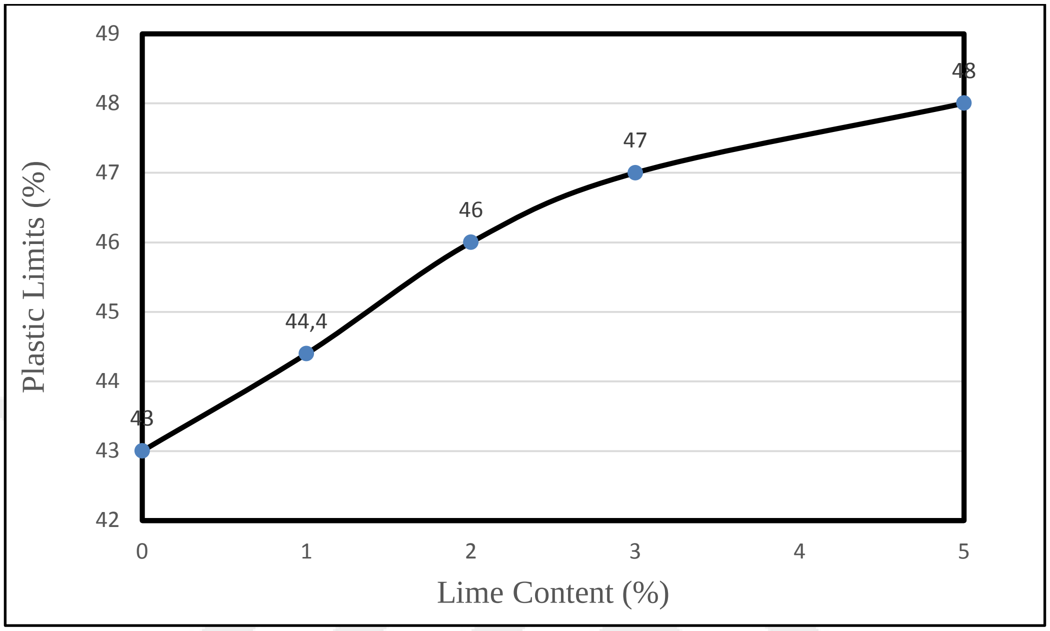

Figure 4.6 The effect of the lime on the plastic limits

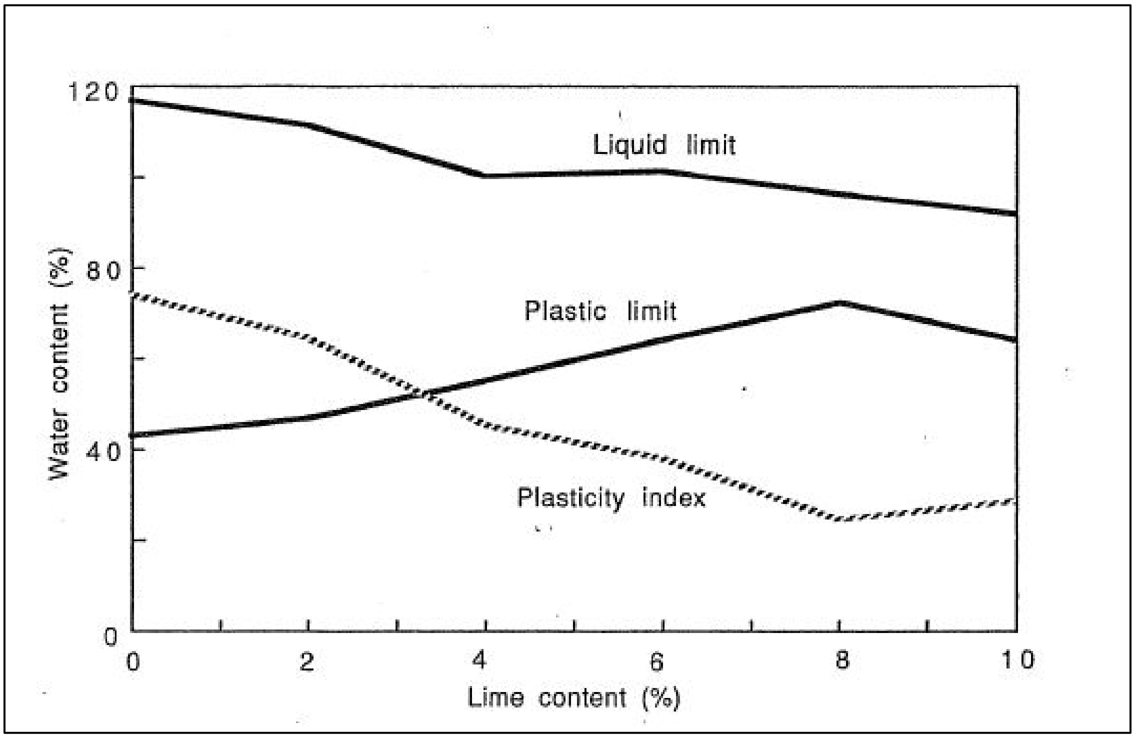

Figure 4.7 The effect of the lime on the consistency limits [54]

4.2 Compaction Parameters

4.2.1 Compaction Parameters of Soil-Sawdust Mixtures

The compaction curves were plotted in the Figure 4.8 and the values of optimum water content and maximum dry unit weight were determined. The addition of sawdust affected the compaction parameters of soil-sawdust mixtures. It was observed that the maximum dry unit weight decreased by 7.60% with the addition of sawdust as shown in Figure 4.9, and the optimum water content also decreased by 9.7% as shown in Figure 4.10, similar behavior was found during the usage of natural fiber with the soil [12]. Values of optimum water contents and maximum dry unit weights were summarized in Table 4.3.The reason behind the reduction of maximum dry unit weight can be understood by knowing that added sawdust has a unit weight less than the unit weight of the soil. On the other hand sawdust reduces optimum water content, this might be happen due to two reasons, sawdust works as a filler for gaps and voids between soil particles. Thus, the soil added sawdust can reach to the maximum unit weight with less amount of water and the second reason is that the added sawdust has less moisture content than the used soil, thus it is normally absorbs water from the soil and reduces the optimum moisture content. Maximum water absorption for the natural fibers happens during the first 24 hours [55], as shown in Figure 4.11.

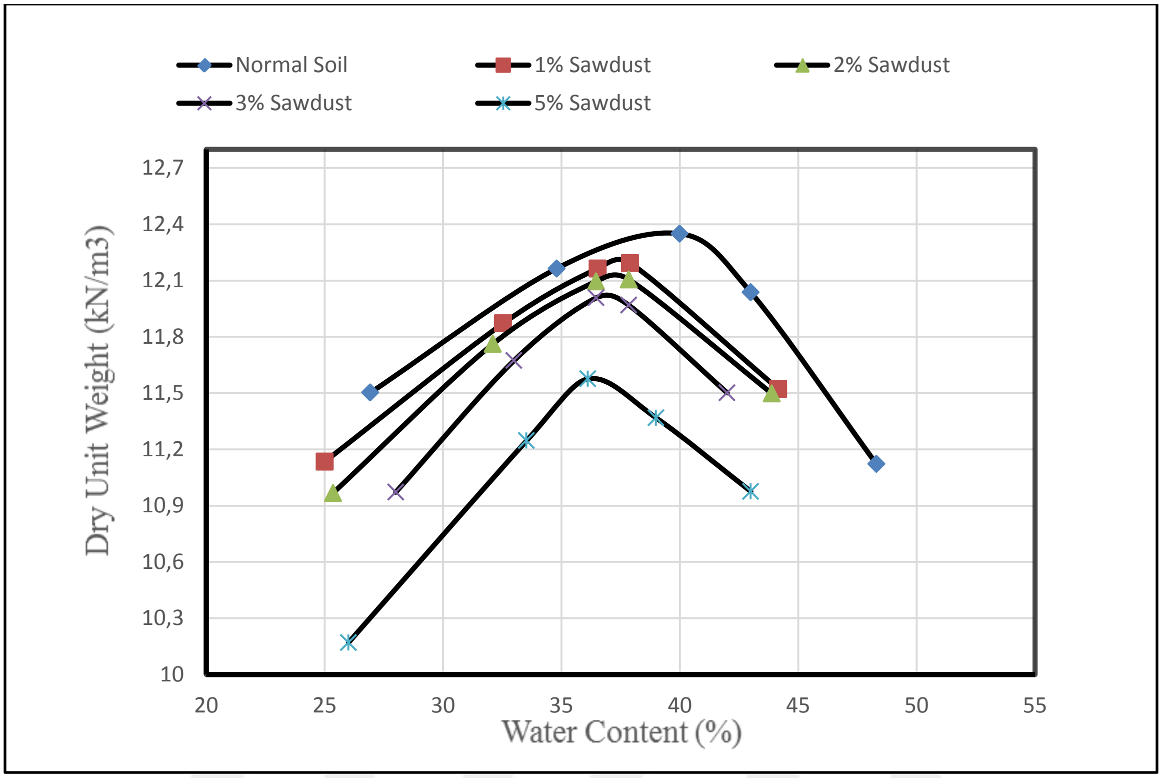

Figure 4.8 The effect of sawdust content on compaction parameters

Table 4.3 The effect of sawdust on compaction parameters

Soil type / property	Row soil	1% sawdust	2% sawdust	3% sawdust	5% sawdust
Optimum Moisture content %	40	37.91	37.85	37.38	36.12
Maximum Dry Unit Weight kN/m^3	12.35	12.18	12.05	11.77	11.41

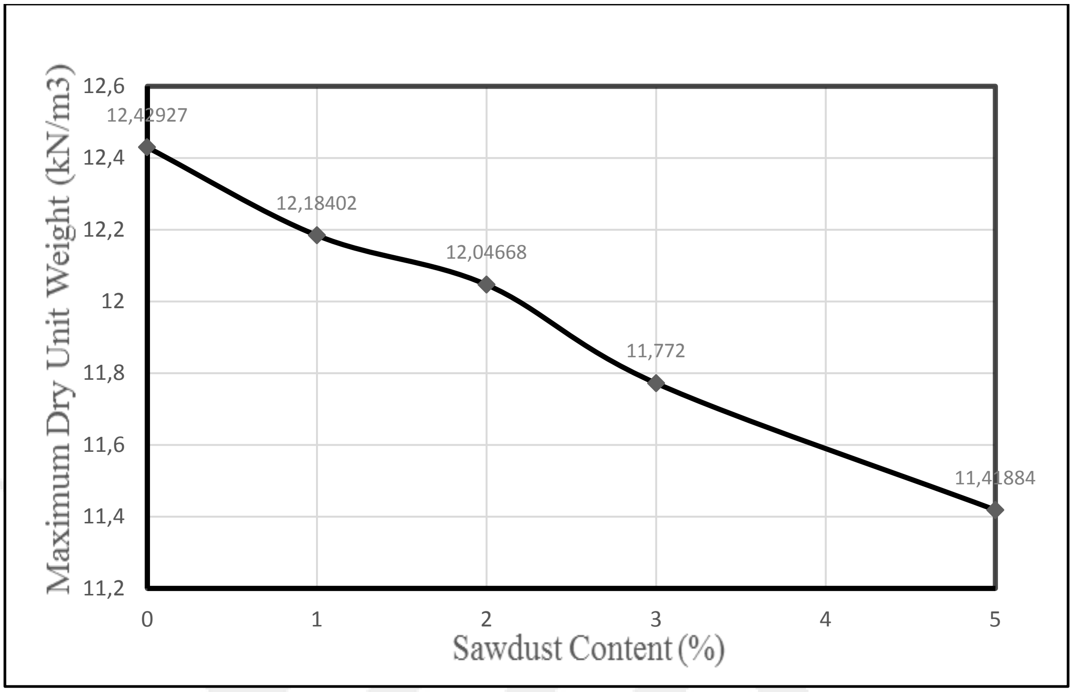

Figure 4.9 The effect of sawdust on the maximum dry unit weight

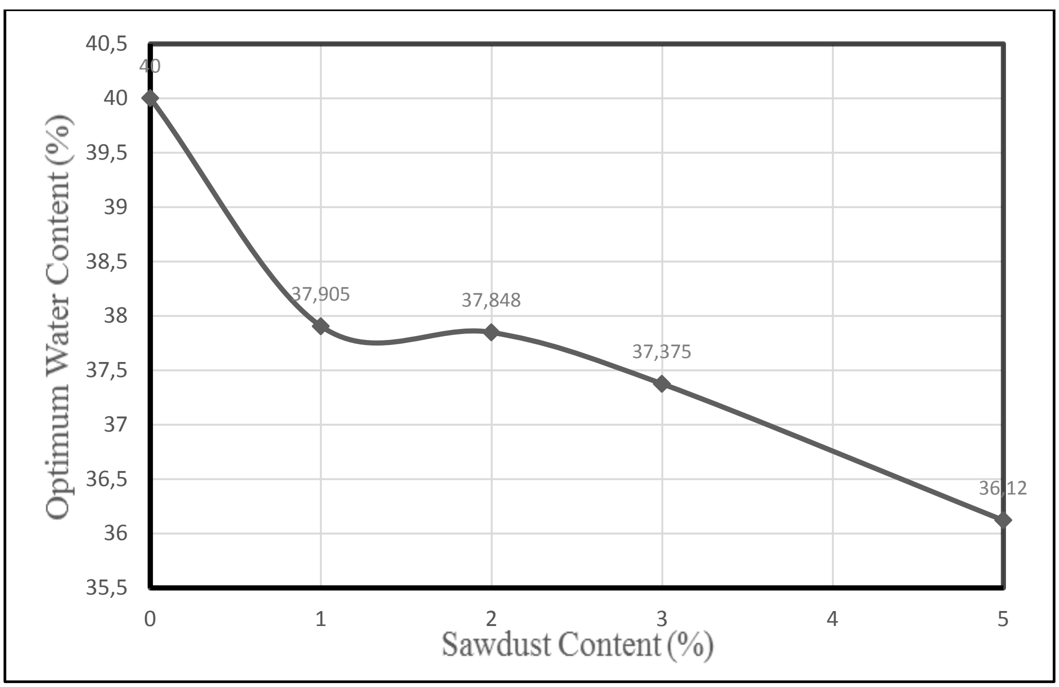

Figure 4.10 The effect of sawdust content on the optimum water content of the mixture.

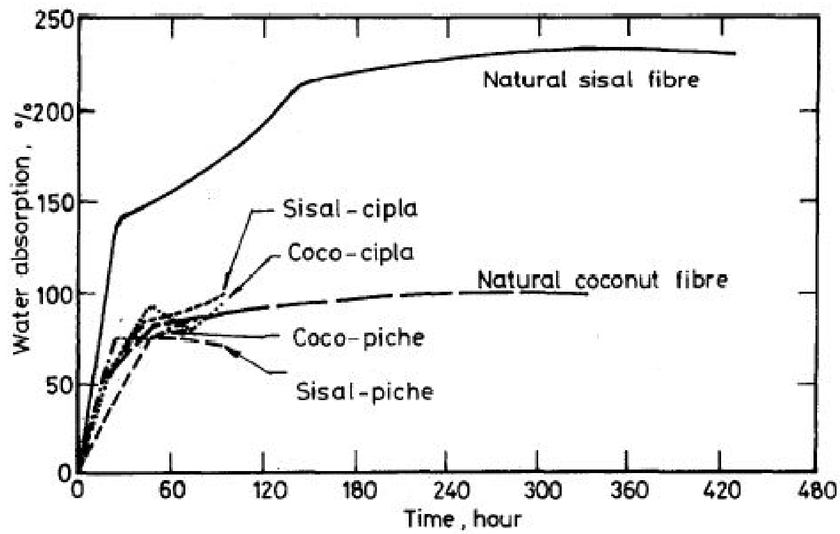

Figure 4.11 Water absorption of natural fibers [55]

4.2.2 Compaction Parameters of Soil-Lime Mixtures

Compaction test was conducted to determine the behavior of optimum moisture content and maximum dry density during the process of adding different percentages of lime figure 4.12 and Table 4.4 the addition of lime to clayey silt soil passed from sieve No. 40 reduce maximum dry unit weight by 4.05% as shown in Figure 4.13, and increase the optimum moisture content by 1.73% as shown in Figure 4.14 for the same compactive effort. Same behavior was also found by other different researchers [23], [26], [33], [35], and [36]. Figure 4.15 shows the mentioned behavior with different size distribution of soils.

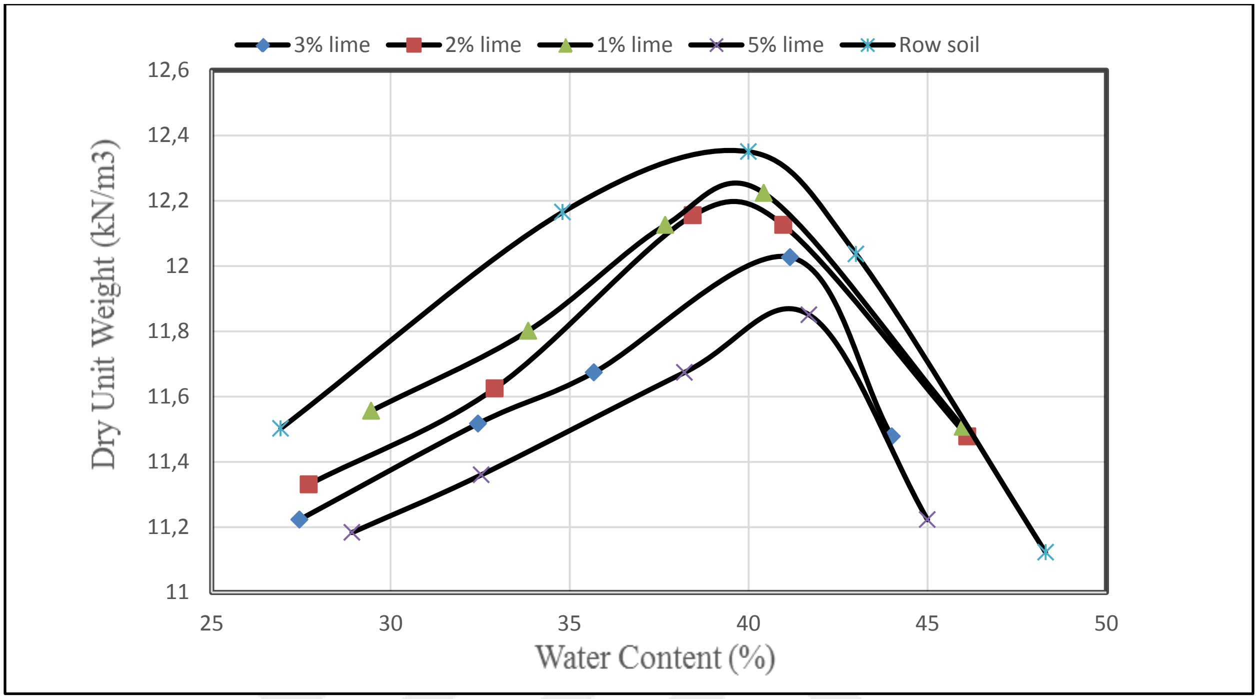

Figure 4.12 The Effect of lime content on compaction parameters.

Table 4.4 The effect of lime on compaction parameters

Soil type / property	Row soil	1% Lime	2% Lime	3% Lime	5% Lime
Optimum Moisture content %	40	40.3	40.97	41.16	41.69
Maximum Dry Unit Weight kN/m^3	12.350	12.223	12.125	12.027	11.85

The reduction in the dry unit weight has a couple of different reasons, it may happen because of the instant formation of cementitious products, the cementitious products have a big volume, thus, it reduce the compatibility [25], lime causes aggregation of particles to occupy larger spaces, for that reason the specific gravity for lime is generally lower than the specific gravity of the used soil [23]. Adding lime increase the porosity of the soil, as well it decrease the unit weight [33].

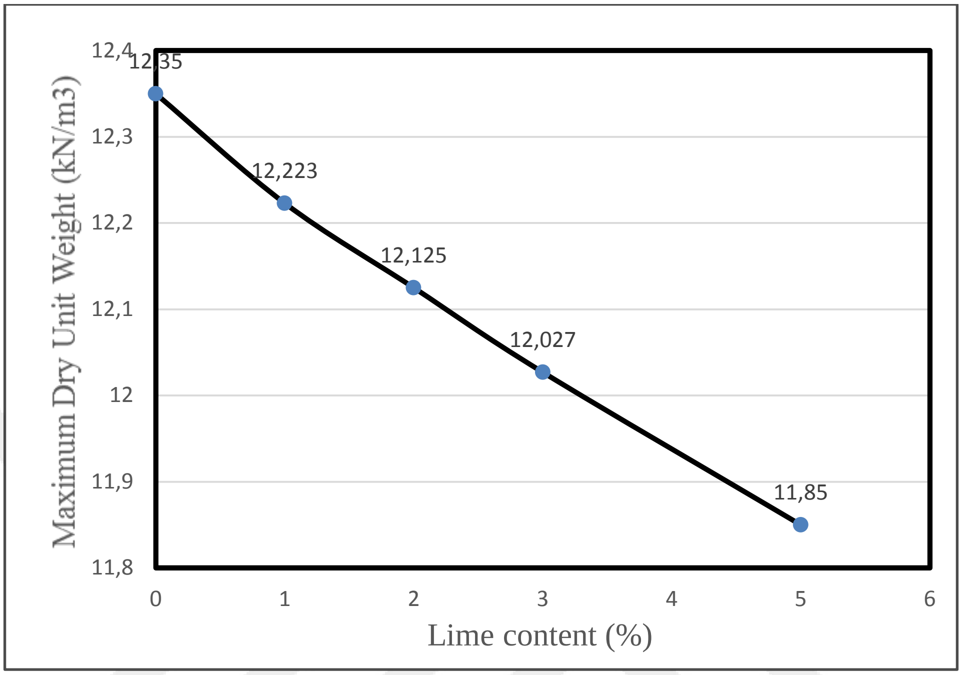

Figure 4.13 The effect of lime content on maximum dry unit weight

Increased water content with the increase in the content of lime is a result of the water required during of the compaction process, by increasing lime amount, more water will be needed for reaction and finally optimum water content will be increased [33] the pozzolanic reaction between clays and lime generally caused increase in optimum water content [23].

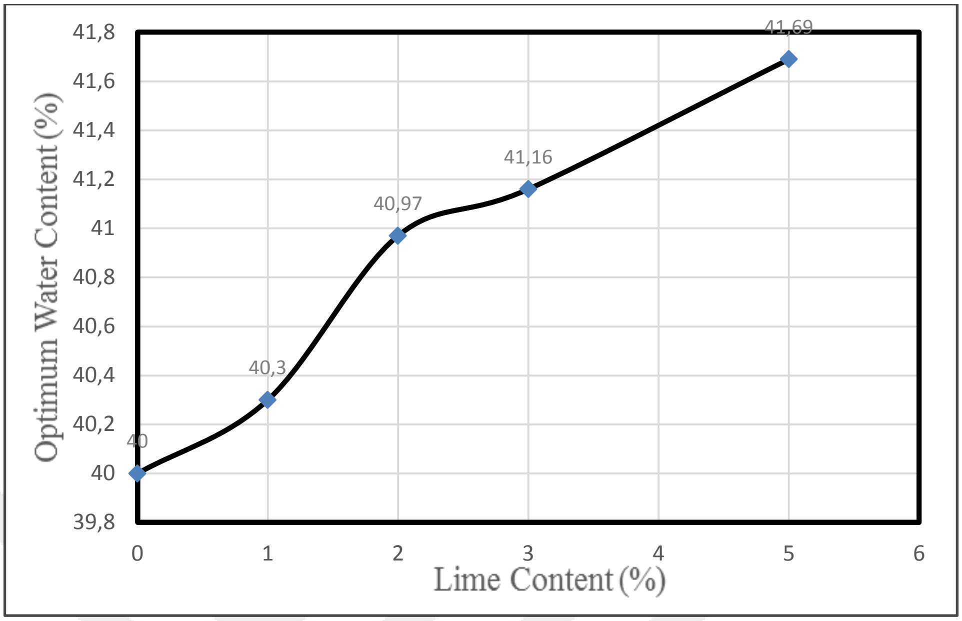

Figure 4.14 The effect of sawdust content on the optimum water content of the mixture

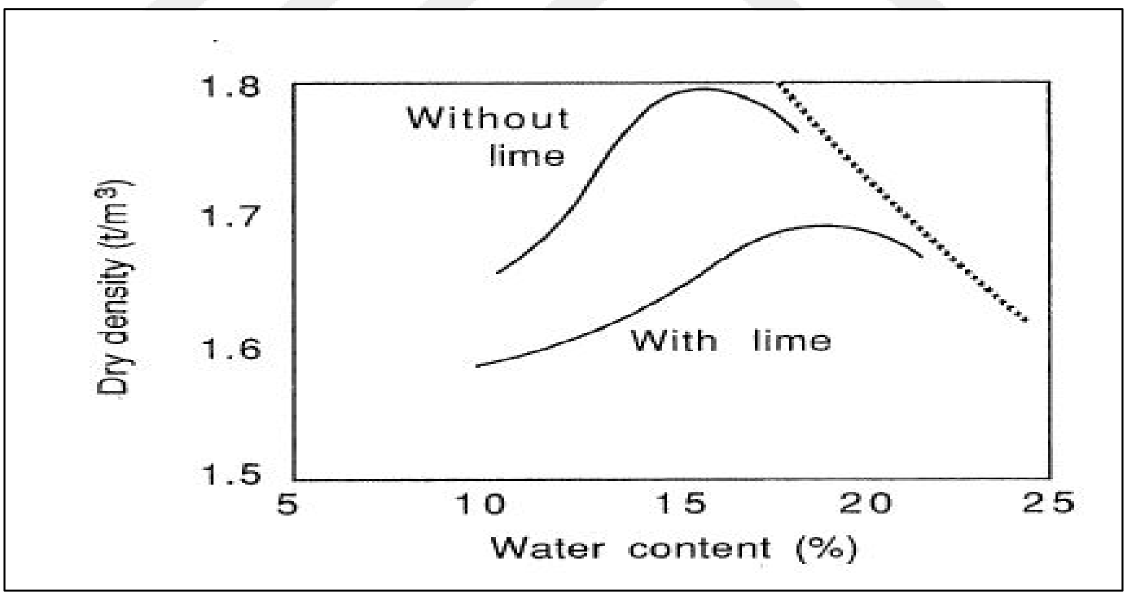

Figure 4.15 Effect of lime on the compaction parameters [5]

4.3 Shear Strength

4.3.1 The Sawdust

General Sawdust-Soil Shear Strength

The Immediate Behavior of Soil-Sawdust Mixtures

The grain size distribution curve clearly shows that more than 60% of the used sawdust has particles less than 0.420 mm (420 micrometers) which consider smaller than the particles of used soil. The 60% of the total weight of the sawdust works as fillers, it fills the void between each successive particles Figure 4.16. On the other hand there is the 40% bigger than soil's size, it collects and covers the particles and increases the bond. Sawdust mainly formed of cellulose, hemicelluloses, and lignin [56]. Cellulose composed mainly of Carbon units. Those Carbon units are linked together by bonds called covalent bonds, it is a strong bonds to contributing to sustain a part of stresses generated as a result of shed loads on the soil samples.

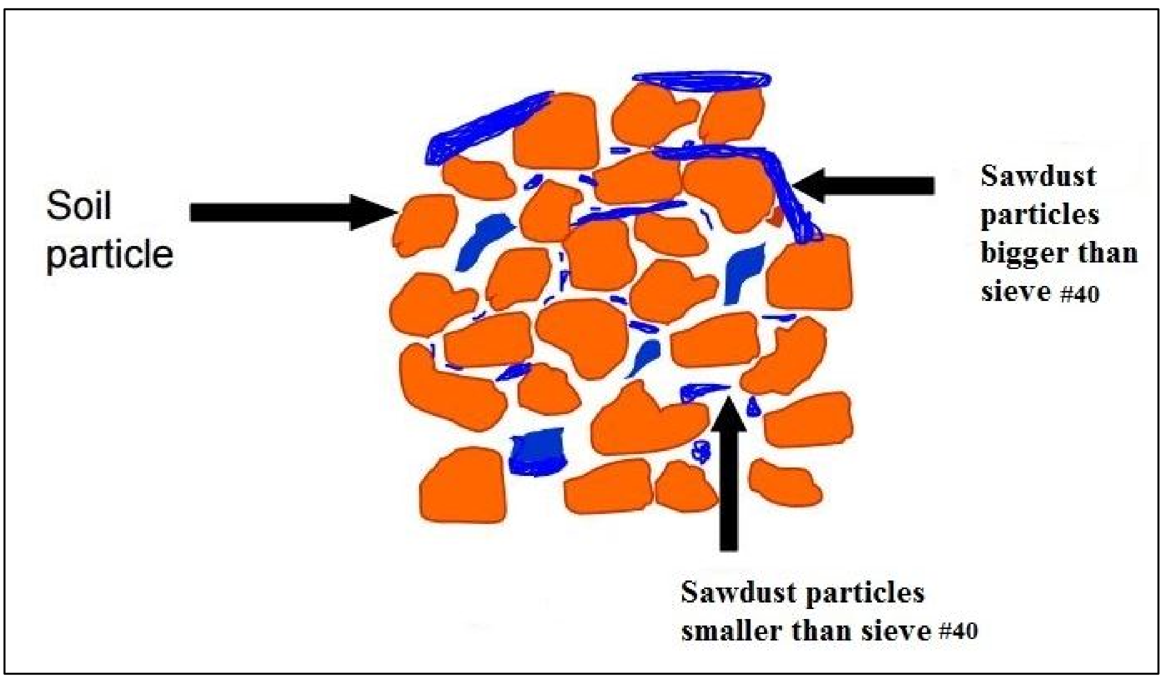

Figure 4.16 The mechanism of working sawdust as filler material

And also sawdust consist of Lignin, the structure of Lignin mainly consist of small vessels look like pipes. During the load application the failure passes through a series of consequences stages:

1-It happens firstly in the Lignin of sawdust which bond the particles

2- Covalent bonds break, then a part of applied stresses absorbs by the fillers then fillers (which works as supports) fails

3- Then the failure happens in the soil particles or by sliding the particles over each other. Each single stage needs an amount of load (in form of stresses) to finish, the majority of stresses assigned by soil particles and the remaining supports by the sawdust, that's why the strength increases. But then it decreases when the amount of sawdust reaches 5%, because sawdust take places of soil particles and sawdust. Unlike the soil, it provides less strength. Sawdust also increases the roughness of the soil.

Long Term Behavior of Soil-Sawdust Mixtures

Shear strength of soil-sawdust mixtures increase over time, the supplementary strength during long term is a result of chemical reactions between sawdust and components of the soil.

Table 4.5 The difference in optimum water content in the immediate and long term tests for soil-sawdust samples.

Property / Soil Type	O.M.C (%)	Actual water content during compaction (%) (0 days)	Actual water content during long term tests (%) (90 days)
1% sawdust	37.905	37.220	36.052
2% sawdust	37.848	37.530	35.300
3% sawdust	37.375	37.450	34.020

5% sawdust	36.120	36.320	33.570

According to Kirsch [57], the long term sawdust-soil reaction decreases Nitrogen gases and Phosphor's content in the soil, while the long term reaction between them increases the concentration of Carbon in the soil mass.

The shortfall in the Nitrogen gases contribute to increase the shear strength, the presence of nitrogen gas inside of the soil causing bubbles which are causing the fragmentation of the soil and also causes instability for the soil which might led to decrease the strength. Getting rid of the nitrogen gas is a power factor of the soil by reducing the voids. In the same way, Phosphorus's lack normally led to increase the shear strength, Phosphorus does not increase soil strength, and may even decrease soil strength, the unconfined compressive strength, during 90 days, decreased with increasing Phosphor acid (Ingles) [58]. The author also stated that phosphoric acid was not able to form insoluble salts. Yoah and Oades [59], found an increase in the proportion of sand and silt size particles due to phosphoric acid treatment. These larger particles are micro aggregates. The researcher suggested that acid treatment decreases the area of contact between soil particles resulting in a decrease the attractive forces between particles.

Sawdust, over time, reduces the water content of the particles. The reduction in the moisture content happens in the particle itself but the water still in the structure of the sample within the mass of the sawdust and that what clearly can be seen while the process of calculating of water content .

Because almost of the strength comes from the soil particle as interlocking and friction, the dry particles has higher strength than the wet particles. That was the main cause behind strength increasing.

4.3.1.1 Unconfined Compression Strength of Sawdust- Soil (Immediate)

The immediate effects of sawdust on the unconfined compressive strength for stabilized clayey silt soil samples was shown in Table 4.6. Figure 4.17 shows the stress-strain relationship. The shear strength of soil due to unconfined compression test of stabilized samples was increased by 41.44% with increasing sawdust content from 0% to 3%.

However, then the strength was decreased by adding 5% of the sawdust. The maximum unconfined compressive strength of clayey soil samples was occurred at 3% sawdust content. Figure 4.18 a curve shows effect of sawdust on the cohesion of the soil in the unconfined compression test.

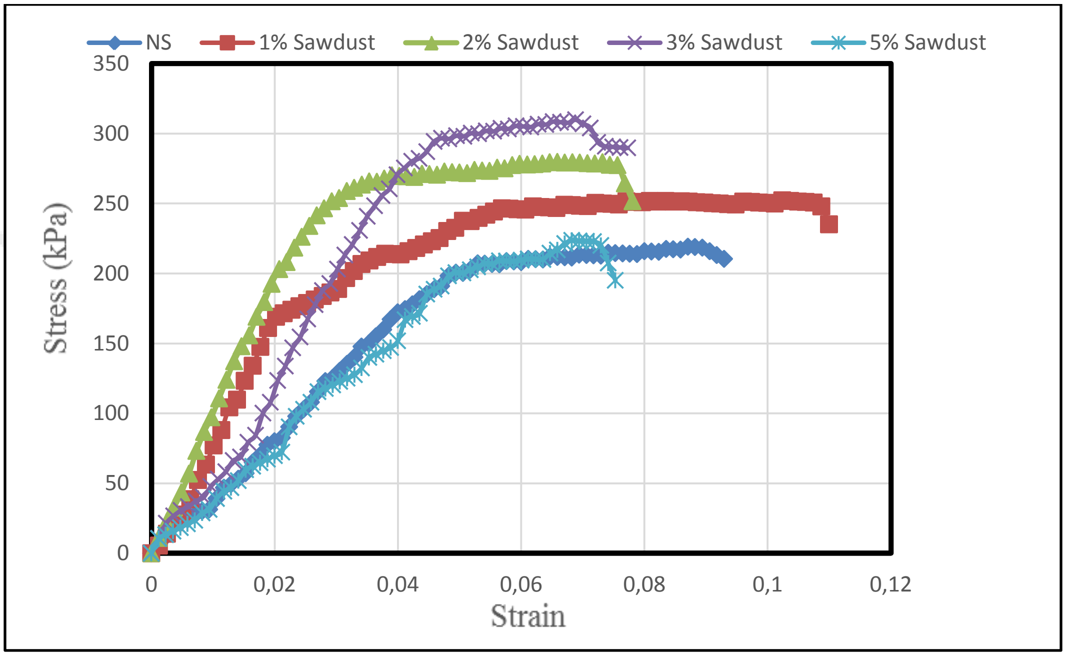

Figure 4.17 Stress-strain relationship of unconfined compression strength immediate tests

Table 4.6 the effect of sawdust on the immediate shear strength due to unconfined compression test

property	Row soil	1% Sawdust	2% Sawdust	3% Sawdust	5% Sawdust
Stress (kPa)	219.08	252.18	279.7	309.86	223.6
Shear strength (kPa)	109.54	126.09	139.85	154.93	111.8

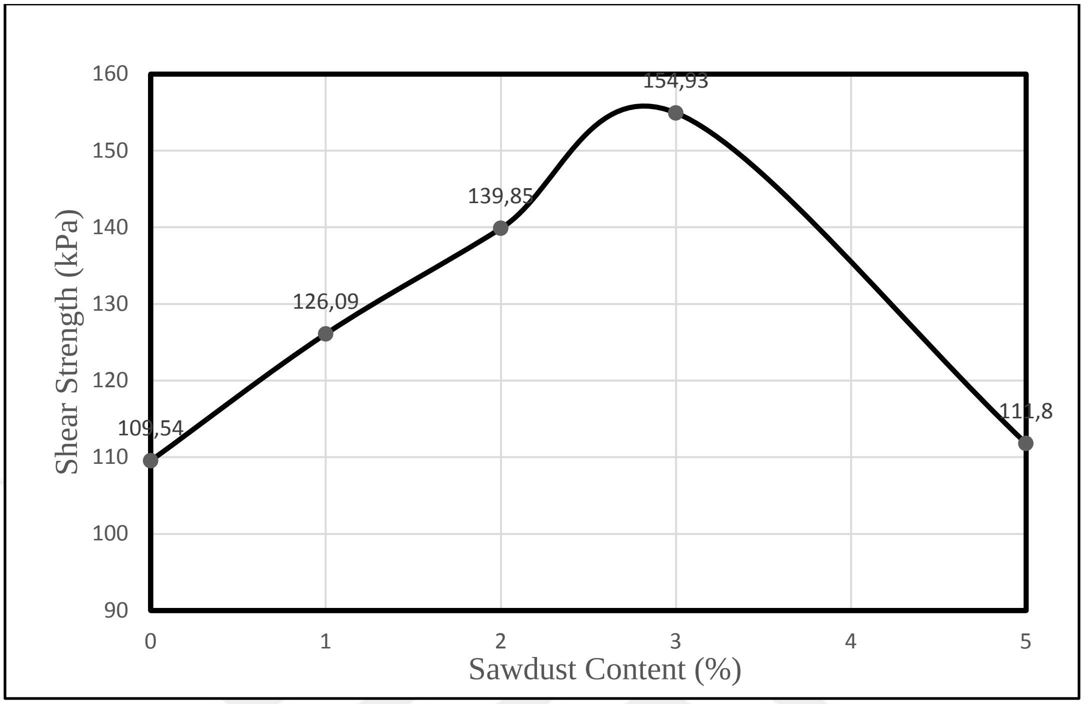

Figure 4.18 Immediate Effect of Sawdust on the shear strength of soil of the soil in the Unconfined Compression test

4.3.1.2 Unconfined Compression Strength of Sawdust Soil (Long Term)

Unconfined compressive strength was conducted to estimate long term behavior of clayey silt soil mixed with four different percentages of sawdust (1, 2, 3 and 5) %. As can be seen from Figure 4.19, sawdust increases the shear strength of soil of the soil. As comparing with the immediate tests, the shear strength for 1% sawdust increased during 90 days from 126.09 kPa to 265.8 kPa 110.80%, during the same period of time 2% mixtures was increased up to 116.68%. As well as 3% increased by 31.68% unlike other sawdust percentages, results show that 5% sawdust-soil mixtures' shear strength was decreased by 0.69% only.

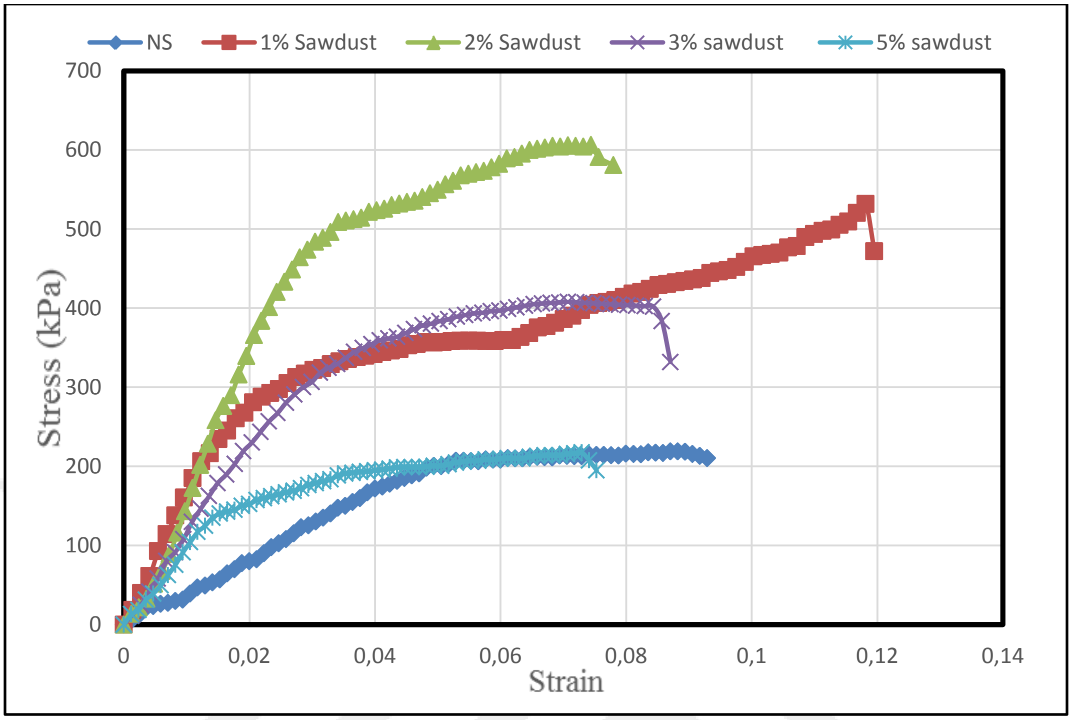

Figure 4.17 Long term effect of sawdust on stress-strain curve of the soil

Table 4.19 The long term effect of sawdust on the shear strength of soil due to unconfined test

Property	Row soil	1% Sawdust	2% Sawdust	3% Sawdust	5% Sawdust
Stress (kPa)	219.08	531.6	606.06	408.02	217.58
Shear strength (kPa)	109.54	265.8	303.03	204.01	108.79

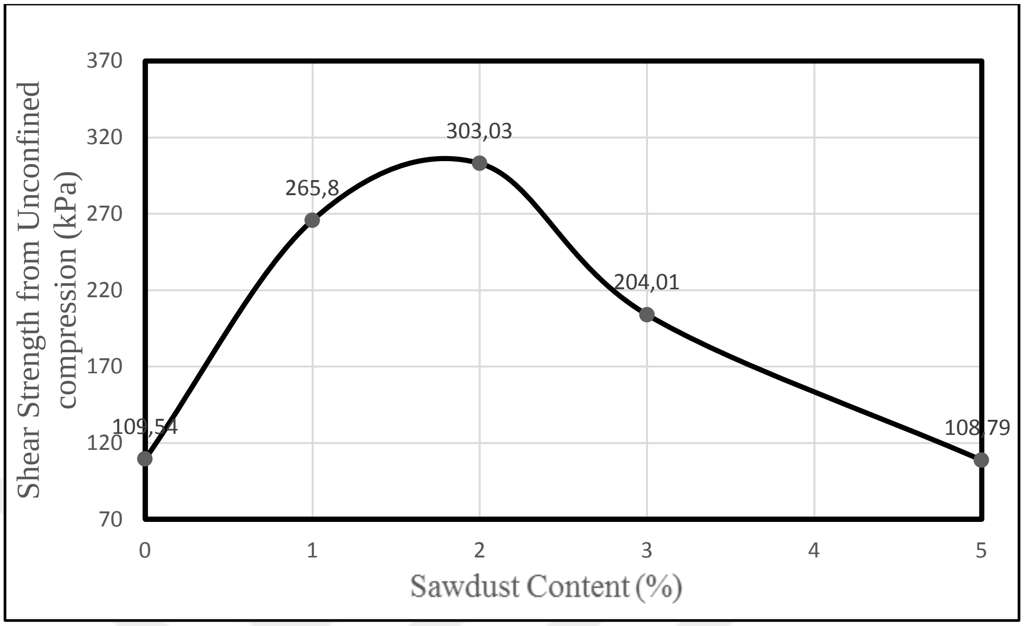

Figure 4.20 Long term Effect of Sawdust on the shear strength of the soil in the Unconfined Compression test.

Instead of 3%, 2% was the optimum sawdust content for the mixture in the long term tests, the reaction between sawdust and the soil was the main cause of the difference. Also observed concentrations of decay on the outer surface of the soil sample for each of percent 3% and 5% are likely to have contributed to lower strength to 3% due to a series of biochemical reactions that micro-organisms that cause osteoporosis soil sample and thus decrease resistance. Figure 4.20 shows the relationship between sawdust content with shear strength of the soil and Figure 4.21 shows the decay on the surface of the soil sample due to storage for 90 days.

Figure 4.21 the decay on the surface of the soil sample due to storage for 90 days

Anyway, 2% sawdust soil mixture improved the shear strength over 90 days up to 176.64% as comparing with the strength of the row soil.

Unconsolidated Undrained Triaxial Test

4.3.1.3 Unconsolidated Undrained Triaxial Test of Sawdust - Soil (Immediate)

The effects of sawdust on the stress–strain relationships of the soils at the confining pressures of 160 and 320 kPa are shown in Figures 4.22 and 4.23, Table 4.8 and 4.9. Brittle failure was observed in the shear failure mode for specimens stabilized with sawdust. Moreover, for the confining pressure 160 kPa, according to the Figure 4.24, the addition of a 1% sawdust has a marginal increasing effect on the shear strength of the soil, it increased by 4.65%, while the other two percentages (2% and 3%) increased the shear strength, 2% increased the strength by 27.91% with respect to the row soil and 3% sharply increased the strength up to 39.53%. on the other hand, increasing sawdust content to 5% decreases the strength by 2.33%. A similar behavior was observed for confining pressures of 320 kPa. 3% was the optimum sawdust content during the immediate tests.

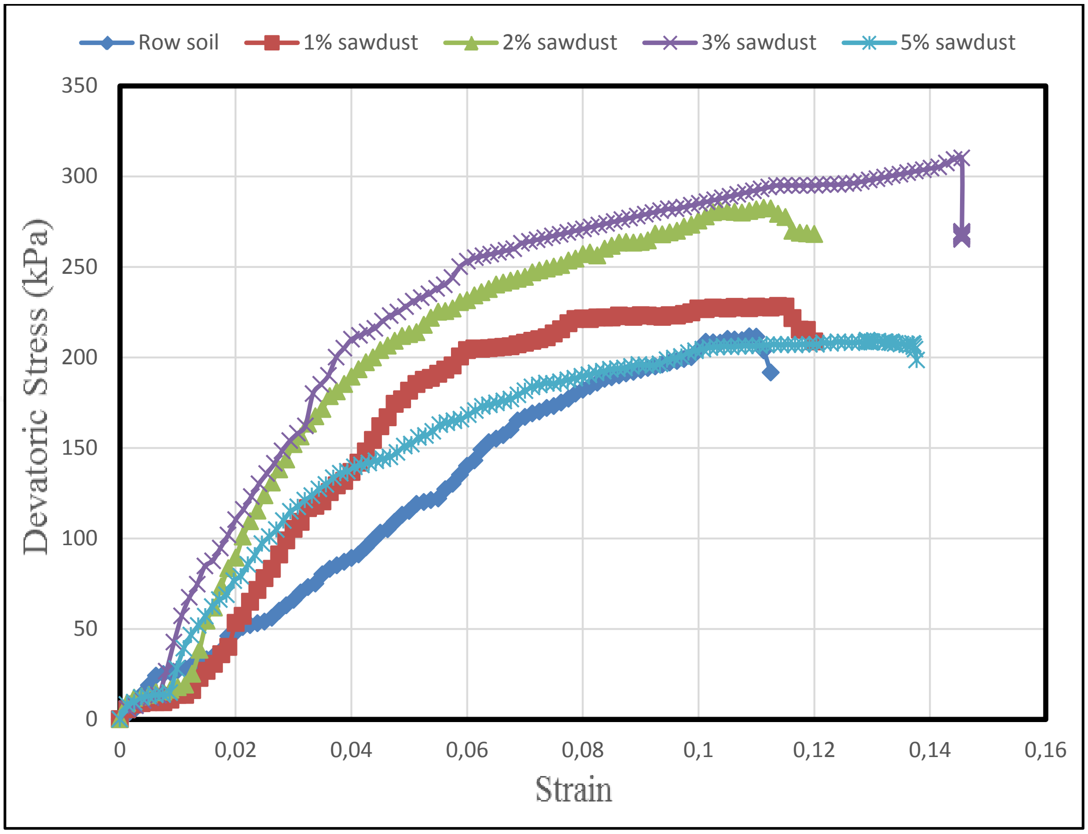

Figure 4.22 Immediate effect of sawdust on stress-strain curve of the soil at confining pressure 160 kPa

Table 2 Table 4.8 The effect of 160 kPa confining pressure on the soil-sawdust mixtures

Sawdust content (%)	σ_3 (kPa)	σ_1 (kPa)	ε_f	Shear strengh (kPa)
0	160	371	0.111	107.5
1	160	390.94	0.115	112.5
2	160	430.27	0.1125	137.5
3	160	472.20	0.14417	150
5	160	368.43	0.1310	105

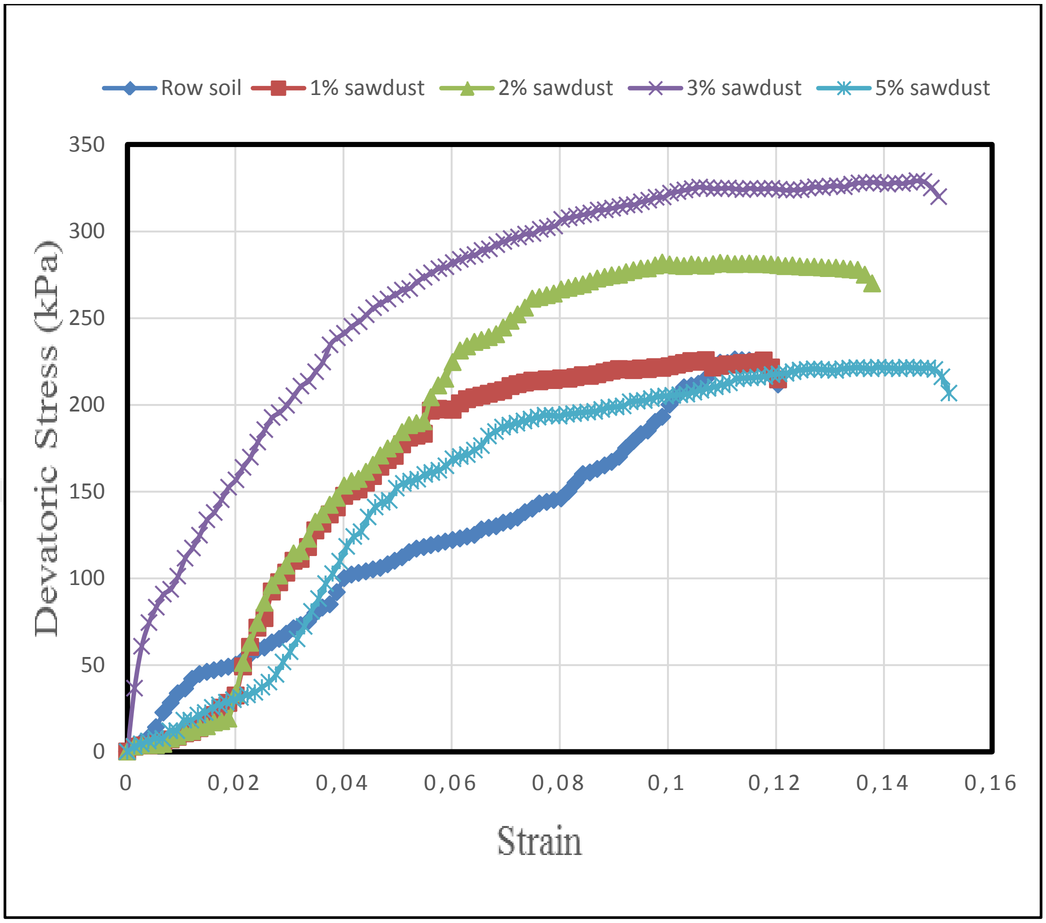

Figure 4.23 Immediate effect of sawdust on stress-strain curve of the soil at confining pressure 320 kPa

Table 4.9 Effect of 320 kPa confining pressure on the soil-sawdust mixture

Sawdust content (%)	σ_3(kPa)	σ_1 (kPa)	ε_f	Shear strengh (kPa)
0	320	546.10	0.111	107.5
1	320	557.94	0.119	112.5
2	320	590.45	0.100	137.5
3	320	641.96	0.154	150
5	320	523.64	0.149	105

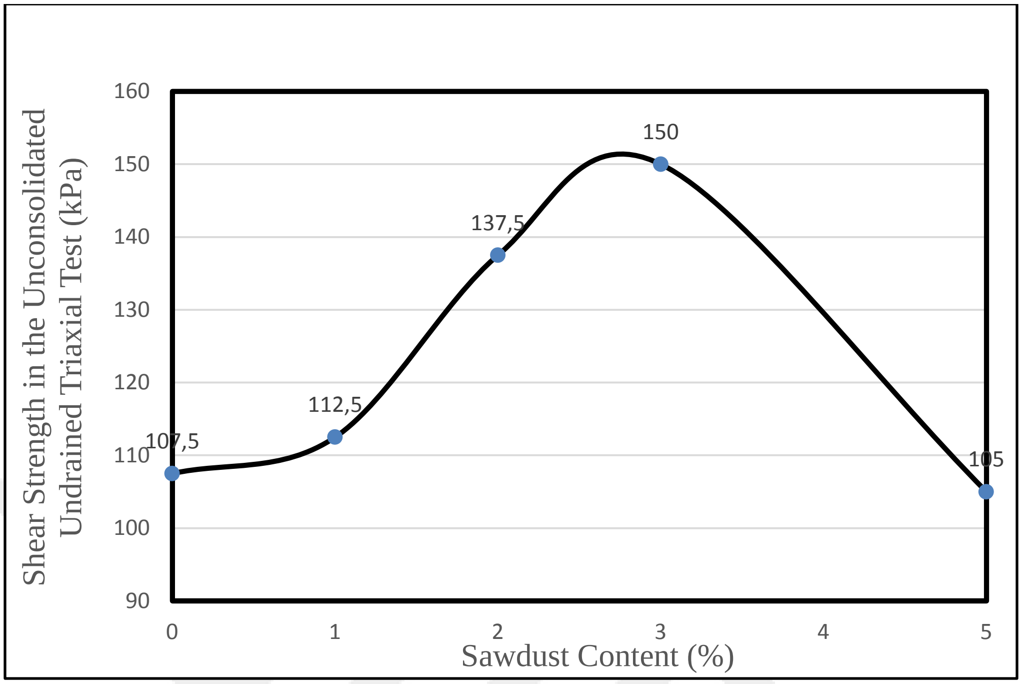

Figure 4.24 Effect of Sawdust on the shear strength of soil in the Unconsolidated Undrained Triaxial Test.

4.3.1.4 Unconsolidated Undrained Triaxial Test of Sawdust Soil (Long Term)

Two samples at least were kept in the desiccator, Figure 3.10, for 90 days (2160 hours) to investigate the difference which might happen in the strength due to the chemical reaction between sawdust and the soil, water content of soil samples was calculated to see weather if the increase of the strength happened due to the chemical reaction, loss of water content, or both of them.

Table 4.5 shows the difference in the water content during immediate and 90 days tests. Same procedure of the immediate test was used, two confining pressure were used 160 and 320 kPa respectively. As comparing with the immediate test, 1% sawdust shear strength of the soil increased during 90 days from 112.5 kPa to 270.02 kPa (140%) due to sawdust, during the same period of time 2% mixtures was increased up to 136.5. As well as 3% increased by 86.66%. Unlike the unconfined compression results, 5% sawdust increased by 57.143% in shear strength of soil due to of triaxial test. Figure 4.24 shows the long term strength against sawdust content. Figures 4.25 and 4.26 also Tables 4.9 and 4.10.

2% sawdust soil mixture improved the shear strength over 90 days up to 202.7% as comparing with the strength of the row soil.

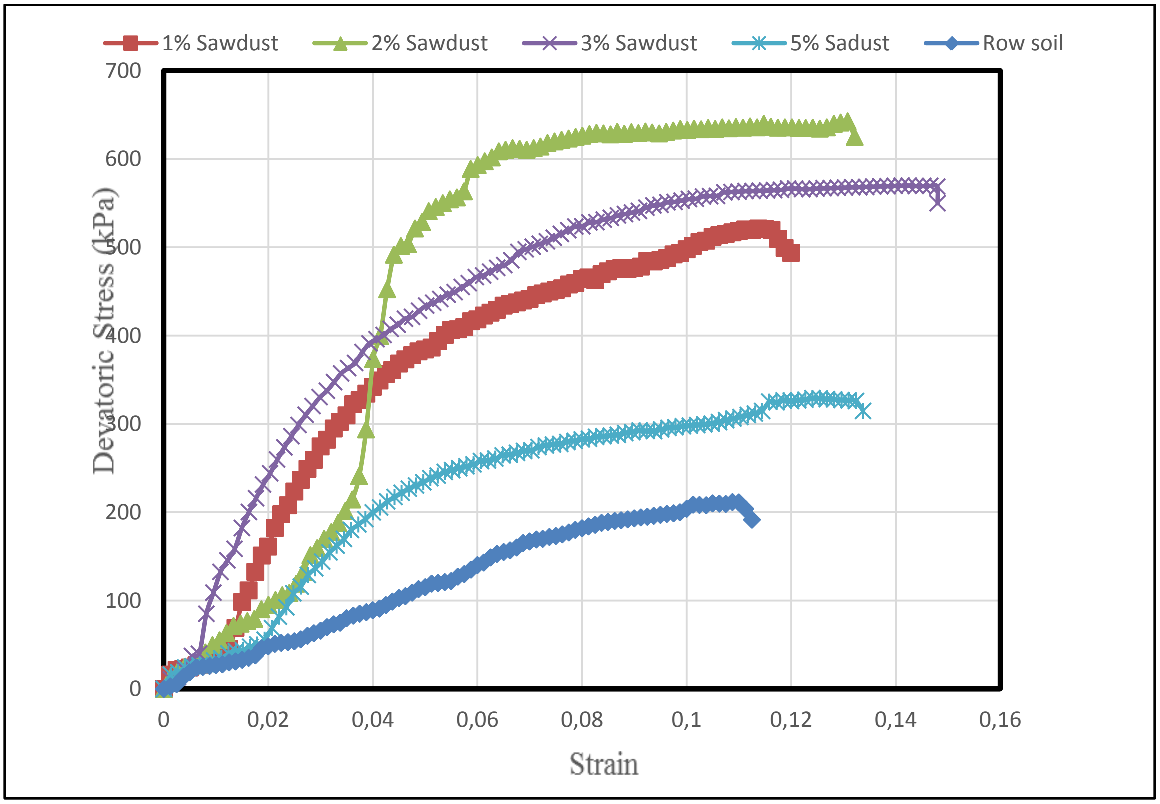

Figure 4.25 Long term effect of sawdust on stress-strain curve of the soil at confining pressure 160 kPa

Table 4.10 the long term effect of 160 kPa confining pressure on the soil-sawdust mixtures.

Sawdust content (%)	σ_3(kPa)	σ_1(kPa)	ε_f	Shear strength (kPa)
0	160	371	0.1110	107.5
1	160	681.23	0.1175	270.03
2	160	802.76	0.130	325.2
3	160	730.07	0.1479	280
5	160	489.34	0.1323	165

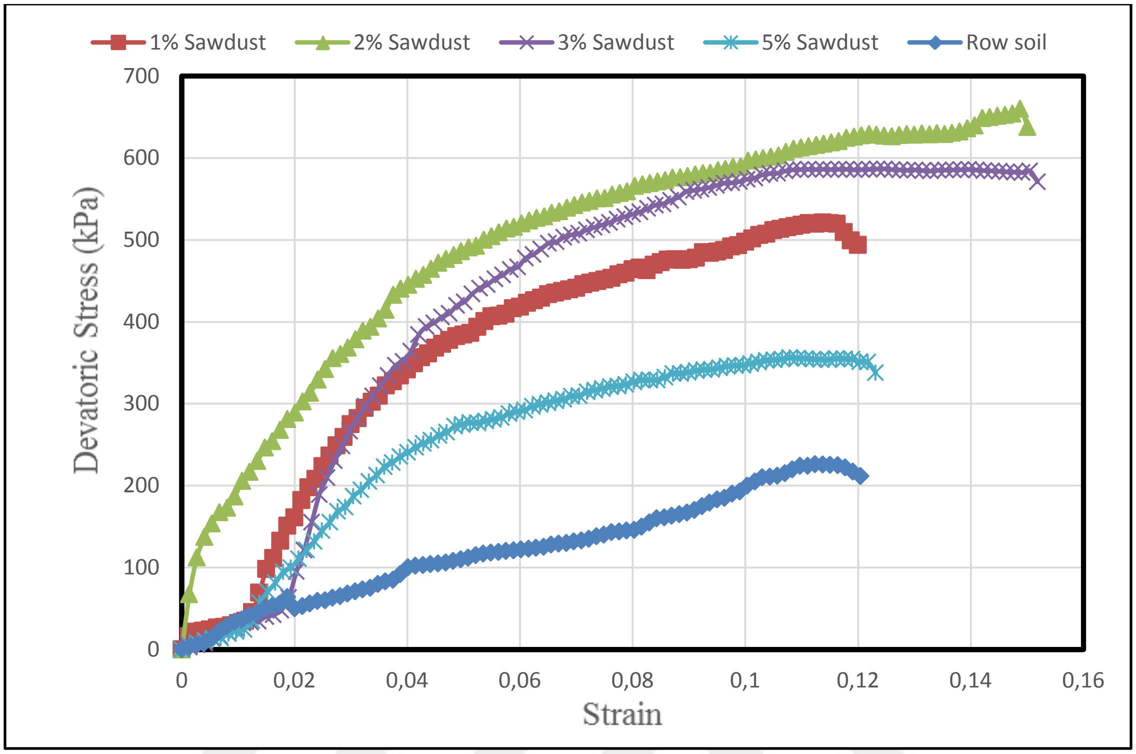

Figure 4.26 Long term effect of sawdust on stress-strain curve of the soil at confining pressure 320 kPa

Table 4.11 The effect of 320 kPa confining pressure on the soil-sawdust mixtures (long term)

Sawdust content (%)	σ_3 (kPa)	σ_1 (kPa)	ε_f	Shear strenght (kPa)
0	320	546.10	0.111	107.5
1	320	879.10	0.1163	270
2	320	960.27	0.1487	325
3	320	886.27	0.1518	280
5	320	676.15	0.120	165

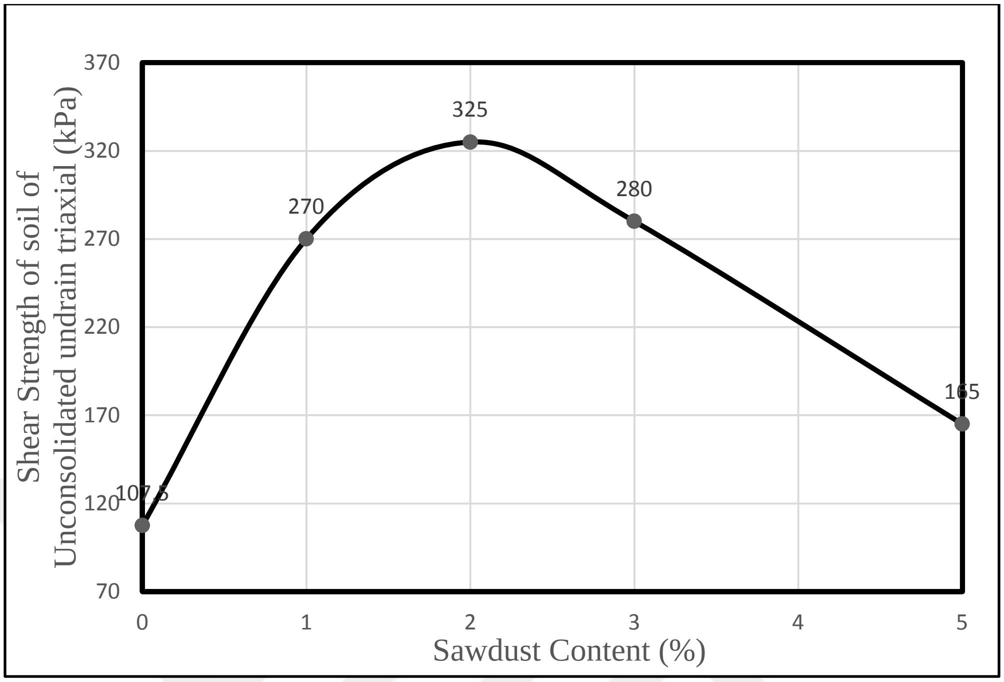

Figure 4.27 Long term shear strength against sawdust content

With respect to the immediate tests, 3% was optimum sawdust content, but over time exactly at 90 days, 2% is the optimum sawdust content Figure 4.27. As it is clear from the long-term tests of the shear strength resulting from unconfined unconsolidated triaxial test is higher than shear strength of unconfined compression test. The reason behind this is that sawdust absorbs water from the soil particles, sawdust swells due to the growth in the size, causing microscopically cracks and internal stresses within the soil mass. When confining pressure is applied, it tries to collect the particles and close those tiny internal cracks causing (increase) the difference in resistance. Figure 4.28 clearly graph a correlation between unconfined compression tests against unconsolidated undrained triaxial test for soil-sawdust mixture (immediate). Figure 4.29 shows a correlation between unconfined compression tests and unconsolidated undrained triaxial test for soil-sawdust mixture (long term).

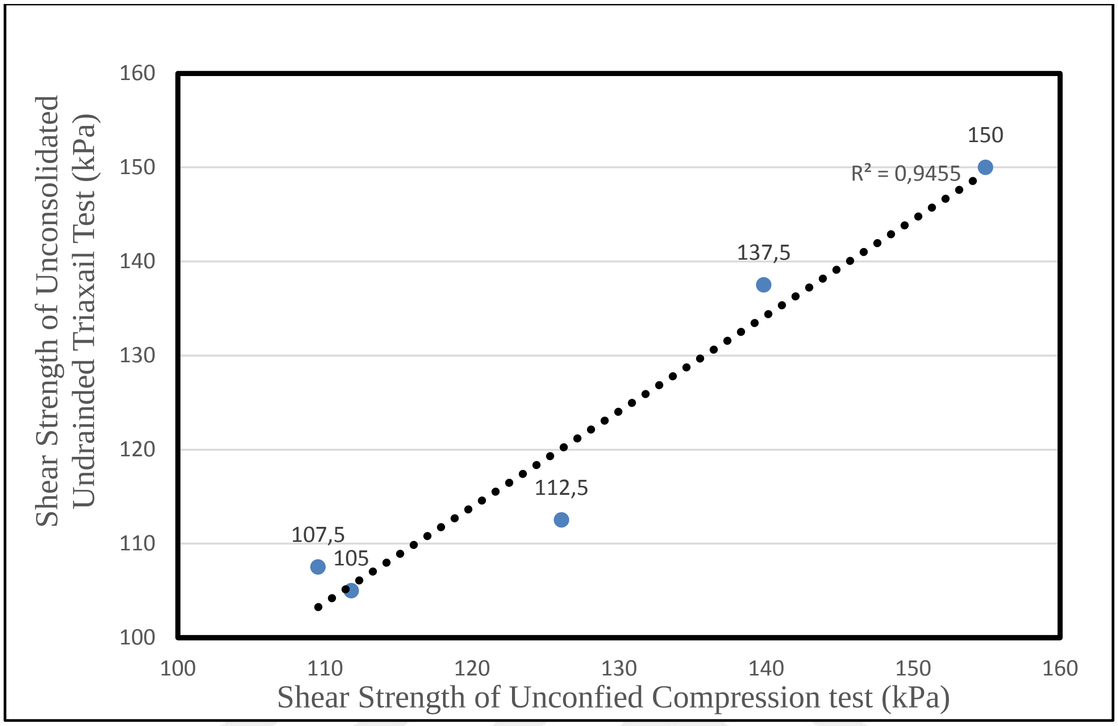

Figure 4.28 The correlation between unconfined compression tests against unconsolidated undrained triaxial test for soil-sawdust mixture (immediate)

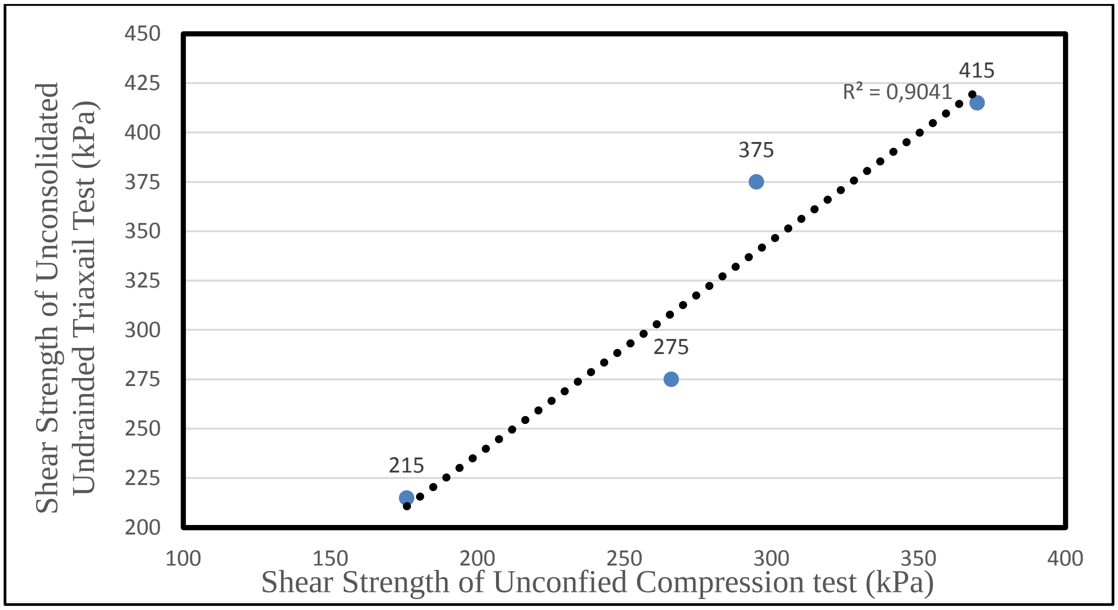

Figure 4.29 The correlation between unconfined compression tests against unconsolidated undrained triaxial test for soil-sawdust mixture (long term)

4.3.2 The Lime

The Immediate Behavior of Soil-Lime Mixtures

The amount of strength by adding lime to a clayey soil depends on the presence of the pozzolans exceedance [23], lime and pozzolans react rapidly to improve the soil-lime mixture. As soon as lime is added to the soil both modification and stabilization process starts together Addition of lime up to lime fixation point (optimum lime content), fixation is used for improving the strength [30]. the absolute required amount of Silica or Alumina to sustain the pozzolanic reaction in the soil is relatively small, thus clays generally show an increase during the process of adding lime as stabilized material [23] When pozzolanic reactions take place and consume lime in solution, equilibrium concentration gradients occur in the pore-water solution between the soil and cementing products must be produced between the more distant soil particles before there is any significant strength increase (Lees) [60]. The size of the particles also have an influence on the strength, by filling the voids between soil particles.

Long Term Behavior of Soil-Lime Mixtures

As the curing time is increasing, the shear strength of the cohesive soil is increasing [30], the reason behind that is over time series of reactions between lime and the soil take place and finish cementation process happen between lime and silica inside of the clay, in this way the strength increase.

Table 4.12 The difference in optimum water content in the immediate and long term tests for soil-lime samples.

Property / Soil type	O.M.C %	Actual water content during compaction % (0 days)	Actual water content during long term tests % (90 days)
1% Lime	40.3	40.16	39.40
2% Lime	40.97	41.08	40.13
3% Lime	41.61	41.5	41.21
5% Lime	41.69	42	41.32

Unconfined Compression Strength

4.3.2.1 Unconfined Compression Strength for Soil Lime (Immediate)

Data presented in the Figure 4.30, bring out that the shear strength of soil from unconfined compression strength of the soil-lime mixture increased with increasing the amount of lime. Figure 4.31. It was found that 5% was the optimum lime content as long as lime used to stabilize such kind of soils. Maximum values of stresses and shear strength were listed in Table 4.13.

The shear strength increased by 18.46%, 40.52%, 55.65% and 69.91% respectively when lime content increased from 1% to 5% as comparing with the row soil.

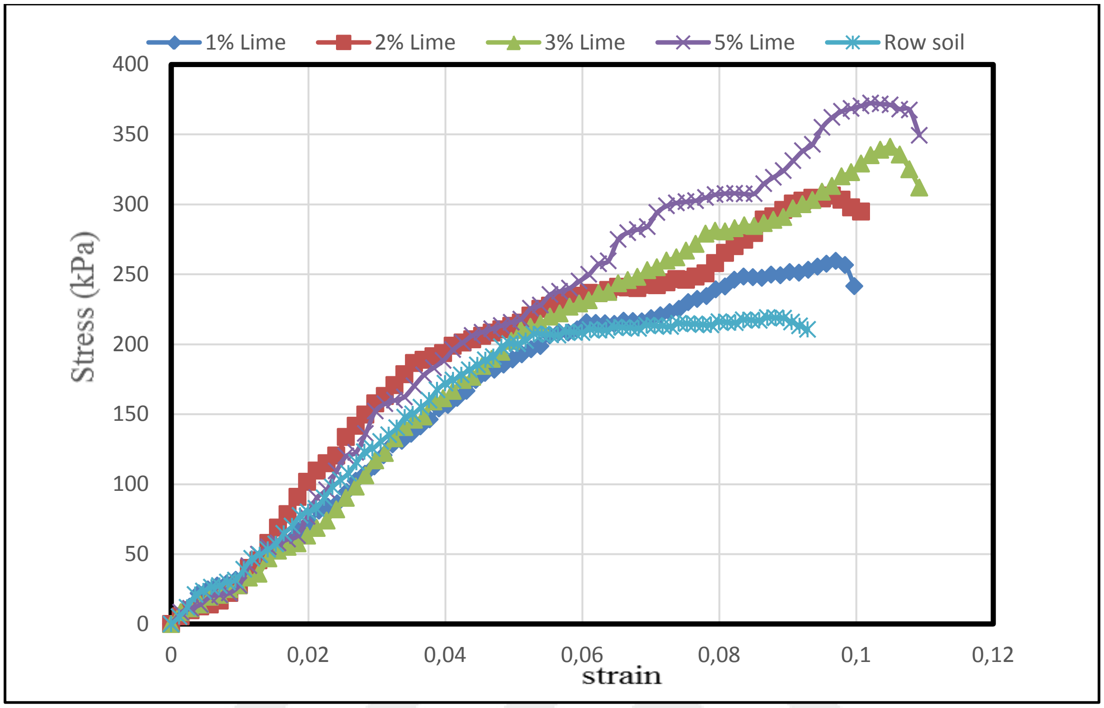

Figure 4.30 Stress-strain relationship of unconfined compression strength immediate tests

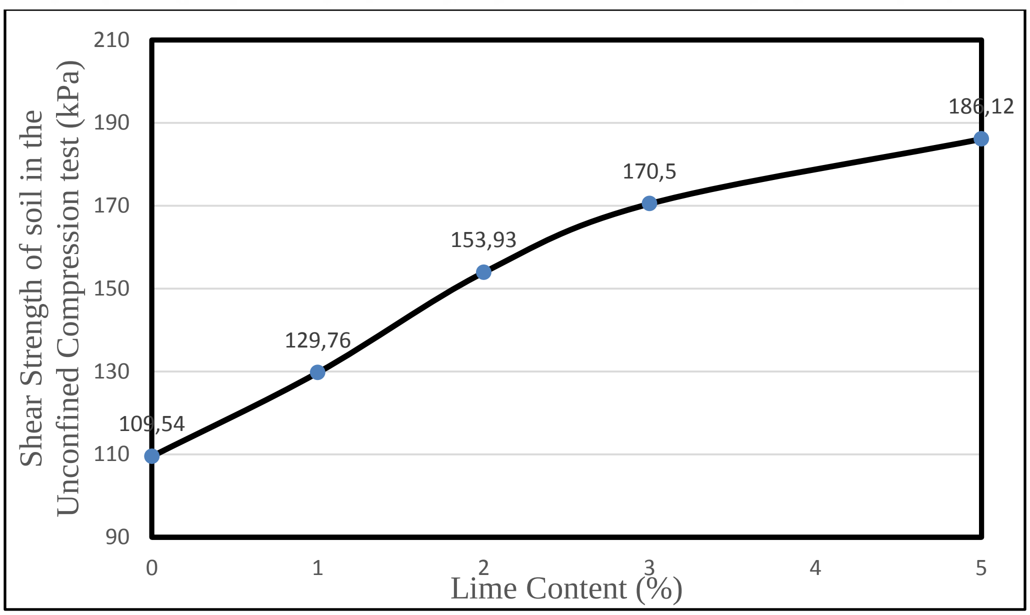

Figure 4.31 Immediate Effect of lime on the shear strength of soil in the Unconfined Compression test

Table 4.13 The effect of lime on the immediate shear strength due to unconfined test

property	Row soil	1% Lime	2% Lime	3% Lime	5% Lime
Stress (kPa)	219.08	259.53	307.85	340.98	372.24
Shear strength (kPa)	109.54	129.76	153.97	170.49	186.12

According to Bell [23], figures 4.32 and 4.33, the unconfined compressive strength of soil treated with lime develops rapidly, with increasing the amount of lime until optimum lime content.

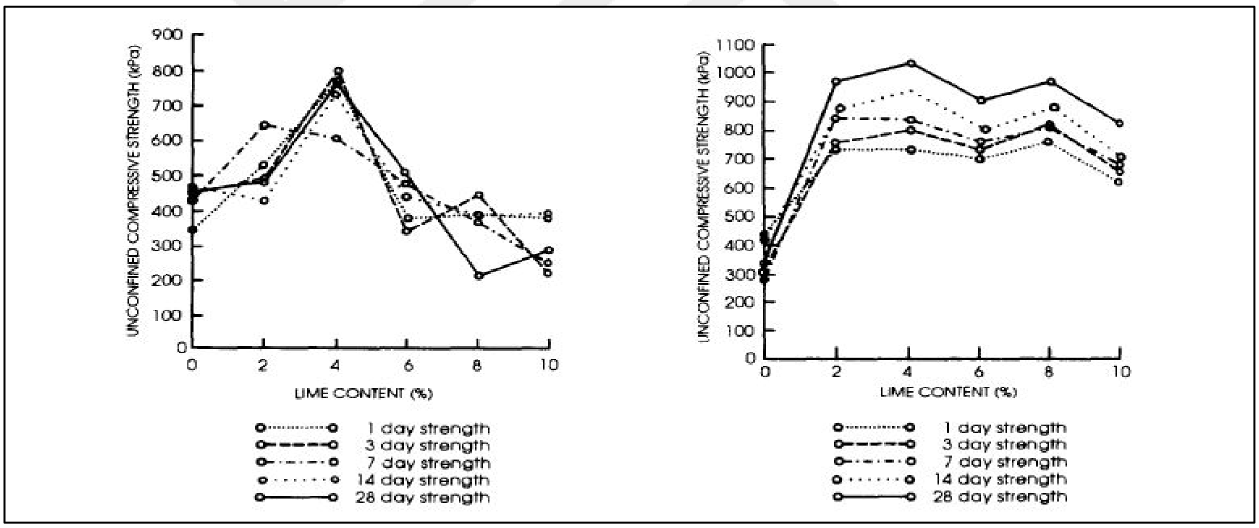

Figure 4.32 Relationship of the unconfined compression strength for Montmorillonite and kaolinite respectively and different lime content over time [23]

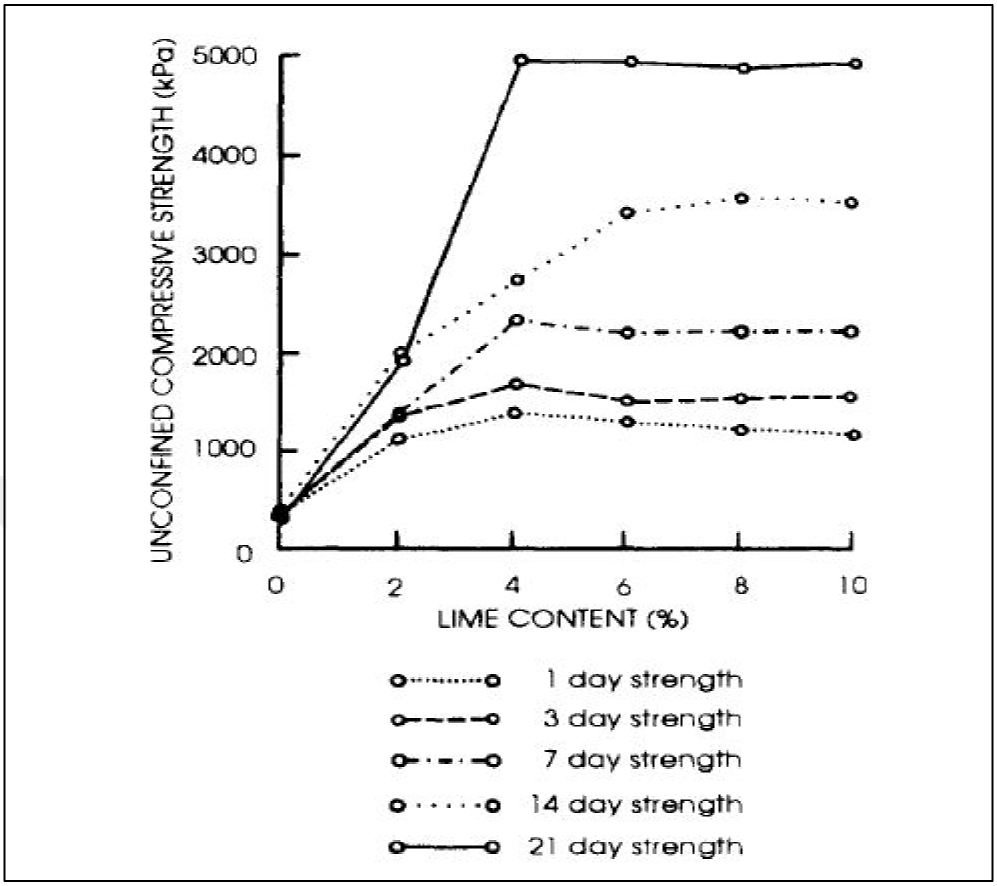

Figure 4.33 Relationship of the unconfined compression strength quartz with different lime content (Bell) [23]

4.3.2.2 Unconfined Compression Strength for Soil Lime (Long Term)

Similar behavior was observed in Figure 4.34 and 4.35, in the unconfined compression test in the long term in terms of lime usage for 4 different percentages (1, 2, 3 and 5) %, as simply can be seen from Table 4.14, lime increases the shear strength of soil. As comparing with the immediate tests, shear strength for 1% sawdust increased during 90 days from 126.09 kPa to 163.42 kPa 25.94%, during the same period of time 2% mixtures was increased up to 72.63%. As well as 3% increased by 72.12% and for 5%, the strength was increased by 102.63%.

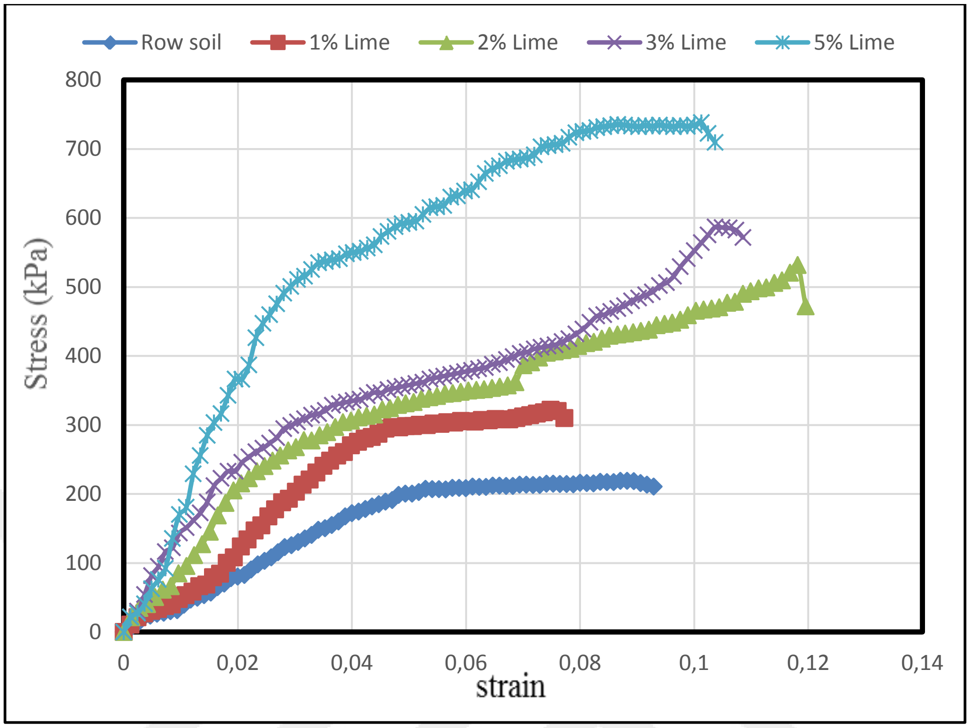

Figure 4.34 Stress-strain relationship of unconfined compression strength long term tests

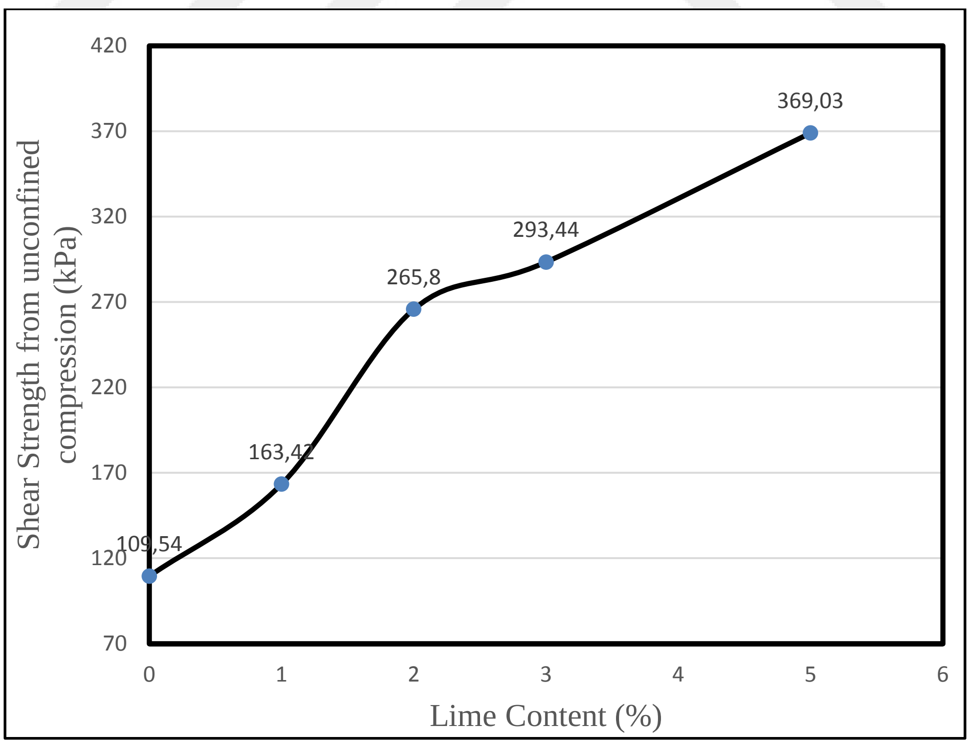

Figure 4.35 Long term Effect of lime on the shear strength of the soil in the Unconfined Compression test

Table 4.14 The effect of sawdust on the long term shear strength of soil in the unconfined test

property	Row soil	1% Lime	2% Lime	3% Lime	5% Lime
Stress (kPa)	219.08	326.54	531.60	586.88	738.05
Shear strength (kPa)	109.54	163.42	265.80	293.44	369.03

Unconsolidated Undrained Triaxial Test for Lime

4.3.2.3 Unconsolidated Undrained Triaxial Test of Soil Mixed with Lime (Immediate)

The Unconsolidated Undrained test was apply on lime-soil mixtures to evaluate the immediate effects of lime on the stress–strain relationships of the mixtures at the confining pressures of 160 and 320 kPa are shown in Figures 4.36 and 4.37, Table 4.15 and 4.16. Moreover, for the confining pressure 160 kPa, according to the Figure 4.38, the addition of a 1% lime increased shear strength by 20.93%, while the other two percentages (2% and 3%) increased the shear strength, 2% increased the strength by 30.23% with respect to the row soil and 3% sharply increased the strength up to 48.84%. as well as, increasing lime content to 5% increases the strength by 62.79%. A similar behavior was observed for confining pressures of 320 kPa. 5% was the optimum sawdust content during the immediate tests.

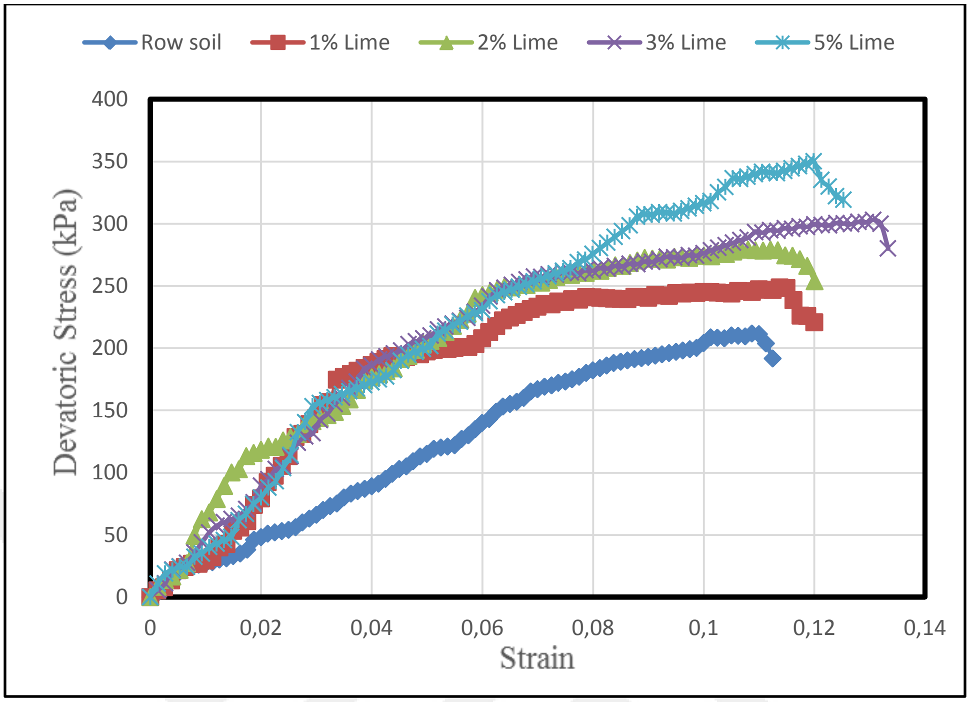

Figure 4.36 Immediate effect of lime on stress-strain curve of the soil at confining pressure 160 kPa

Table 4.15 the effect of 160 kPa confining pressure on the soil-lime mixtures

Lime content (%)	σ_3 (Kpa)	σ_1 (Kpa)	ε_f	c_u (Kpa)
0	160	371	0.111	107.5
1	160	408.66	0.11375	130
2	160	439.47	0.10672	140
3	160	488.45	0.1520	160
5	160	509.99	0.11992	175

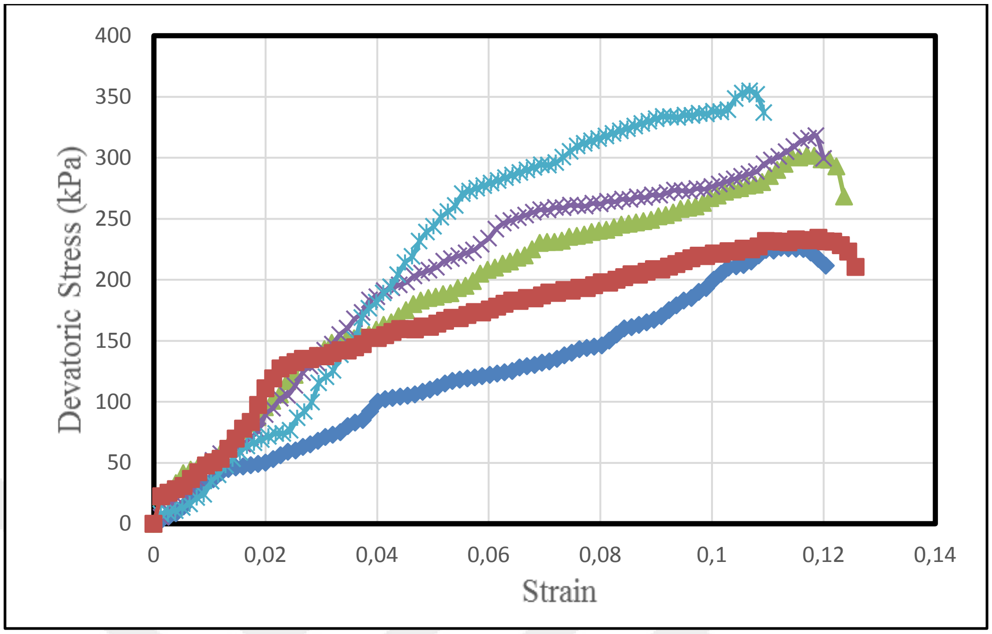

Figure 4.37 Immediate effect of lime on stress-strain curve of the soil at confining pressure 320 kPa

Table 4.16 The immediate effect of 320 kPa confining pressure on the soil-lime mixtures

Lime content (%)	σ_3 (kPa)	σ_1(kPa)	ε_f	shear strength (kPa)
0	320	546.10	0.111	107.5
1	320	554.37	0.11908	130
2	320	602	0.11702	140
3	320	637.73	0.1248	160
5	320	674.81	0.10675	175

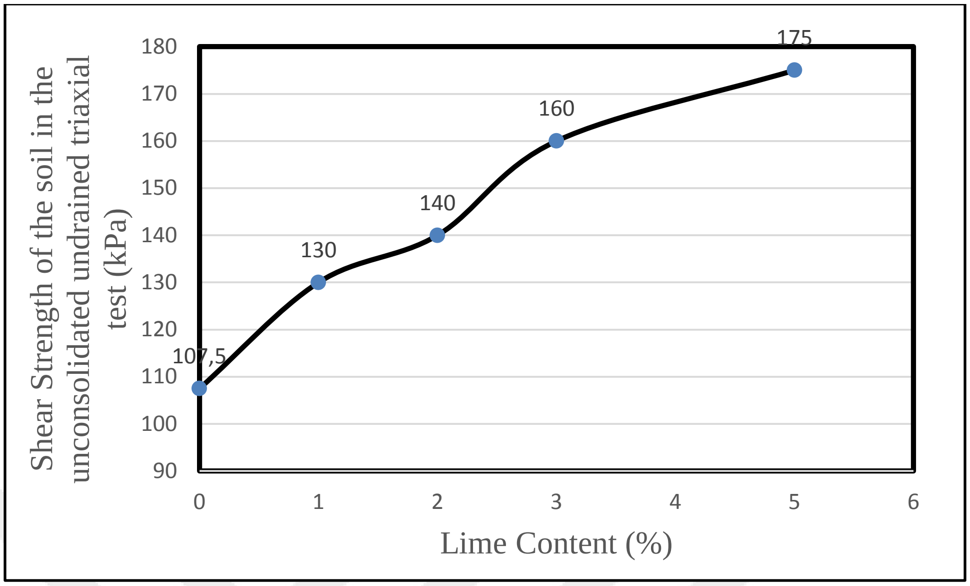

Figure 4.38 Immediate Effect of lime on the shear strength of the soil in the unconsolidated undrained triaxial test

4.3.2.4 Unconsolidated Undrained Triaxial Test of Soil Mixed with Lime (Long Term)

Two samples at least were kept in the desecrator, Figure 3.11, for 90 days (2160 hours) to investigate the difference which might happen in the strength due to the chemical reaction between lime and the soil, water content of soil samples was calculated to see weather if the increase of the strength happened due to the chemical reaction, loss of water content, or both of them. Table 4.12 shows the difference in the water content during immediate and 90 days tests same procedure of the immediate test was used, two confining pressure were used 160 and 320 kPa respectively. As comparing with the immediate test, 1% sawdust during 90 days increased the shear strength of the soil in the unconsolidated undrained triaxial test from 130 kPa to 225 kPa (73.08%), during the same period of time 2% mixtures was increased up to 90.429%. As well as 3% increased by 134.38%. And 5% lime increased by 134.29% .the shear strength of soil treated with lime increases with increasing lime content [30]. Figure 4.37 shows the long term strength and sawdust content. Figures 4.39 and 4.40 also Tables 4.18 and 4.19.

5% was the optimum lime content, it increases the strength during 90 days to 281.40% as comparing to the strength of row soil .Figure 4.41 shows long term effect of lime on the shear strength of the soil in the unconsolidated undrained triaxial test, Figure 4.42 clearly shows a correlation between unconfined compression tests against unconsolidated undrained triaxial test for soil-lime mixture (immediate) and Figure 4.43 the correlation between unconfined compression tests against unconsolidated undrained triaxial test for soil-lime mixture (long term).

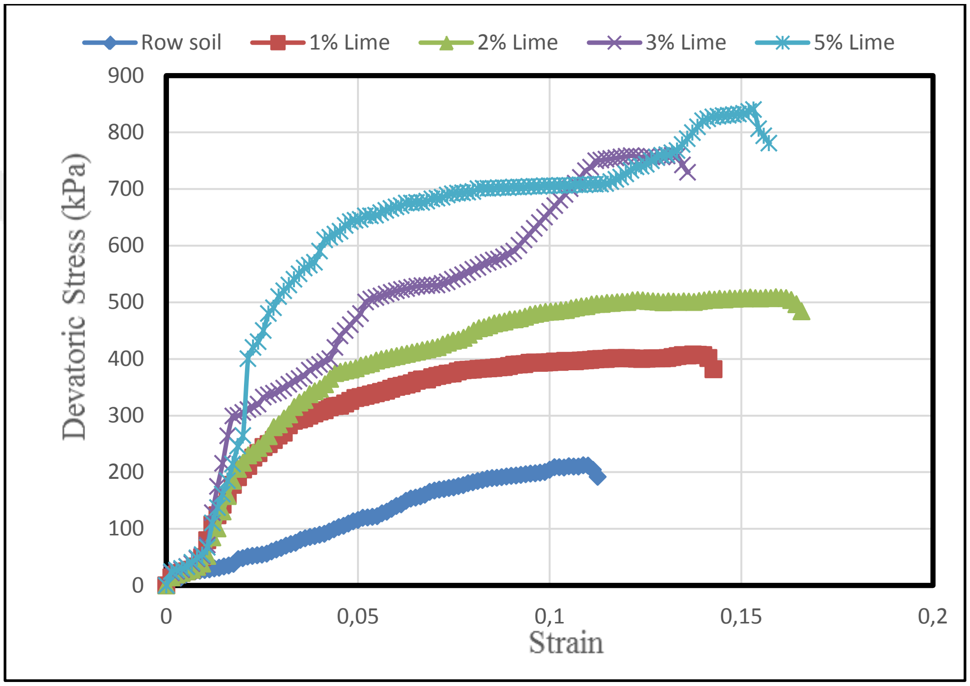

Figure 4.39 Long term effect of Lime on stress-strain curve of the soil at confining pressure 160 kPa

Table 4.17 The effect of 160 kPa confining pressure on the soil-lime mixtures

Lime content (%)	σ_3 (kPa)	σ_1 (kPa)	ε_f	Shear strength (kPa)
0	160	371	0.13741	107.5
1	160	567.68	0.115	225
2	160	668.43	0.15905	275
3	160	918.85	0.1320	375
5	160	999.56	0.15323	410

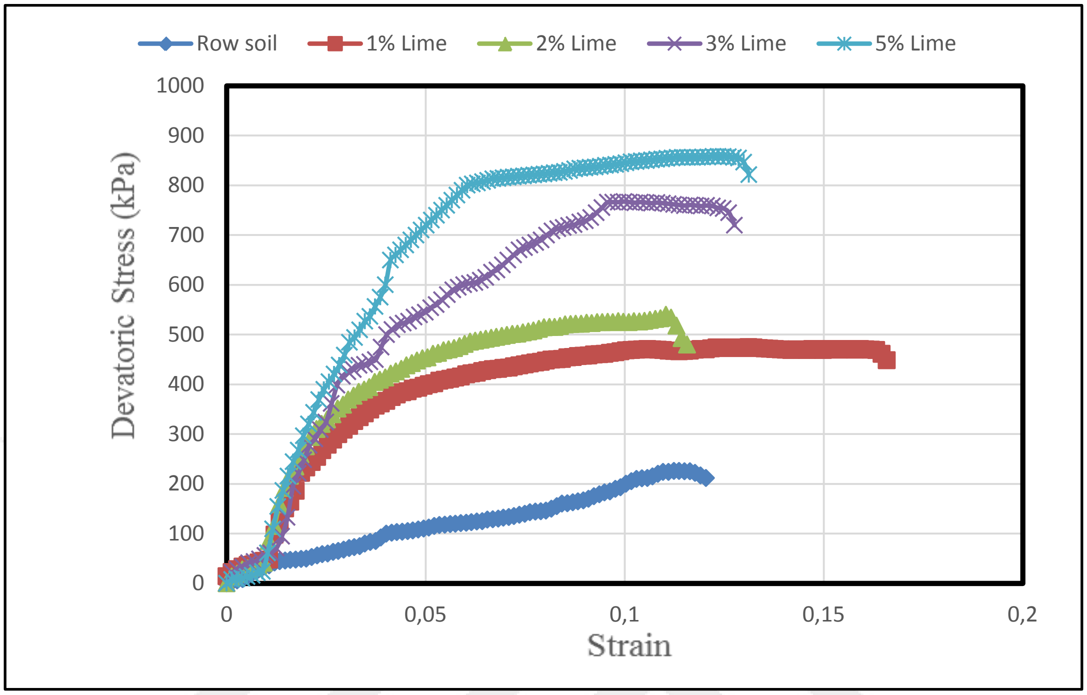

Figure 4.40 Long term effect of Lime on stress-strain curve of the soil at confining pressure 320 kPa

Table 4.18 The effect of 320 kPa confining pressure on the soil-lime mixtures

Lime content (%)	σ_3 (kPa)	σ_1 (kPa)	ε_f	shear strength (kPa)
0	320	546.10	0.11370	107.5
1	320	794.63	0.13112	225
2	320	889.46	0.11702	275
3	320	1087.46	0.09985	375
5	320	1177.24	0.12605	410

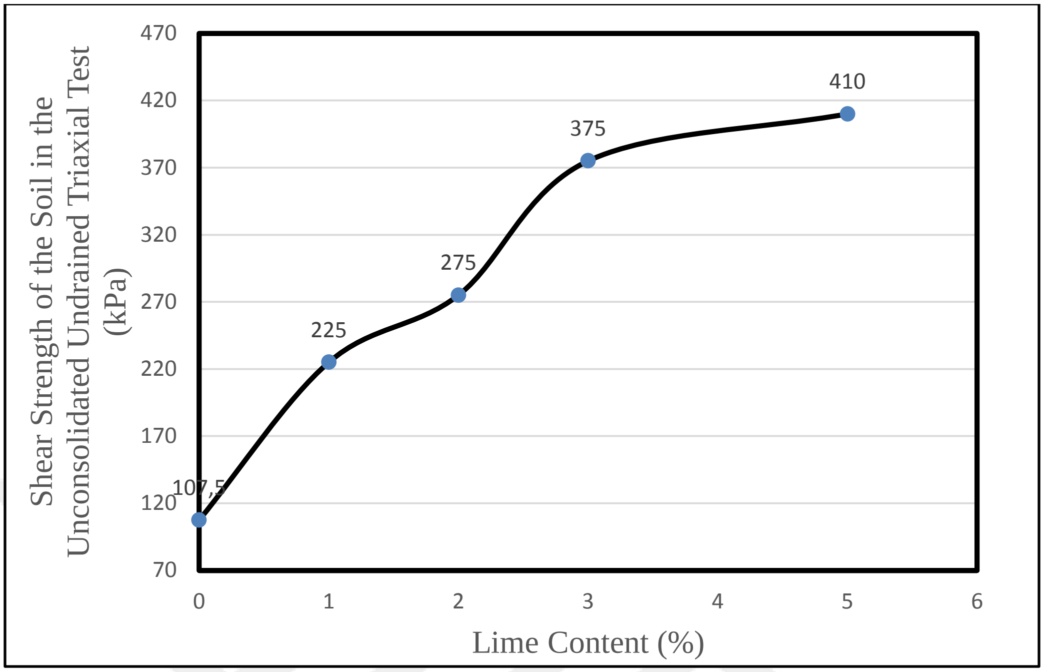

Figure 4.41 Long term Effect of lime on the shear strength of the soil in the unconsolidated undrained triaxial test

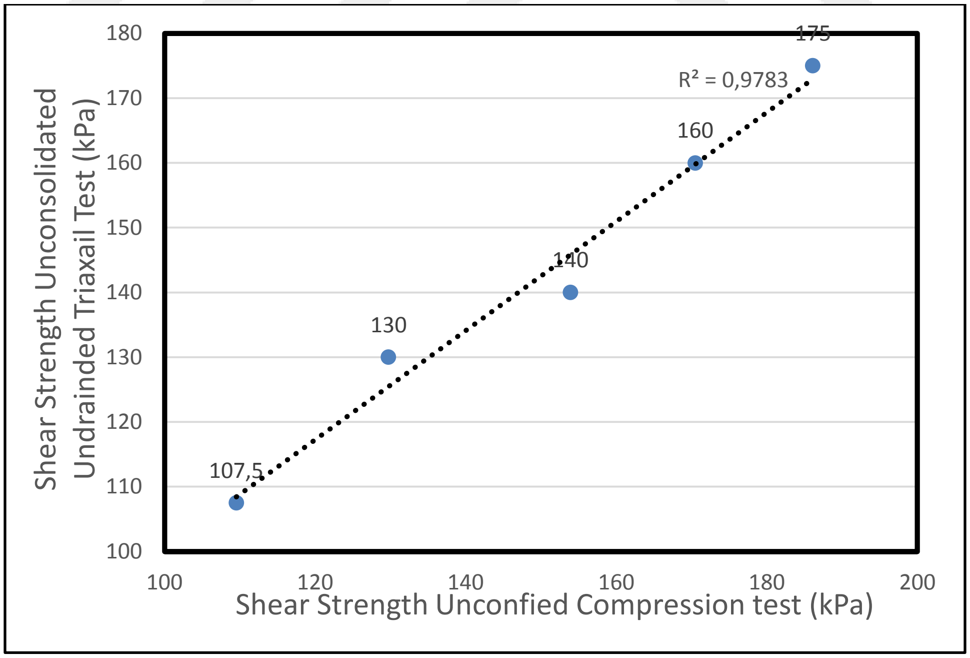

Figure 4.42 The correlation between unconfined compression tests and unconsolidated undrained triaxial test for soil-lime mixture (immediate)

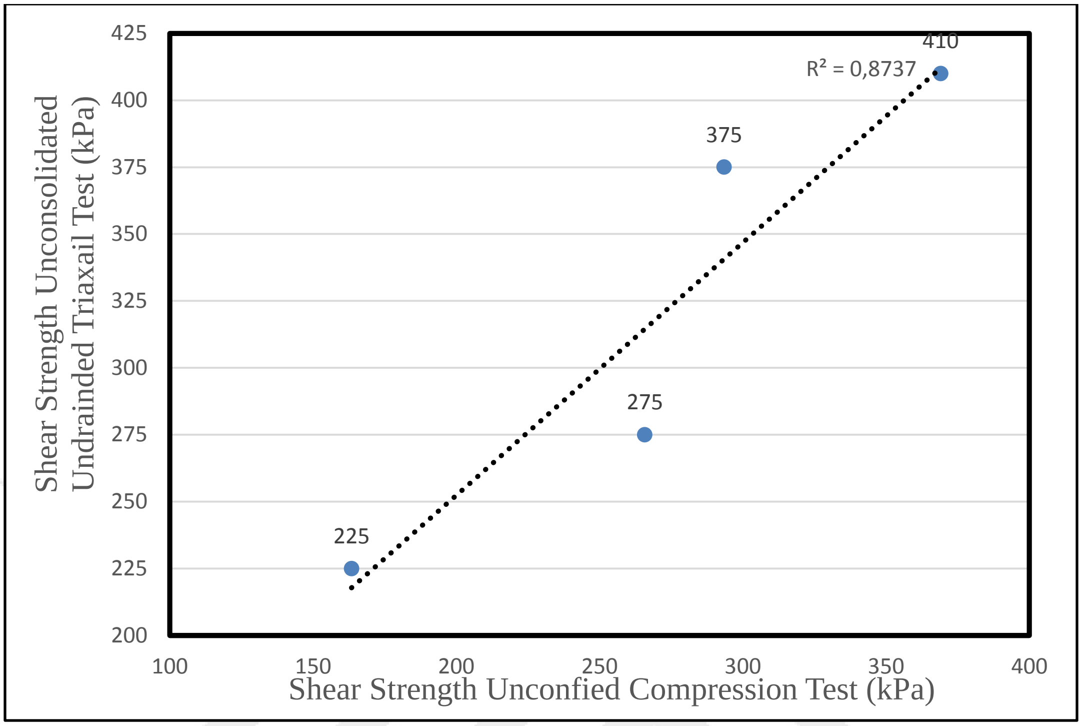

Figure 4.43 The correlation between unconfined compression tests and unconsolidated undrained triaxial test for soil-lime mixture (long term)

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

1. No specific behavior for the natural fiber additives, it might depends on the type of the soil and the moisture content of the soil and the sawdust as well and the distribution method. On the other hand, the behavior of lime is clear enough on the soil passed from sieve No.40.
2. The liquid limit, plastic limit and plasticity index values decreased with increasing sawdust content up to 5%.by 13.01%, 11.63%, and 15% respectively. The tests concluded that the addition of lime up to 5% decreased the liquid limits and the plasticity index by 14.52% and 52% respectively, but it increased the plastic limits by 11.63%.
3. Addition of sawdust to 5% reduces optimum moisture content and maximum dry unit weight by 9.7% and 7.59% respectively, same behavior was observed by lime in terms of maximum dry unit weight, it reduced by 4.05% while optimum moisture content was increased by 1.73%.
4. The immediate effect of sawdust increased the shear strength in both of (unconfined compression test and unconsolidated undrained triaxial test) more than 39% as increasing sawdust content from 0 to 3% then decreased, the immediate effect of lime as well increased the shear strength more than 62%.
5. Optimum sawdust content was 3% for the immediate tests, as comparing with the lime 5% for the same interval of time.
6. The long term of sawdust increased the shear strength about 115% (minimum) as increasing sawdust content from 0 to 3% then decreased by 0.6% in the unconfined compression test only, the long term effect of lime as well increased the shear strength by more than 163%.

7. Optimum sawdust content was 2% for the long term tests, but the optimum lime content was 5% for the same interval of time
8. Sawdust works as very good filler material, lime increases the cohesion of the soil because of the cementation property.
9. Sawdust and lime might be a very good stabilizer for high water content soils.
10. Over time, sawdust absorbs the water, it decomposes and swells, causes internal cracks in the soil samples. While lime increases the cohesion and reduces the cracks
11. Soil containing sawdust could be a suitable place for the growth of mold and bacteria it was seen a Bacteria on the high sawdust content sample (3 and 5) % and it reduced the strength. From the other hand there were not Bacteria on the samples of lime.
12. Sawdust might change in the physical properties of the soil and react chemically over time, while the lime reacts chemically and fill soil gaps.

5.2 Recommendations for Future Work

1. The possible effects of sawdust in the internal structure of the soil
2. Study the effect of sawdust on swelling, permeability and specific gravity of soils
3. Investigating the effect of sawdust on shear strength parameters using direct shear test, van shear test and other triaxial tests.
4. Study the behavior of sawdust-soil over different periods of time.
5. Investigating the effect of sawdust with lime on shear strength parameters.

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PUBLISHMENTS

Papers

1. Jasim O.H., and Cetin D., (2016) “Effect of Sawdust Usage on the Shear Strength Behavior of Clayey Silt Soil” Sigma J Eng & Nat Sci 34 (1):31-40

Conference Papers

1. Mahmoud A.S, Khalil Y.T., Jasim O.H., (2013). “Development of Normal Strength Concrete from Crushed Al-Ma'mora Stones”, Sixth International Conference on Developments in eSystems Engineering, DOI 10.1109/DeSE.2013.61, 303-307