# IMPROVEMENT OF STRENGTH PROPERTIES OF CLAYEY SOILS FOR CONSTRUCTION PURPOSES

**MASTER THESIS**

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**Mohammad Ehsan ALAMI**

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## CONTENTS

**Page No**

# ÖZET

## Killi Zeminlerin Dayanım Özelliklerinin İyileştirilmesi Ve Temel Alt Zemini Olarak Kullanılabilirliğinin İncelenmesi

Zemin iyileştirme yöntemleri, oturmaları azaltmak ve zeminin kayma mukavemetini ve taşıma kapasitesini arttırmak için en ucuz yöntemlerden biridir. Bu nedenle, bu tezde, kırıntı lastik parçalarını kil zemin ve kum zemin ile farklı yüzdelerde kullanarak zeminin dayanımı üzerindeki etkisi araştırılmıştır. % 100,% 80,% 70 ve% 60 kaolini kili %0, 20, % 30 ve %40 oranlarındaki kum ile karıştırılmıştır. Yapılan laboratuvar çalışmasında (% 5 kireç zemine eklenerek kirecin etkisi de incelenmiştir. zeminin optimum nem içeriğine (wopt) sahip standart Proctor sıkılığında sıkıştırılmasından sonra, numune 28 gün boyunca desikatöre koyulmuştur. 28 gün sonra serbest basınç deneyi yapılarak numunelerin mukavemeti belirlenmiştir.

Bu tezin ikinci bölümünde,% 100,% 80,% 70 ve% 60 oranında kaolin kilini % 0,% 20,% 30 ve% 40oranlarında kum ile karıştırarak, her biri karışım için, zemine belli oranlarda eklenen lastik çiplerinin (%0,%1, %2 ve %4 etkisi incelenmiştir. Tüm örnekler, Proctor sıkılığında sıkıştırılmıştır. Numuneleri hazırladıktan sonra, serbest basınç deneyi yapılarak numunelerin mukavemeti belirlenmiştir.

Deney sonuçlarına göre, zeminde kullanılacak kırıntı lastik oranının optimum değeri %2’dir. Bu değerden daha fazla kırıntı lastik kullanılması dayanımda aşırıdüşüşlere sebep olmaktadır.

Deneylerin ikinci kısmında, kil-kum karışımlarına eklenen %5 kirecin dayanımı artırdığı ve bu artışın bazı numunelerde %146 civarında olabildiği belirlenmiştir. Bu artış oranı karışımdaki kum oranını azaldıkça azalmaktadır.



**Anahtar Kelimeler:** Toprak İyileştirme, Kaolin Kili, Lastik Kırıntısı, Kireç taşı, Nehir Kumu.

**SUMMARY**

**Improvement of Strength Properties of Clayey Soils for Construction Purposes**

Soil improvement methods is one of the cheapest methods for foundation stability. This method has used for preventing the settlement, and increase the shear strength and bearing capacity of the soil. So, in this thesis, tried to use the tire crumbs with clay type of soil and sand in different percentages. The amount of, 60%, 70%, 80% and 100% of kaolin clay mixed with 40%, 20%,30% and 0% of river sand. Then the optimum percent of lime powder (5%) added to the soil. After compacting the soil at standard proctor density with optimum moisture content ($w_{opt}$), the sample had put in to desiccator for 28 days. After 28 days by unconfined compressive tests, the bearing capacity of the soil has determined.

In the second part of this thesis, mixture of 60%, 70%, 80% and 100% of kaolin clay mixed with 0%, 20%,30% and 40% of river sand after finding the ($w_{opt}$) for each sample, the amount of 0%,1%, 2%, %4 of tire crumbs by total weight added to the soils. All samples compacted at the proctor density. After preparing the samples, unconfined compressive strength test (UCS) of all samples were determined.

According to test results, it has founded that the optimum ratio of tire crumbs is 2%. After that, the strength decreases dramatically. Adding tire crumbs increases the unconfined compressive strength up to 56%.

In second part of the studies it has founded, that adding 5% of lime powder to the kaolin clay-sand mixture can increases the unconfined compressive strength parameter up to 146%. However, the strength ratio is directly depends in sand percentage. While decreasing the sand percentage the strength ratio also decreases.

**Key Words**: Soil Improvement, Kaolin Clay, Tire Crumbs, Lime Powder, River Sand.

## LIST OF FIGURES

**Page No**

## LIST OF TABLES

**Page No**

## SYMBOLS LIST

| | |
|---|---|
| **(AASHTO)** | : American Association of State Highway and Transportation Officials |
| **A** | : Area ($m^2$, $cm^2$) |
| **µ** | : Micro particles size |
| $A_f$ | : The cross-sectional area of the soil sample during failure |
| **ASTM** | : American Society for Testing and Materials |
| **BAP** | : Best Administration Project |
| **c** | : Cohesion |
| **CBR** | : California Bearing Ratio |
| $C_c$ | : Coefficient Of Curvature |
| **CH** | : Inorganic clay of high plasticity |
| $C_u$ | : Coefficient Of Uniformity |
| **d** | : Diameter |
| $D_{10}$ | : Effective Grain Size |
| $D_{30}$ | : Particle size at 30 % finer |
| $D_{50}$ | : Median Grain Size |
| $D_{60}$ | : Particle size at 60% finer |
| **Dr** | : Relative Density |
| **e** | : Void Ratio |
| $e_{max}$ | : Maximum Void Ratio |
| $e_{min}$ | : Minimum Void Ratio |
| $\gamma_{d(max)}$ | : Maximum Dry Density ($kN/m^3$) |
| $\gamma_{d(min)}$ | : Dry unit weight of the soil at its loosest condition ($kN/m^3$) |
| $\gamma_n$ | : Bulk Density ($kN/m^3$) |
| **Gs** | : Specific Gravity |
| $H_0$ | : Initial Height |
| **LL** | : Liquid limit (%) |
| **Mpa** | : Mega Pascal |
| **°C** | : Degree of Celsius |
| **°F** | : Fahrenheit Degree |
| **OMC** | : Optimum Moisture Content % |

| | |
|---|---|
| **pH** | : Degree of acidity or alkalinity |
| **PI** | : Plasticity Index (%) |
| **PL** | **:** Plastic Limit (%) |
| **$P_{max:}$** | : The maximum load at break point |
| **$q_u$** | : Shear Stress, Free Pressure load |
| **SP** | : Poorly Graded Sand |
| **SS** | : Stabilized Soil |
| **UCS** | : Unconfined Compressive Strength |
| **UNEP** | : United Nations Environmental Program |
| **USCS** | : Unified Soil Classification System |
| **V** | : Volume |
| **W** | : Weight |
| **w/c** | : Water Content % |
| **$\gamma_d$** | : Dry density ($kN/m^3$) |
| **$\Delta H$** | : Changes in height |
| **$\varepsilon$** | : Shear Displacement |
| **$\sigma'$** | : Effective stress |

## 1. INTRODUCTION

Soil is not most of the time a favorite material for the construction works. Such problem can be settlement, low shear strength, soil in high permeability. In this case, the engineer should decide to change the position of the building or apply cheapest way to increase the mechanical properties of the proposed area.

For instance in last century, after many earthquake catastrophes, with high casualties the necessity and need of soil improvement become more and more popular. As the soil, investigation will cost lots of money in the construction sector. The engineers are trying to find some economical way to lower the costs of construction and reduce the duration of construction work. Therefore, the need for soil improvement come to existence. There are certain type of soil improvement methods, which may be adding some extra additive materials to the soil, like lime powder, chemicals, cement, fly ash to increase the bearing capacity of the soil to be prepared for construction.

There are a lots of soil improvements technique, but the engineers are selecting, a cheapest and eco-friendly technique, to improve the weak soil. One of these technics are soil stabilization with lime powder and tires crumbs. Because tire crumbs are waste materials which cannot be recycled again. Secondly, it will stay in the soil for long time without rotting. In addition, the mixture of tire crumbs cannot mixed with soil structure and it will remind in the soil for long time without any changes. So, because of its high durability can increase the soil tensile strength, stability, durability and reduces the settlement and permeability rates.

In this research, kaolin clay with river sand taken from Murat River, Elazig Turkey are used. The experiments are based on a mixture of tire crumbs with kaolin type of clay and river sand in different ratios. The amount of 60%, 70%, 80% and 100% of kaolin clay soil mixed with 40%, 30%, 20%, and 0% of river sand respectively. Then the optimum percent of lime powder (5%) added to the soil. Afterwards, to achieve the maximum density the standard proctor test, applied to the each sample. With the same optimum moisture content ($w_{op}t$), the samples had placed to desiccator for 28 days. After 28 days by unconfined compressive test, (UCS) applied to the samples to find out, the maximum bearing capacity of each sample.

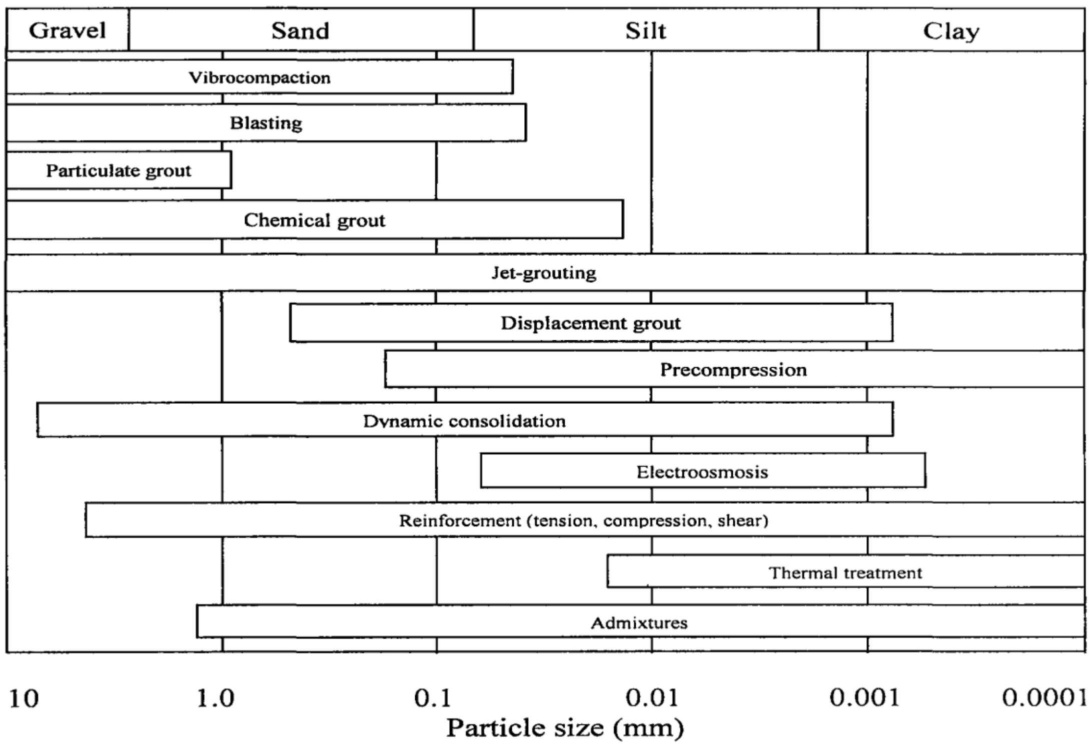
In the another part of this studies, mixture of 60%, 70%, 80% and 100% of kaolin clay mixed with 40%, 30%,20%, and 0% of river sand after finding the optimum water content ($w_{opt}$) for each sample, the amount of (0,1, 2, 4)% of tire crumbs added to the soils. All samples compacted at the proctor density. After preparing the samples, unconfined compressive strength of all samples determined.

## 2. SOIL IMPROVEMENT METHODS

Because of soil complex structure and behavior, it is very hard to classify soil improvement methods. Bergado (1992) categorized ground improvement methods to four parts, a) Reinforcement, b) grouting (cementation), c) compaction, d) extraction waters from voids). Most of these techniques and mechanism had changed but the principles still are the same. Cabalar et al. (2009) divided the soil improvement methods to four types, extracting voids from the mass of soil by applying static loads, consolidation or extracting water from the mass, adding some additive to the soil mass, and warming or freezing the soil mass.

The soil stabilization method had been divided into the fallowing categories by (Askay ,2002) as shown in the Table 2.1.

**Table 2.1.** Soil stabilization methods for various type of soil, by Askay (2002)




### 2.1. History of Soil Mixing

The concept of adding chemicals to soil goes back to 50 years in USA. In contrast, the current soil mixing technics and skills had developed in Europe and Japan in the last 50 years.

Seventy years back in USA the technique of mixing the soil on the site had come to existence, The piles had driven into the soil mass to improve the bearing capacity of the constructed area. Ten year's latter in Japan and Sweden, research had done on how adding different materials can increase the soil strength parameters. The aim was to find the result for mixing the soil in depths of ground, which included site test plus laboratory. Six years after many new mixing methods were invented, which included mixing soil in the wall and shallow for slope stability, mixing lime and cement with soil at depths. Afterward in 21$^{th}$ century various methods for soil improvements were developed, soil mixing with different materials like lime, cement, fiber etc... , injection with high pressure, adding geo-textile, and geo-membranes, many type of pile foundation, etc. (Caraşca, 2015)

### 2.2. Soil Compaction

By increasing the building constructions in the last decades, soil compactions attract much attention, especially in roads, bridges, and buildings construction. Compaction is a process, which extracting the air voids from the soil mass by applying loads to the mass of soil, so the soil particles are coming in a dense state, and it is one of the most significant way for soil stabilization (Xia, 2014). The process depends on soil properties for example, soil texture, water content, soil organic content and soil aggregates. (Ekwue and Stone, 1995, Kay et al., 1997, Canbolat et al., 2002, Imhoff et al., 2004)

The first aim for soil compaction is to increase the bulk density, shear strength, and bearing capacity for construction purposes (Mosaddeghi et al, 2000). The second goal is to decrease the settlement and permeability rates. The shear strength, make an important effect to soil bearing capacity, which changes with water content and dry density. While water content increases the shear-strength increases up to optimum moisture point, after that adding water decrease the shear strength of soil. Therefore, the compaction process will enable the soil surface to carry extra load and react stronger to the future loads, and settlement cases.

#### 2.2.1. History of Compaction

The compaction process had applied through the eras for constructing roads, buildings, bridges, dams etc... However, in 20th century due to the industrial revolution and necessities for highways and buildings, the technology and techniques changed rapidly.

The evidence had shown that compacting soil for construction process started in old city of Babylon (Iraq), Roman Empire (Italy) and Egypt. They run big cylindrical stone on the ground to make ways for passing the horses. In 19th century as earliest compaction machine in United States, and England sheep were token to the field few time to compact the soil for construction purposes and that was one of the causes for innovation of Sheepsfoot rollers. (Elizzi, 1965)

In the start of 19th century the necessity for city planning become more and more. Therefore, in France the Engineering Corps Institute had created, the aim of this institute was to train engineers for pavement construction purposes. Later in 20th century, after the First World War the evolutionary development of compaction mechanization started, many rollers, vibratos and compacting machineries had invented. In addition to, an American scholar by the name of Porter during 1930, discover the California Bearing Ratio (CBR) test to determine the mechanical strength properties of the soil by penetrating to the soil surface. In the same period, in Los Angeles, USA a civil engineer by the name of Proctor studied the relationship between the dry density, moisture content and applied energy. He draw a curve by the name of Proctor curve, which shows the relationship between the maximum dry density and water content. Soon his publication accept by American Association of State Highway and Transportation Officials (AASHTO) and Porter accepted the Proctor compaction density test as a standard for soils before preforming CBR Test. Nowadays, the advanced compactors machine is use for soil stabilization and the process of modernizations still keep going on. (Ebels, 2004)

### 2.2.2. Type of Compaction

According to the soil classification and characteristics, the compaction type changes as soil the soil type changes. Mainly there are two different ways of the compacting methods used for different type of soil. For cohesive soil (clay, silt) commonly, the dynamic loading method is used while for non-cohesive soil such as gravel and/or sand vibrator compacting way is more common. (Xia 2014). So according the ASTM standards, we have only two standard tests for preforming compaction. A). Standard Proctor Tests B). Modified Proctor Test.

### a) Standard Proctor

The Standard Proctor Test is preformed to discover the relationships between the dry density and soil moisture content by drawing Proctor curve. The hammer weight of 2.49 kg drops from height of 305 mm to the soil mass. Soil prepared in three layers in a mold with (944 $cm^3$) volume, each with 100 mm height. Twenty-five blows of hammer will drop to the each layer, which produces the 600 kN-m/$m^3$ compressive force. (ASTM D698-07)

### b) Modified Proctor Test

“Modified Proctor Soil Compaction Test’’, is referring to the standard proctor test, but the only difference is in dropping hammer with different height and weight. The hammer of 4.53 kg falling down from height of 457 mm to five layers of subjected soil in a mold with (2124 $mm^3$) volume. Which generates the effort about 2700 kN-m/$m^3$, in which the soil mass compacted in high density. (ASTM D698-07)

## 2.3. Studies Related to Research

Ali and Mohamed (2018) had found the reaction of lime with clay type of soil in different temperature. The ratio of 5 %, 7%, 9 %, 11 % and 13 % of lime add to the clay samples. The sample prepared in constant ratio of dry unit weight of 12.15 gr/$cm^3$ and water content of 40 %. The environment temperature had limited to 68 and 108°f (Fahrenheit degree). The unconfined compressive strength (UCS) result presented that, due to the high temperature and the fast chemical reaction between lime powder and clay particles, that sample which had papered in 108 °f have eight times more bearing capacity than the samples, which prepared in 68 °f.

Temga, et al (2018) had studied the mechanical properties of the sand, clay and lime mixture. The amount of (0%, 25%, 50% and 100%) clay with (75%, 50%, 25% and 0%) sand and (0%, 4%, 8% and 12%) of lime mixed with each other. The finding shows that the sand ratio increase the strength parameters of clay soil, beginning from 1.4 to 3.2 MPa, and adding lime (4-12%) with clay, it changes the values form 0.4 MPa to 5.1 MPa. In the same time, the highest value of 5.1 MPa was obtained by mixing the 25 % of sand with 12% of lime. This mixture can improve soil strength parameters significantly but it does not have enough resistance against the water.

Sharma, et al (2018) in their article had discussed about the mixing lime and cement with mountainous soil. It had shown that adding cement to the soil is more effective than adding the lime. However, if the 5% lime mixtures with soil keep for 28 days it can increase the durability from four to six times. Therefore, it can be a cheap solution for improvement of mountainous soil for highway and road constructions.

Muhmed and Musa, (February 2017) started working with a mixture of hydrated lime with the kaolin clay, unconfined compressive test (UCS) was carried out in 0, 7, 14 and 28 days. The water content change in three different values, optimum moisture content ($w_{opt}$=33.3%), wet moisture content ($w_{wet}$ = 37.5%), and dry Moisture content ($w_{dry}$ = 29.9%). The results indicate that after 14 days, strength changes form 189 kPa to 205.2 kPa in 28 days at wet moisture content. In addition, in dry moisture content, it changes from 252.25 kPa to 344.6 kPa in 28 days, but the maximum values were determined in ($w_{opt}$), where 183 kPa to 244 kPa and in 14 days and 28 days it is 390 kPa. The researchers find out that the upper or lesser water content then the optimum moisture content ($w_{opt}$) drops down the (UCS) values. Figure 2.1.  Shows a descriptive effect of moisture content on unconfined compressive strength of kaolin clay

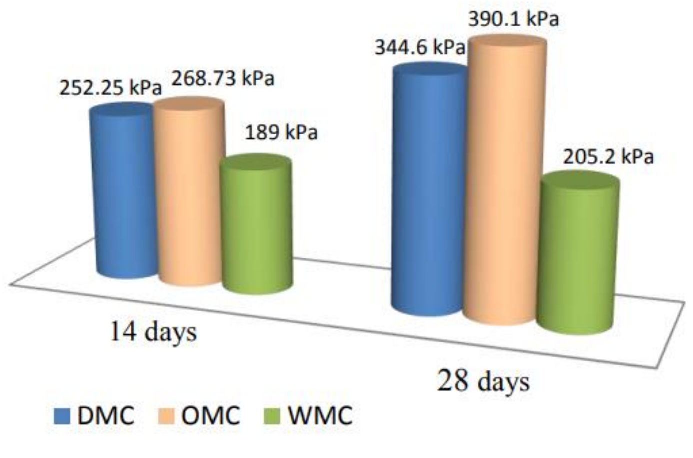



**Figure 2.1.** Effect of moisture content on UCS strength of kaolin clay

Yadav and Tiwari (2017) in their work had found that mixing rubber with cement in soil can reduces the dry unit weight and increases the durability. They mixed 0.8-2 millimeter small rubber crumbs in amount of 0%, 2.5%, 5%, 7.5%, and 10%, with 0%, 3%, 6%, cements in the clay type of soil. Their experiment had done after 7, 14 and 28 days. The findings shows that adding fiber with cement from 5% up 7.5 % can increase the unconfined compressive strength. While in California, bearing ratio (CBR) test the optimum point is 2.5% and after that, the resistance against penetration reduce when adding the rubber. In the same time the dry unit weight also decrease by adding the rubber and cement. Such material can be used as filing material for low traffic roads, slopes, and canal fillings.

Canakci and Celik (2016) had tested the, behavior of clay soil with mixing soda lime glass powder. In the beginning, the lime glass powder passed from sieve number #200 and added to clay in 3 %, 6%, 9% and 12 % values. Strength had tested in, 7, 14 and 28 days. The result shows that adding 12 % lime glass powder bring down the soil swelling potential from 5.5% to 1.65 present. As well as increases the strength properties of the soil and positively change the maximum dry density (MDD), optimum moisture content (OMC), liquid and plastic limit and (CBR) properties of the soil.

Qiang, et al (2014) had mixed fiber with clay soil to determine the (UCS), permeability, fragileness , and shear stress values of subject soil. 0.3 % fiber added to the samples and had waited for 25 days to dry. The cracking factor decrease up to 1.79 %. In addition, the resistance against permeability had a high decrease with mixing the fiber. Water in the normal sample in 10-millimeter rainfall per day had entered to 15 centimeter depth of soil. In the contrast, in the samples with 0.3 % fiber at 50-millimeter raining per day water had not get to 15-centimeter depth, In addition, the unconfirmed compression strength reaches to the 0.459 MPa with internal friction angle of 21.6 degree. As this soil had high resistance against permeability, it can be used as filling materials in dams and highway projects in places with high humidity.

Mohamed (2013) added the wheat hay to clay soil to determine the strength and swelling parameter of soil. The mixtures were 0.5%, 1%, and 1.5 % according to the soil weight. The unconfined compressive test result shows that the strength properties of soil improve up to 1% of hay addition and the swelling decrease up to 20 %. In addition, by adding hay to the soil the dry density, liquid limit, plastic limit, reduces. In the same time, the tensile strength rise up to 30 % and the shrinkage values dramatically changes after adding one percent of the hay.

Jafari and Ashari (2012) prepared a mixture of the nylon fiber with lime and clay soil. The ratio of mixing was 0%, 4%, and 8% lime and 0%, 0.5%, 1%, and 1.5%) fiber respectively added according to the total weight. The unconfined compressive tests were carried out in frozen sample. By adding the 1.5 % fiber and 4% of lime to the soil, strength increase to the optimum while in 8 % lime and 1.5 % fiber it had decreased significantly.

Huat, et al (2005), did experiments by mixing the lime and cement in tropical organic soil. The amount of 5 -15 % cement with 5% up to 25 % lime added to the soil. The results show that the addition of cement and lime increase the bearing capacity and unconfined compressive value of the soil but reduce the liquid limit values of the tropical organic soil. In addition, the soil dry density is also had a significant increase while the water content decreases with adding the cement and lime.

Bell (1996), carried out some tests such as (CBR) and (UCS) in the kaolin, montmorillonite and quartz sand. The samples were mixed with 0%, 2%, 4%, 6%, 8%, and10 % of lime. By adding the lime, the physical property in kaolin and montmorillonite had improved, and the plasticity properties decreased. The result have shown the young modulus with the hardness had significantly increased.

### 2.3.1. Usage of Waste Tires at Soil Improvement

Increasing numerous number of old tires each year in different parts of the world is a big problem toward our eco-system. For instance, Masad et al. (1996) specified, in his research that every year 278 millions of rubbished tire added to the total deposed of two billon waste tire in USA. A mixture of 50% sand and 50 % tire and 70% sand and 70% small chips had done in his research. The size of the tire chips was about 4.75 mm, and a serious of test such as (permeability, consolidation, specific gravity, void ratio, and tri-axial test) had conducted. The final founding shows an upgrading value while adding the tire chip to the sand. Moreover, the investigator strongly suggest that the mixture for lightweight filling material in highway constructions.

(Dickson et al., 2001), mentioned that used tired is not an ecological issue but is a financial problem due to not recycling of the tires. Over 500 million tires in USA and 27 million in Canada throw out each year. (Yoon et al., 2006) about 30 percent of these tire are entomb while 70 percent are used in second type of industry. Edincliler (2008), yearly three million tons of rubbished tire are added to deposits in Europe, due to the lack of space for depositing to this amount must reduce to certain amount. So one of the ways which can be recycle is using in soil stabilization and construction sector. (Young et al., 2003) According to the rubber compositions, it is not so easy to separate into simpler compounds. Therefore, the engineers are trying to study to learn about the physical parameters and reactions of rubber with mixing to the sandy soil. Which can used in numerous geotechnical projects, as lightweight filling materials, insulating of layer to decrease the permeability, bridge and dams slopes, and for dewatering purposes.

Ahmed (1993) stated that mixing sand with tire crumbs could increase the durability. The friction angle shows a maximum value with adding of 30 percent tire chip with dense sand. (Bosscher et al. 1997; Tweedie et al. 1998; Humphrey el al. 1998; lee et al. 1999; Dickson et al. 2001; Zornberg et al. 2004) these articles indicate tire chips combinations can bring lower the compressibility and higher durability and changes the performance of the soil while using the tire crumbs.

While constructing bridges the soil with tire crumbs mixture can be great filling material, due to the high shear strength and less unit weight and little steeper in slopes. It causes to minimize the volume of used material and decrease the settlement and slip off.

Tatlisoz et al (1998), Edil and Boscher (1994), placed shard tire vertically in sand particles. The founding proves that placing tire shards vertically can bring high durability rather placing horizontality. The tire itself or chips because of it is light weight can be used in geotechnical engineering in numerous areas. Therefor many researches such as laboratory studying, physical modelling, numerical analysis, conducts that tire can be used as lightweight filling for embankment of bridges, dams, lightweight traffic, etc.

Ahmed (1993), Bernal (1996), Masad et al (1996) Lee et. al (1999), Tweedie et. al (1998), Bergado and Ynowai (2003), Humphrey plus tweedie (2002), Edil (2002), Akbulut with Cabalar (2103) And, Lee (1999) had worked on the idea of cementing rubber chips together by using different glues to shape a block form.

Ghani et al., (2002) worked with shard rubber beside rubber chips, to improve geo-mechanical properties of the embankments back fillings materials and inclusions. These materials due to lower densities can be a suitable for road construction projects. Lighter soil with high bearing capacity decrease the foundation design requirement. , which reduce the cost to certain amount, secondly it minimize the project construction duration and avoids from ground settlements. Lightweight filling material bring down the lateral earth pressure, so it can reduce overall structure supplies such as steel, cements, wall and foundations supplies.

The usages of various types of waste tire materials in which has used in geotechnical engineering for soil improvements objectives are described the Table 2.2.

**Table 2.2.** Usages of various type of waste tire materials in geotechnical engineering (Ghani, et al. 2002)

| Type | Mass | Strength | Usage |
|---|---|---|---|
| **Whole Tire** | - | - | Most of the time used in soil stabilization. As well as suitable for tunnel, slope, embankment, reinforcement. |
| **Tire chips** | 3.0-3.55 kN/m$^3$ , if compacted 7.99-8.99 kN/m$^3$ | Changes between 36.99-42.99 grade for soft soil and 84 for compressed. | Better dewatering option with absorbing water one cm/sec. |
| **Tire chips Mixed with Soil** | With density of 12.9 kN/m$^3$ for soil mixture 50% soil and 50% Tire shreds | Compaction needed. | Mixing of 37-39% tire chips provide best economically efficient and technical suitable benefits. |
| **Tire shared with mandatory additives** | Density changes from 4.9-9.9 kN/m3, the density of other additive also has to be consider. | The compaction force must to apply to up to necessity. | Best dewatering properties. Used in dams or places where dewatering is necessary. |

### 2.3.2. Legislators Reaction to Waste Tires

Eul-lex (1999), suggest that tires due to the large volume and low-density it is not suitable to deposit in landfills. Due to the rising number of waste tire each year in many countries, the legislators and environmental activists, has been took an action to suggest the disposal or re-usage of these materials in industry. As an example can be European communities, where tire used in geo-technical engineering activities such as soil stabilization.

USEPA United State Environmental Protection Agency (2006), report mentions within the European community, the politics and regulations are inspiring the Best Administration Practices (BAP) for recycling of waste tires produces. This procedure had also been applied in many other countries e.g every state in U.S.A, Germany, Norway, etc. The of waste tire problems also debated among the United Nations Environmental Program (UNEP), which finally lead to (BAP) project. In addition, they listed a technical usage of old tire as construction materials.

By way of, example U.S.A and Europe through making regulation for inspiring the usage of waste tire as alternative for industry sector it will open new ways to other countries such as turkey to invest in such projects. Like (waste tire as construction materials)

### 2.3.3. Background of Waste Tire studies

The rising stockpiles of tire waste resulted in an interest of developing new ways to reuse or recycle tire wastes. Shredding tire waste and using it as a construction material is an option to reuse the tire waste, so many studies had been done to evaluate the characteristics of the tire waste alone and as an additive to basic material as sand, clay, concrete or asphalt. In the some of these studies, the tire waste shear strength properties were determined, just like in this thesis, which will be evaluated in further sections.

Humphrey, et al. (1993) has conducted large- scale direct shear test. Tire chips with length smaller than 72 mm were used. This study reported that friction angles and cohesion were ranging between 19$^{o}$ and 25$^{o}$ and 7.7 -8.6 kPa respectively, in the study it is stated that tire chips are useful for constructing lightweight embankments over soft soil.

Tatlisoz et al. (1998), conducted a large-scale direct shear tests with tire chips with sand and sandy silt mixtures. In the study, it is reported that sand-tire chips mixture had an increasing behavior of shear strength as the volume of the tire chips increased up to optimum percentage, whereas the sandy silt-tire chip mixture did not have a change in the angle of internal friction, but just an increase in the cohesion.

Edincliler et al. (2004), conducted the large-scale direct shear tests with tire buffing’s, in sand tire buffing’s mixtures. In the study, it is reported that 10 percent by weight tire buffing addition to sand alters the deformation behavior of the mixture by stiffening the material at low strains and softening the mixture at large strains. Based on beneficial technical and acceptable ecological influence a selection of best management practice has been performed. In all application, the environmental opinion must be considered. Based on the tire waste material properties the following applications have been identified as potential interest. Lightweight fill, Thermal insulation, Drainage layer, Backfill Material, Elastic Layer.

### 2.4. Reactions between Lime Powder and Soil Particles

After lime powder mixed into the soil mass, hydration of lime powder is a reason for fast drying soil. Dry lime powder will have an additional prominent result in the soil drying more than lime with water (hydrated lime). Accordingly, in construction areas, lime powder has verified as an active beneficial materials for aerating the wet sites.

While mixing lime powder to a plastic soil, the plasticity index decreases also the texture varying in advance. The biochemical variations happening in soil mass are most of the time described with accepted mechanisms, certain changes such as, flocculation, and combination or dispersing between the particles is happening. The flocculation, happens directly afterward lime is mixed to the soil mass while the accumulation is totally depend on the temperature and the extensive period reaction between the clay particles and lime powder.

Chemical changes are considered as an important factor for alterations, which is happing in the plasticity properties of the soil. All the plasticity and accumulations changes are the reason for various type of cations replacing in the isotropic series. All together, the cations in the sequence remains along with the isotropic orders, Lithium$^{+}$< Sodium $^{+}$< Hydrogen$^{+}$< Potassium $^{+}$< $NH_4^{+}$< $Mg^{2+}$< $Ca^{2+}$< $Al^{3+}$, i.e., every cation is replacing the cation next to it by replacing power. The replacement of sodium or potassium ions and calcium meaningfully decrease the plasticity properties of a clay soils.

Adding lime powder into the soil increase the pH values in the soil, which accelerate the cations exchange abilities. In addition, even soil with high calcium content reduces the plasticity properties while adding lime powder to the soil. Decrease in plasticity properties generally accompanied through reduction in shrinkage.

Adding lime powder to the soil caused a frequent change in the soil texture. Because of atom accumulation, the clay type of soils converted to silt and sand in performance. Therefore, the size of clay soil particles smaller than (2µm) reduced while the lime amount rises up. Verhasselt (1990) studied many possible links between the clay particles and lime power in his research. His results shown that accumulation is most likely the result of

Hydrogen ($H^+$) and hydroxyl ($OH^-$) connection, which make the calcium hydroxyl (Ca ($OH_2$)) on clay soil particles surface. Constant with high strength particles shaped via clay

soil particles and lime powder reactions, thus clay soil weak particles joint together and causes a mass of soil with high strength. (Ipek, 1998) in his research had proposed that stabilization happens while the suitable quantity of the lime powder is mixed with the subjected soil. Ingles and Metcalf (1972) suggested lime-mixing ratio for different type of soil. Table 2.3 mention about the different percentages of lime mixing with different type of soil soils.

**Table 2.3.** Recommended Lime Percentage for various type of soil by (Ingles, 1972)

| Soil Category | Modification Content | Stabilization Content |
|---|---|---|
| Fine crushed gravels | 2-4% | Not Recommended |
| Well graded clay gravels | 1 – 3 % | ~ 3 % |
| Sands | NR | NR |
| Sandy-clay | NR | ~ 5 % |
| Silty-clay | 1 – 3 % | 2 – 4 % |
| Heavy clay | 1 – 3 % | 3 – 8 % |
| Very heavy clay | 1 – 3 % | 3 – 8 % |
| Organic soils | NR | NR |

Soil improvement is different from soil modification; soil stabilization is a long –time strength development process by pozzolonic reactions. The pozzolonic engagements is the creation of calcium silicate hydrates ($H_2CaO_4Si$) plus calcium aluminate hydrates ($CaAl_2H_2$). The (Ca) from lime powder react with the (Al) and ($SiO_4$) of soil and make a solubilized form in the clay particles surface. The engagements is suddenly starts and it continues for long period even several years and the pH of soil leftovers higher. As result of reactions soils particles can produce high strength.

The most important factor for strength is the right mixing ratio of lime and soil. The strength factor varies for different type of soil, in many soils resilient modulus can have an increasing value of 10 times but shear strength factor can be change up to 20 times or more. In most cases the environmental humidity, external loading and, period of loading, effect the strength factors.

Wibawa, 2003 in his work explained the how clay soil particles disperse while mixing with lime powder to the soil mass Figure 2.2.

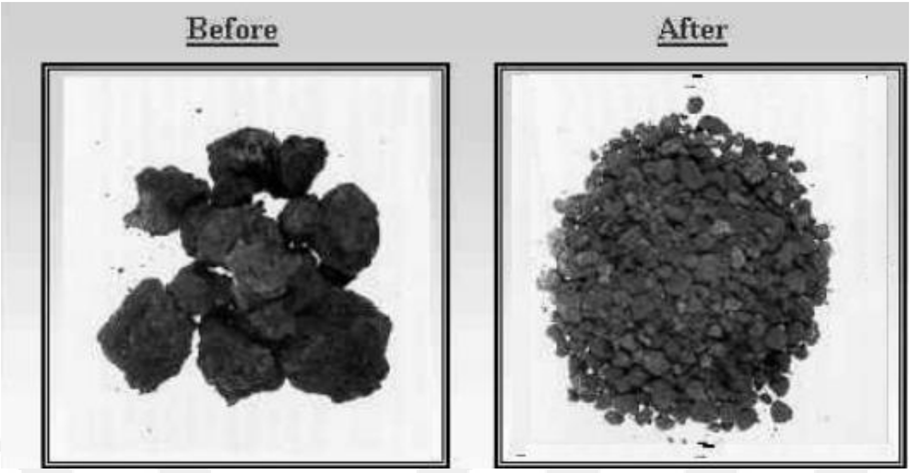



**Figure 2.2.** The variation after- mixing the lime powder in to the soil has presented, (Wibawa, 2003)

The final goal of this research is to find the shear strength, plasticity properties and maximum dry density of sandy-clay mixture with tire crumbs and lime powder. This research based on comparative observation between tire crumbs and lime powder. The soil mixed with tire crumbs had tested after the compaction, while the samples with lime powder had kept for 28 days in desiccator and later the unconfined compressive strength (UCS) test had performed for the samples.

# 3. MATERIALS AND METHODS

## 3.1. Materials

The materials, which used in this research, is river sand, kaolin clay, tire crumbs, and lime powder.  All the tests have done according to ASTM standards.

### 3.1.1. Sand

The sandy soil used in the study was the locally available soil from a river near Elazığ city at Turkey, Table 3.2. Show the index property of the sandy soil sample. The grain size distribution of the sand had been obtained through sieve analysis tests that were carried out according to ASTM D422-63, shown in the Table 3.1. The specific gravity of subjected sand have obtained according to ASTM D854-14. Maximum dry density, minimum dry density, maximum void ratio ($e_{max}$) and minimum void ratio ($e_{min}$) are measured by experiment based on ASTM D4254-16. The sand classified as poorly graded sand (SP) according to Unified Soil Classification System.

**Table 3.1.** Particle sizes of sand used in experiments

| Grain size (mm) | Weight percentage content (%) |
|---|---|
| 1.00-0.42 | 57.2 |
| 0.42-0.177 | 34.6 |
| 0.177-0.149 | 4.8 |
| 0.149-0.074 | 3.6 |
| < 0.074 | 0.1 |

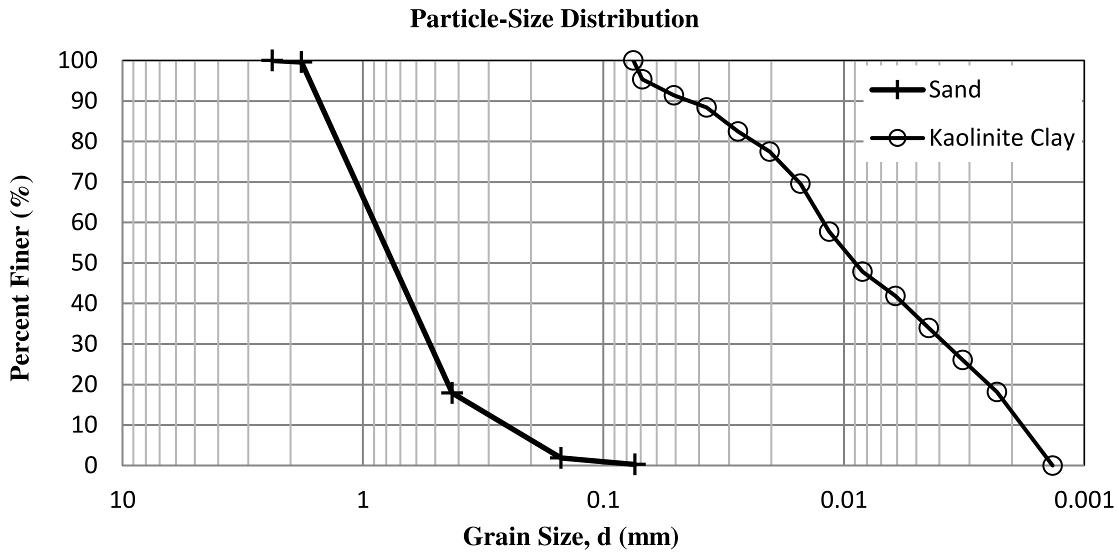



**Figure 3.1.** Particle size distribution of soils used in experiments

**Table 3.2.** Index properties of sand used in experiments

| Index Properties | Values |
|---|---|
| Gs | 2.77 |
| $\gamma_{natural}$ (gr/cm$^3$) | 1.54 |
| e | 0.905 |
| $e_{max}$ | 1.064 |
| $e_{min}$ | 0.667 |
| $D_{10}$ (mm) | 0.19 |
| $D_{30}$ (mm) | 0.30 |
| $D_{50}$ (mm) | 0.47 |
| $D_{60}$ (mm) | 0.54 |
| $\gamma_{d(max)}$ gr/cm$^3$ | 1.66 |
| $\gamma_{d(min)}$ gr/cm$^3$ | 1.34 |
| $C_u$ | 2.82 |
| $C_c$ or $C_g$ | 0.87 |
| Soil Class | SP |

### 3.1.2. Kaolin Clay

The soil used in this study is a type of kaolin clay. It was bought from a company name “Utelka”. The soil have classified according to Unified Soil Classification System (USCS) and the soil is high plasticity clay (heavy clay). The liquid limit, plasticity limit, and plasticity index test carried out according to ASTM D4318 - 17e1, which have shown Table 3.3. In addition, the grain size distribution is determined by using hydrometer test according to ASTM E100-17, (Figure 3.1) shows the particles size curve of the used soil. The specific gravity estimated according to ASTM D854-14, the specific gravity value ($G_S$) is equals to 2.6 gr/cm$^3$ pointed out at (Table 3.3). Finally the maximum dry density $\gamma_{d(max)}$, optimum moisture content ($w_c$) and void ratio (e) is determined based on standard proctor test - ASTM D698 – 07, shown in (Table 3.3).

**Table 3.3.** Index parameters of clay soil used in experiments

| Index Properties | Values |
|---|---|
| Gs | 2.60 |
| $\gamma_{natural}$ (gr/cm$^3$) | 1.54 |
| $\gamma_{d(max)}$ gr/cm$^3$ | 1.455 |
| liquid limit (LL) | 51.60% |
| Plastic limit (PL) | 31.10 % |
| Plasticity Index (PI) | 19.40 |
| ($w_{opt}$) | 26 % |
| Soil Class | CH |
| | |

### 3.1.3. Tire Crumbs

The tire crumbs used in the experiments of this study was obtained from a local waste gathering company located in Elazig, Turkey. Before to start the experiment crushed tire-crumbs washed to be clear of any dust. Figure 3.2 shows the tire crumbs material used in experiments. The gradation curve of the tire crumbs had been obtained through sieve analysis test that was carried out according to ASTM D422-63; Figure 3.3. shows the gradation of tire crumbs. The specific gravity of shared rubber carried out in accordance to ASTM D854-14, was 1.1 gr/cm$^3$, the maximum dry density for tire crumbs were 0.634 gr/cm$^3$ based on ASTM D698- 07. The tire crumbs were angular shape with an optimum moisture content of 4.6%. Their dimensions were between 0.075 to 2.5 mm (Table 3.4).

**Table 3.4**. Property parameters of tire crumbs used in experiments

| Index Properties | Values |
|---|---|
| $G_s$ | 1.10 |
| $\gamma_{natural}$ gr/cm$^3$ | 0.66 |
| e | 0.735 |
| $\gamma_{dmax}$ gr/cm$^3$ | 0.63 |
| ($w_{opt}$) | 4.64 % |

**Figure 3.2.** Tire crumbs used in experiments.

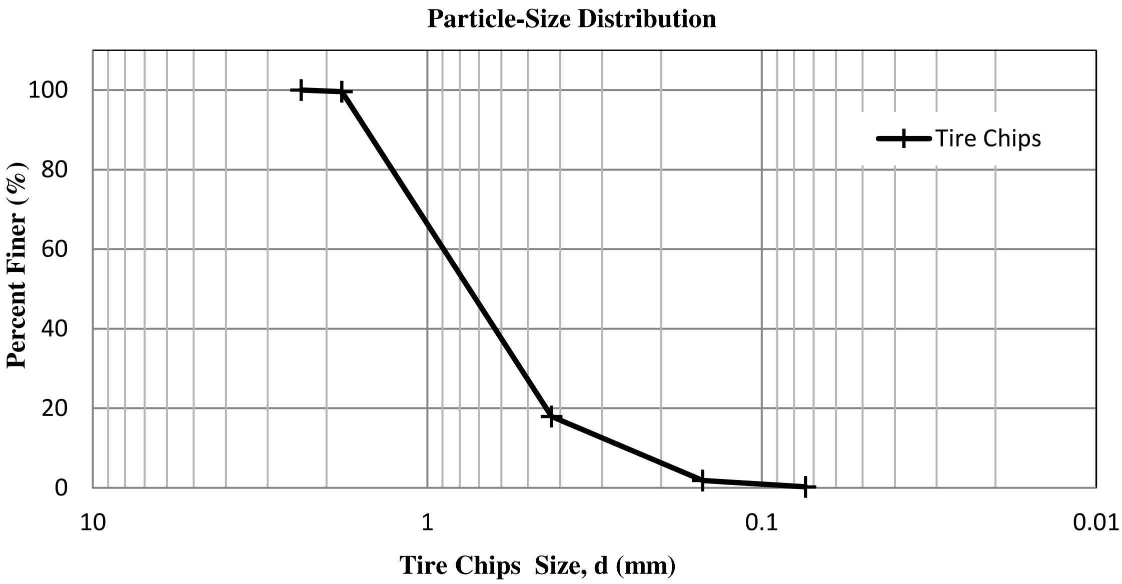



**Figure 3.3.** Grain size distribution curve, of tire crumbs.

### 3.1.4. Lime Powder

The geotechnical parameter (specific gravity) of lime powder had obtained according to ASTM D854-14 standards. The results showed value of 2.68. The entire lime powder had passed through sieve #200 as stated by ASTM D422-63, all the particle has the diameter smaller than 0.0075 mm (Figure 3.4). In the same time the chemicals properties of the same type of lime powder was analyzed by Başer (2009). Başer in his Master thesis had performed a serious of x-ray diffraction analysis to determine chemical composition of lime powder. Table 3.5 shows the percentage of the chemical elements of the lime powder.

**Table 3.5.** Percentage of the chemical elements of subject lime powder according to Başer in (2009).

| Elements | Content % |
|---|---|
| **Carbon (C)** | 37.9 |
| **Oxygen (O)** | 47.0 |
| **Calcium (Ca)** | 8.5 |
| **Magnesium (Mg)** | 6.6 |

**Figure 3.4.** Lime powder, which have used in these experimental studies.

## 3.2. Laboratory Studies

### 3.2.1. Experimental Setup

All the experiments have performed in two main classes

1- Class one of the laboratory research focuses on the mixture of river sand with kaolin clay and tire crumbs.

a) The amount of 60% clay 40% river sand and 0% tire crumbs mixed by total weight, for the first sample. Afterwards the amount of 60% clay mixed with 40% river sand and 1% tire crumbs for the second sample. Then the tire crumbs ratio changes to 2% and 4 % in third and fourth samples.

b) In second part of class one the ratio of 70% clay mixed with 30% river sand and 0% tire crumbs for the first sample and the 70% clay with 30% river sand and 1% tire crumbs for second sample. After the tire crumbs ratio changes to 2% and 4 % in third and fourth samples.

c) In third part of class one, the ratio of 80% clay mixed with 20% river sand and 0% tire crumbs for the first sample and the 80% clay with 20% river sand and 1% tire crumbs for second sample. After the tire crumbs ratio changes to 2% and 4 % in third and fourth samples.

2- Class two of the laboratory work, is depended on a mixture of river sand with kaolin clay and lime powder after finding the ($\gamma_{dmax}$).

a. The amount of 100 % kaolin clay mixed with 5 % of lime by total weight, for the first test.

b. The amount of 60% kaolin clay mixed 40%river sand and 5 % of lime by total, for the second test.

c. The amount of 70% kaolin clay mixed 30%river sand and 5 % of lime by total, for the third test.

d. The amount of 80% kaolin clay mixed with 20% sand with 5 % of lime by total, for the fourth test.

Figure 3.5. and Figure 3.6. Shows the tire crumbs specimens mixture, which is used experiments.

### 3.2.2. Sample Preparing Procedure

Two different soil samples (river sand and kaolin clay) with lime powder and tire crumbs have used for this research. All the materials have different gradation curves, which varied from 2.38 mm up to 0.0014 mm. The Proctor dry density had taken for all the samples.

In laboratory research, primarily all of the properties tests had done for all four type of materials, which used in the research. The liquid limit, plastic limit, specify gravity, hydrometer, sieve analysis and relative density of all materials had tested according to ASTM standards.

Afterward the samples were mixed in 12 different ratios. Figure 3.5 show the mixing ratio, the Standard Proctor compaction test had performed (ASTM D698– 07) for each sample to find out the maximum dry density.

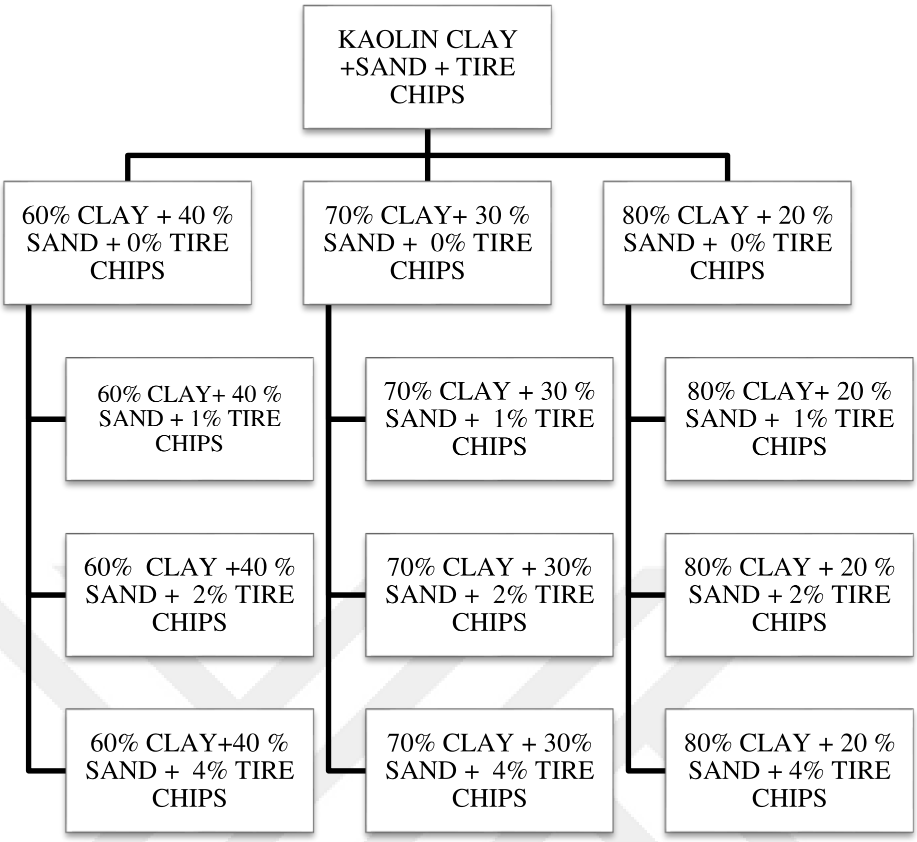



**Figure 3.5.** Mixing ratio of kaolin clay + Sand + tire crumbs, which used in lab operations.

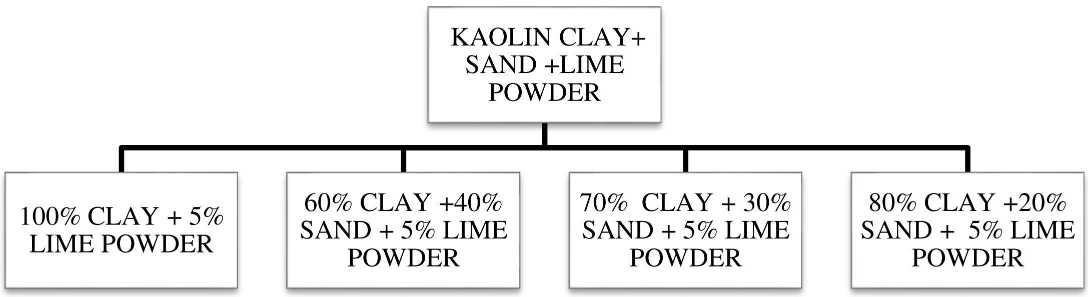



**Figure 3.6.** Shows the lime powder specimens mixture with Sand and kaolin clay, which used in lab operations.

After finding ($\gamma_{dmax}$), that samples were prepared at Proctor density to conducted the unconfined compressive test (UCS) test. A handmade sampler had use to take samples for unconfined compressive (UCS) test. For the samples with tire crumbs, the (UCS) test had performed after the sample had taken according to (ASTM D2166-00). On the other hand, for the samples with lime powder the UCS test had carried out after 28 days. Figure 3.7. Shows over all sampling method.

**Figure 3.7.** Shows over all the specimens preparing procedure.

### 3.3. Tests

#### 3.3. 1. Unconfined Compressive Strength Testing Process (UCS)

In these experimental studies, the ELE Tritest-50 test machine, which exist in the Firat University Civil Engineering department geotechnical laboratory, is used to determine the unconfined compressive strength (UCS) of the specimen. According to (ASTM D2166-00).

The ultimate capacity of the loading ring of unconfined compressive testing machine was 2kN for first class of soil samples and 28kN for second class of soil samples. Sharing load rate of the unconfined compressive test machine had adjusted 0.5mm/min during the test. The free constant load applied to the samples, the ring readings and deformations has been recorded. Thus, the maximum shear strength had been founded; one of the unconfined compressive test specimen sample preparation and test performing are shown in Figure 3.8 and Figure 3.9. According to ASTM D2166-00 the ($q_u$) or unconfined compressive strength is defined as compressive stress at which the free form of load fail down in cylindrical sample of soil. Furthermore, in such method of testing the compression strength is considering as maximum load reached per unit area.

The test calculation is based on the following formula.

$$qu = \sigma = P\,max \,/\, A_f \quad \textbf{(3.1)}$$

$P_{max}$ : The maximum load at break point

$A_f$ : The cross-sectional area of the soil sample during filler

$q_u$ : Ultimate compressive strength

Calculating the cross-sectional area of the soil sample during fracture

$$V = Constant = A_0 H_0 = A_f H_f \quad \textbf{(3.2)}$$

$$H_f = H_0 - \Delta H \qquad A_f = \frac{A_0}{1 - \frac{\Delta H}{H_0}} \quad \textbf{(3.3)}$$

$$c = {q_u}/{2} \quad \textbf{(3.4)}$$

A manual sample taker with diameter of 3.8 cm and length of 10 cm is used to take sample from the Proctor compaction sample. (Figure 3.8). After that an extractor is used to take out the sample for (UCS) test.

**Figure 3.8.** Unconfined compressive strength test sample preparation.


**Figure 3.9.** Unconfined compressive strength test

### 3.3. 2. Standard Proctor Test

For the purpose of observing the behavior of sand, kaolin clay tire crumbs (TC), and their mixtures, standard compaction Proctor test was used to determine the maximum dry density and optimum moisture content of sand alone and sand-kaolin clay (KC) mixture ratios. In the Standard Proctor test had performed according to ASTM D698 – 07, the sand was compacted in a mold that has a volume of 943 cm$^3$. The diameter of the mold was 101.6 mm. During the laboratory test, the mold was attached to a baseplate at the bottom and to an extension at the top (Figure 3.11). The sand was mixed with kaolin clay in various amounts of water and then compacted in three equal layers by a hammer (Figure 3.12) that delivers 25 blows to each layer. The hammer has a mass of 2.5 kg and has a drop of 305 mm. Figure 3.10, and Figure 3.1), is a photograph of the laboratory equipment required for conducting a standard Proctor test.

For each test, the moist unit weight of compaction, $\gamma_n$, calculated as

$$\gamma_n = \frac{W}{V_m} \tag{3.5}$$

Where;

W = weight of the compacted soil in the mold

$V_m$ = volume of the mold (944 cm$^3$)

For each test, the moisture content of the compacted sand is determined in the laboratory.

With the known moisture content, the dry unit weight can be calculated as

$$\gamma_d = \frac{\gamma_n}{1+w_c} \quad \textbf{(3.6)}$$

Where;

$\gamma_n$ = natural or moist unit weight of soil

$w_c$ = percentage of moisture content.

The values of $\gamma_d$ determined from Eq. (3.5) can be plotted against the corresponding moisture contents to obtain the maximum dry unit weight and the optimum moisture content for the sand. These procedures are done for clay, each clay- sand, tire crumbs mixture ratios, the maximum dry density, and optimum moisture content is determined for each ratios.

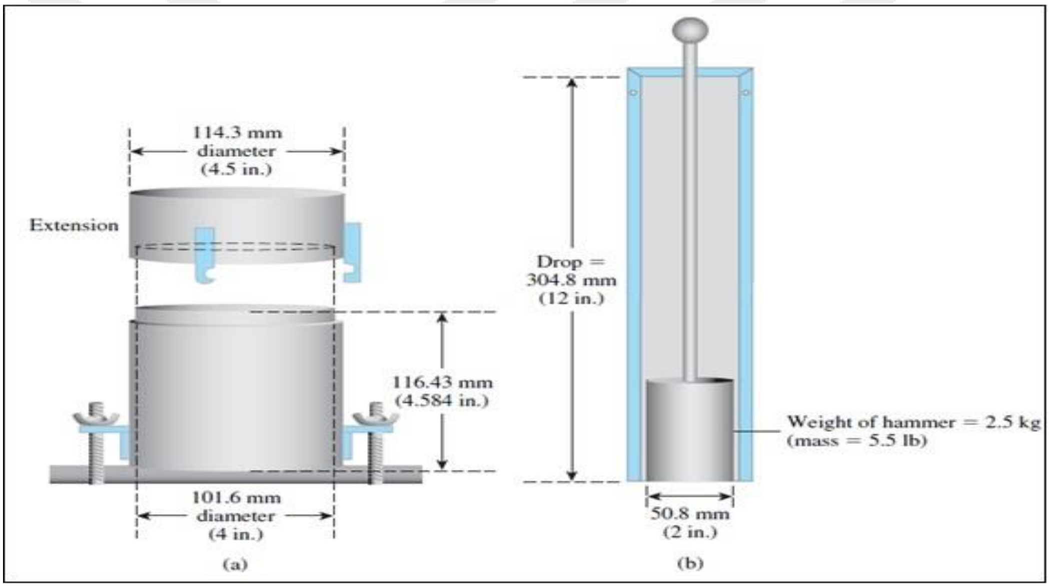



**Figure 3.10.** Standard proctor test equipment: (a) mold; (b) hammer

**Figure 3.11**. a). Standard proctor test mold with base plate b).electrical standard Proctor test machine.

# 4. RESULTS AND DISCUSSIONS

## 4.1 Standard proctor test

The experimental results obtained from standard proctor test for clay soil and compacted sand with kaolin content were plotted in various forms. The results showed that the maximum dry density ($\gamma_{dmax}$) increases with increasing of sand content because the sand particles enter to the pores of the mixture. The optimum moisture content ($w_{opt}$) decreases with increase sand content thus, due to increase surface area and non-cohesive material. The results obtained from standard proctor test for sand and kaolin mixture shown in the (Figure 4.1).

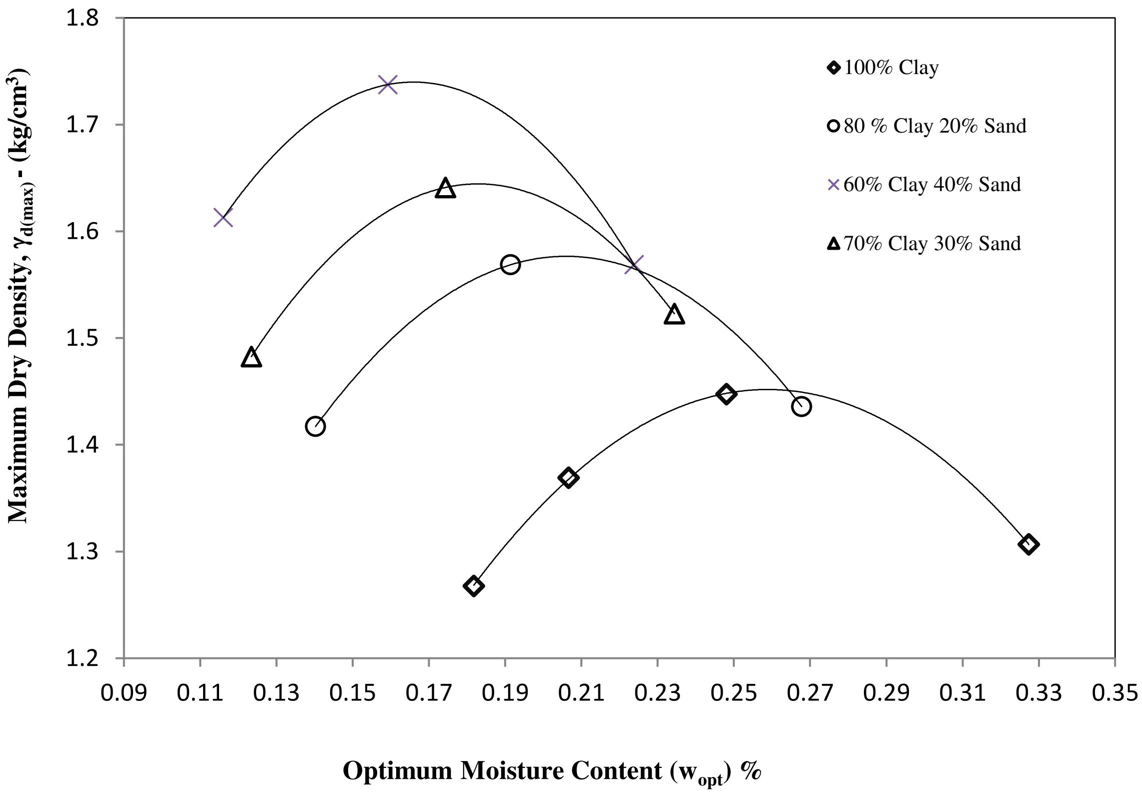



**Figure 4.1.** Maximum Dry Density and Water Content Variation, in different clay-sand Mixtures

The linear relationships between maximum dry density of kaolin clay and additive sand show in the (Figure 4.2), as well as the linear relations between moisture content of kaolin clay and additive sand show in the graph (Figure 4.3).

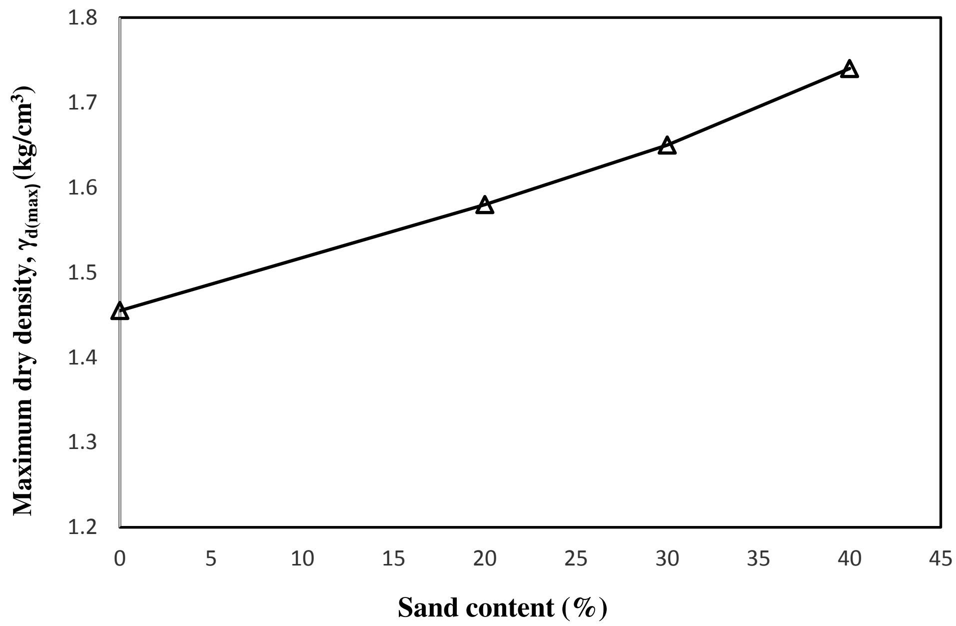



**Figure 4.2.** Maximum dry density and additive sand ratio, linear relationships

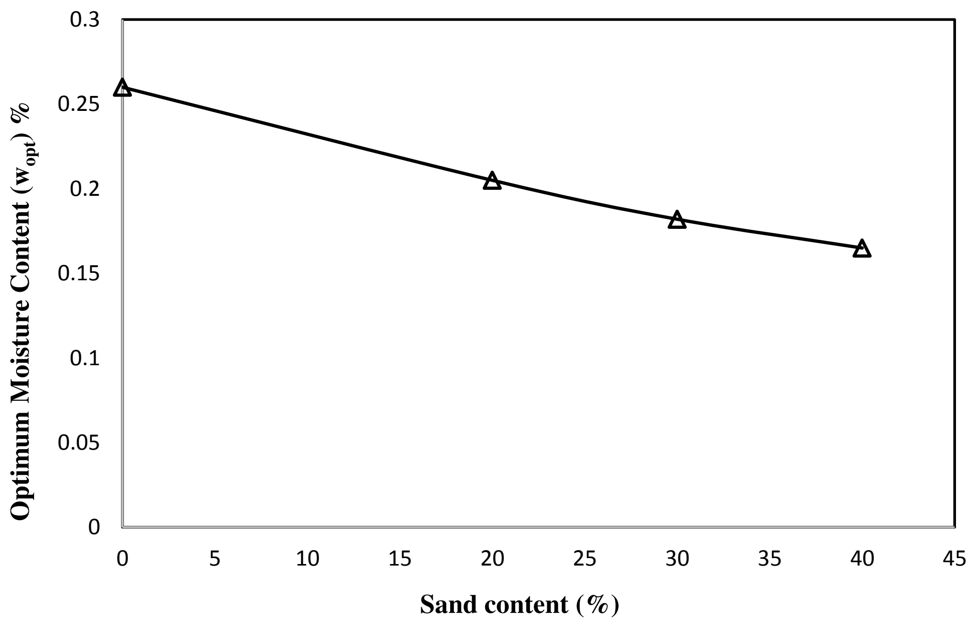



**Figure 4.3**. Moisture content and additive sand ratio, linear relationships

Table 4.1. Shows clay-sand mixture Maximum dry density, $\gamma d(max)$ and Optimum Moisture Content ($w_{opt}$) %, values obtained from Proctor test by addition of tire crumbs.

**Table 4.1.** Clay-sand mixture Maximum dry density, and Optimum Moisture Content values**.**

| **Maximum dry density, $\gamma_{d(max)}$ and Optimum Moisture Content ($w_{opt}$) % variation in tire crumbs-sand samples** | | | |
|---|---|---|---|
| **Clay-Sand Sample** | **Tire Crumbs Content %** | **Maximum dry density, $\gamma_{d(max)}$ ($kg/cm^3$)** | **Optimum Moisture Content ($w_{opt}$) %** |
| 60% Kaolin Clay + 40 % Sand | 0% Tire Crumbs | 1.740 | 16.50 |
| | 1% Tire Crumbs | 1.730 | 16.56 |
| | 2% Tire Crumbs | 1.715 | 16.72 |
| | 4% Tire Crumbs | 1.698 | 17.10 |
| 70% Kaolin Clay + 30 % Sand | 0% Tire Crumbs | 1.660 | 18.20 |
| | 1% Tire Crumbs | 1.644 | 18.24 |
| | 2% Tire Crumbs | 1.628 | 18.45 |
| | 4% Tire Crumbs | 1.612 | 18.90 |
| 0% Kaolin Clay + 20 % Sand | 0% Tire Crumbs | 1.580 | 20.50 |
| | 1% Tire Crumbs | 1.568 | 20.55 |
| | 2% Tire Crumbs | 1.550 | 20.63 |
| | 4% Tire Crumbs | 1.525 | 21.20 |

Figure 4.4. Shows clay-sand mixture Maximum dry density, γd(max) graph variation while adding tire crumbs.

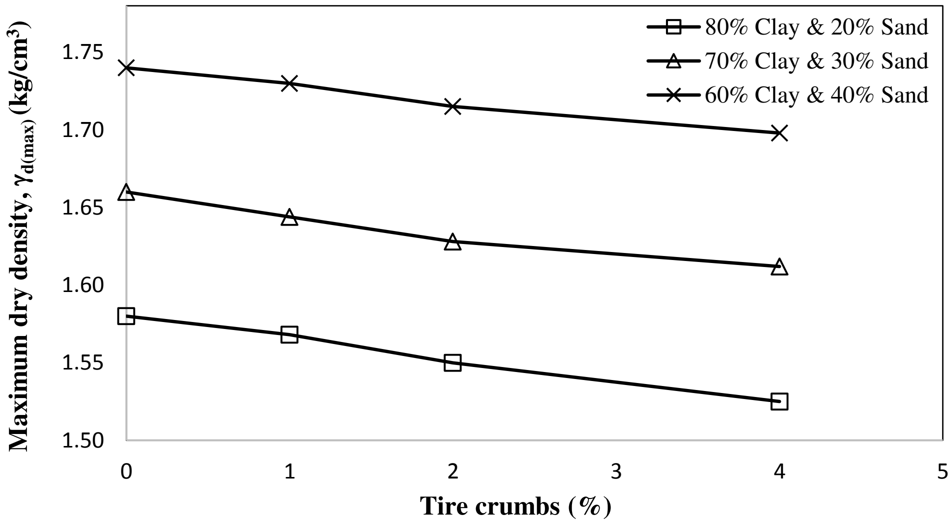



**Figure 4.4**. Maximum Dry Density changes with adding tire crumbs to the clay-sand mixture.

In the same, time the Figure 4.5. Shows clay-sand mixture the Optimum Moisture Content ($w_{opt}$) %, graph changes while adding tire crumbs to the mixture.

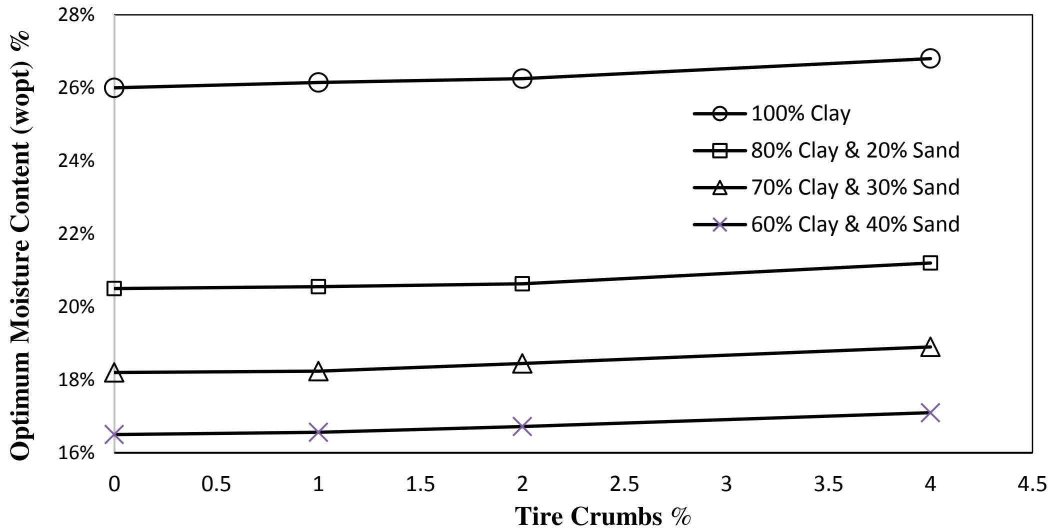



**Figure 4.5.** Optimum moisture content changes with adding tire crumbs to the clay-sand mixture**.**

## 4.2 Liquid Limit and Plastic Limit Test Results

Due to the low plasticity, feature in sand while adding sand to kaolin clay the liquid limit (LL) and plastic (PL) limit in kaolin clay decreases, but the plasticity index (PI) index have increased.

Table 4.2. Shows the Liquid limit, plastic limit and plasticity index values of kaolin clay-sand in different mixture.

**Table 4.2.** Liquid limit, Plastic limit and Plasticity index final values.

| Mixture Type | Liquid limit (LL) % | Plastic limit (PL) % | Plasticity index (PI) % |
|---|---|---|---|
| **100% Kaolin clay** | 50.6 | 31.11 | 19.48 |
| **80% Kaolin clay + 20 % Sand** | 49.4 | 26.5 | 22.9 |
| **70% Kaolin clay + 30 % Sand** | 48.9 | 24.7 | 24.2 |
| **60% Kaolin clay + 40 % Sand** | 48.4 | 22.7 | 25.7 |

The Figures (4.6, 4.7, and 4.8.) indicate the variation of liquid limit, plastic limit and plasticity index while adding sand to the kaolin clay.

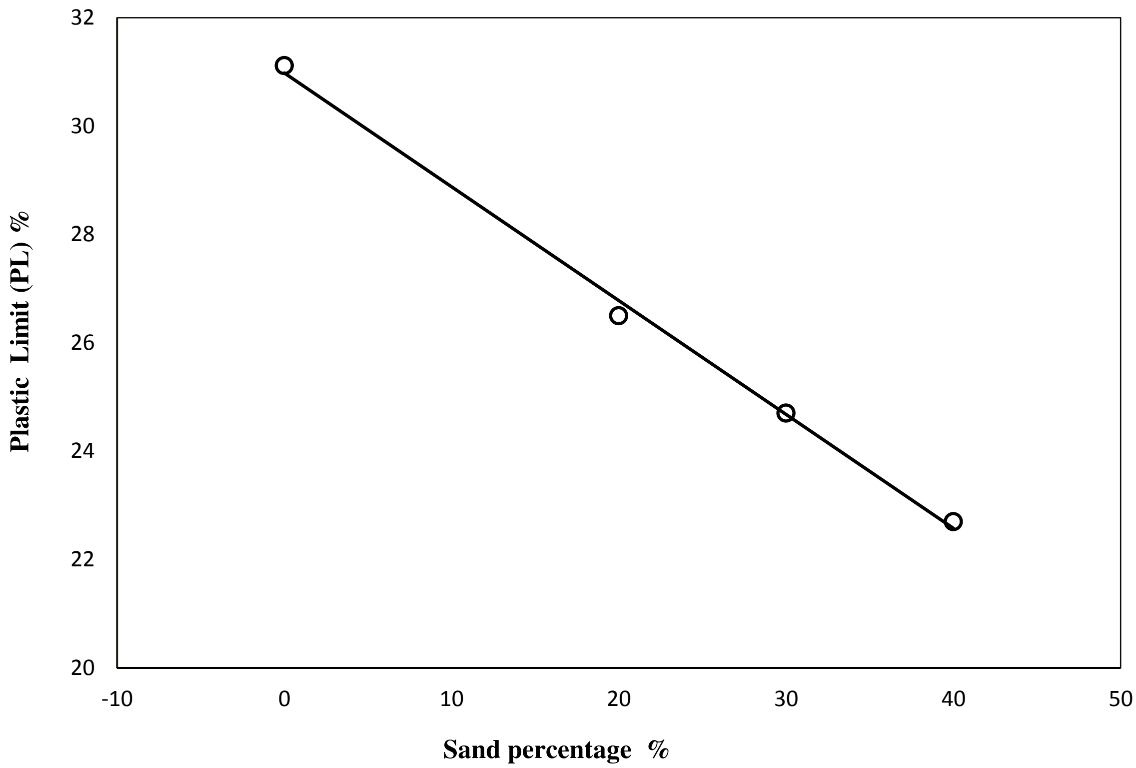



**Figure 4.6.** Variation of plastic limit by increasing sand percentage.

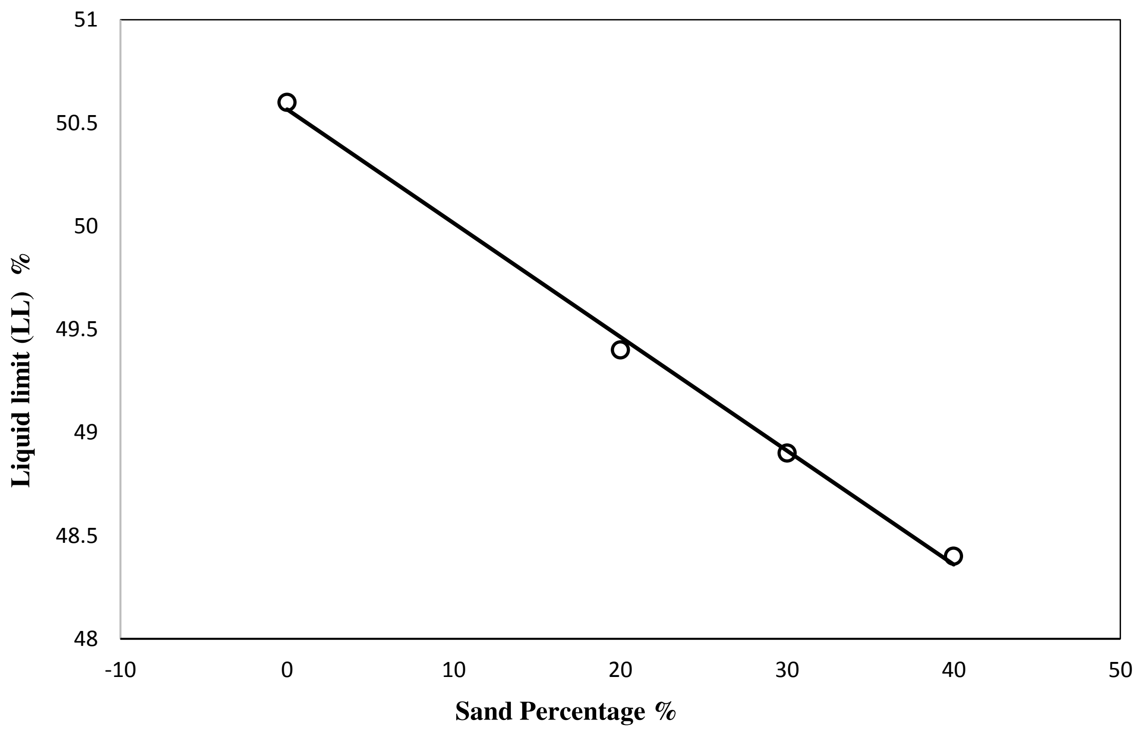



**Figure 4.7.** Variation of liquid limit by increasing s and percentage.

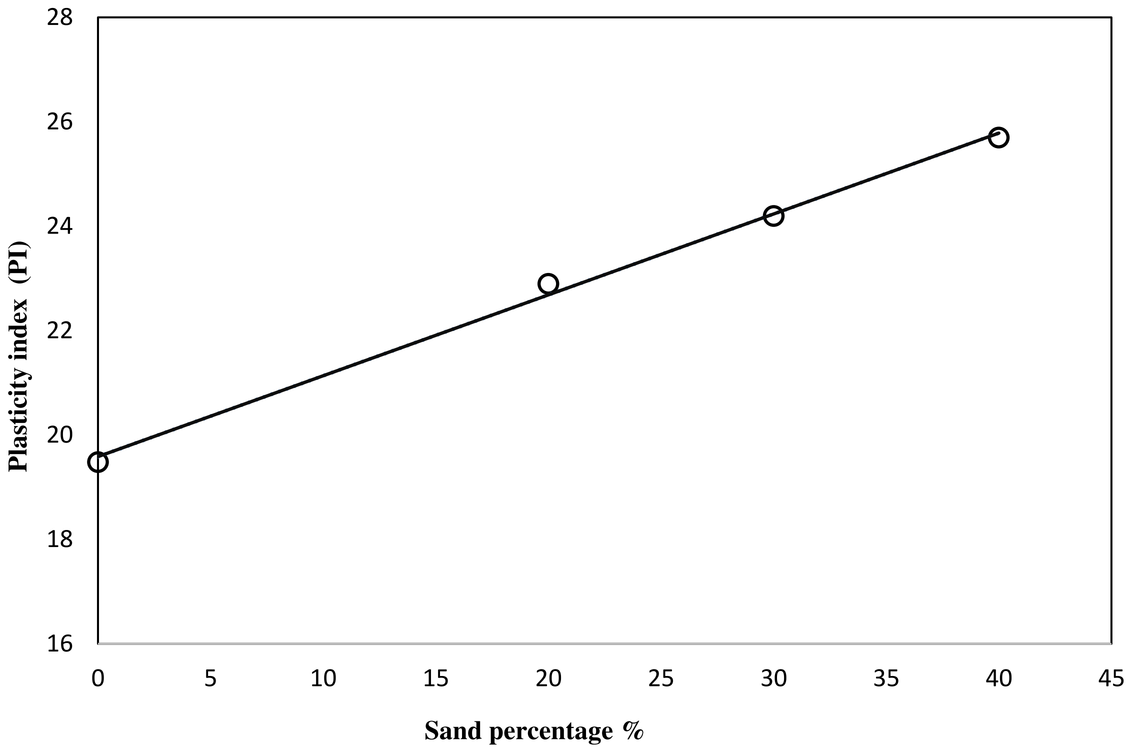



**Figure 4.8.** Plasticity index and sand percentage relationship.

## 4.3 Unconfined Compressive Strength test Results (UCS).

During the unconfined compressive strength test (UCS) for every ratio 3 sample were prepared and after the test result the average of 3 values had taken as real strength.

Throughout the studies, it had founded that optimum percentage of tire crumbs is 2%.It has founded that shear strength of clay-sand with tire crumbs mixture greater than the clay and sand mixture alone and, In this case; an increase in tire crumbs up to 2% increases the unconfined compressive strength (UCS). Figure 4.9. and Figure 4.10.

The shear strength values for 60% clay and 40% sand mixture increases from 2.76 kg/cm$^2$ to 4.30 kg/cm$^2$ with 2 % tire crumbs Table 4.3.

The shear strength value for pure 70% kaolin and 30% sand mixture is 2.58 kg/cm$^2$, and with 2% tire crumbs addition it changes to 3.88 kg/cm$^2$ Table 4.4.

The unconfined compressive strength (UCS) values for 80% clay and 20% sand mixture increases from 2.28 kg/cm$^2$ to 3.33 kg/cm$^2$ with 2 % tire crumbs Table 4.5.

The unconfined compressive strength (UCS) values drop down when sand percentage decreases to 30%, 20 % and 0 in the mixture.

**Table 4.3.** Unconfined compressive strength test values of 60% kaolin clay and 40 % sand mixture

| 60% Kaolin Clay + 40 % Sand | | | |
|---|---|---|---|
| **Sample No** | **Mixture Type** | **Maximum strength (kg/cm$^2$)** | **Average Maximum strength (kg/cm$^2$)** |
| a | 0%Tire Crumbs | 2.825 | 2.767 |
| b | 0% Tire Crumbs | 2.707 | |
| c | 0% Tire Crumbs | 2.767 | |
| a | 1% Tire Crumbs | 3.855 | 3.901 |
| b | 1% Tire Crumbs | 3.951 | |
| c | 1% Tire Crumbs | 3.896 | |
| a | 2% Tire Crumbs | 4.300 | 4.305 |
| b | 2% Tire Crumbs | 4.278 | |
| c | 2% Tire Crumbs | 4.337 | |
| a | 4% Tire Crumbs | 2.726 | 2.724 |
| b | 4% Tire Crumbs | 2.743 | |
| c | 4% Tire Crumbs | 2.702 | |

| 70% Kaolin Clay + 30 % Sand | | | |
|---|---|---|---|
| **Sample No** | **Mixture Type** | **Maximum strength (kg/cm$^2$)** | **Average Maximum strength (kg/cm$^2$)** |
| a | 0% Tire Crumbs | 2.581 | 2.583 |
| b | 0% Tire Crumbs | 2.625 | |
| c | 0% Tire Crumbs | 2.544 | |
| a | 1% Tire Crumbs | 3.535 | 3.503 |
| b | 1% Tire Crumbs | 3.507 | |
| c | 1% Tire Crumbs | 3.467 | |
| a | 2% Tire Crumbs | 3.910 | 3.884 |
| b | 2% Tire Crumbs | 3.889 | |
| c | 2% Tire Crumbs | 3.852 | |
| a | 4% Tire Crumbs | 2.462 | 2.464 |
| b | 4% Tire Crumbs | 2.456 | |
| c | 4% Tire Crumbs | 2.474 | |

**Table 4.4.** Unconfined compressive strength test values of 70% kaolin clay and 30 % sand mixture

**Table 4.5.** Unconfined compressive strength test values of 80% kaolin clay and 20 % sand mixture

| 80% Kaolin Clay + 20 % Sand | | | |
|---|---|---|---|
| **Sample No** | **Tire Crumbs Content %** | **Maximum strength (kg/cm$^2$)** | **Average Maximum strength (kg/cm$^2$)** |
| a | 0% Tire Crumbs | 2.314 | 2.284 |
| b | 0% Tire Crumbs | 2.257 | |
| c | 0% Tire Crumbs | 2.280 | |
| a | 1% Tire Crumbs | 3.085 | 3.083 |
| b | 1% Tire Crumbs | 2.985 | |
| c | 1% Tire Crumbs | 3.178 | |
| a | 2% Tire Crumbs | 3.356 | 3.333 |
| b | 2% Tire Crumbs | 3.311 | |
| c | 2% Tire Crumbs | 3.332 | |
| a | 4% Tire Crumbs | 2.306 | 2.275 |
| b | 4% Tire Crumbs | 2.270 | |
| c | 4% Tire Crumbs | 2.249 | |

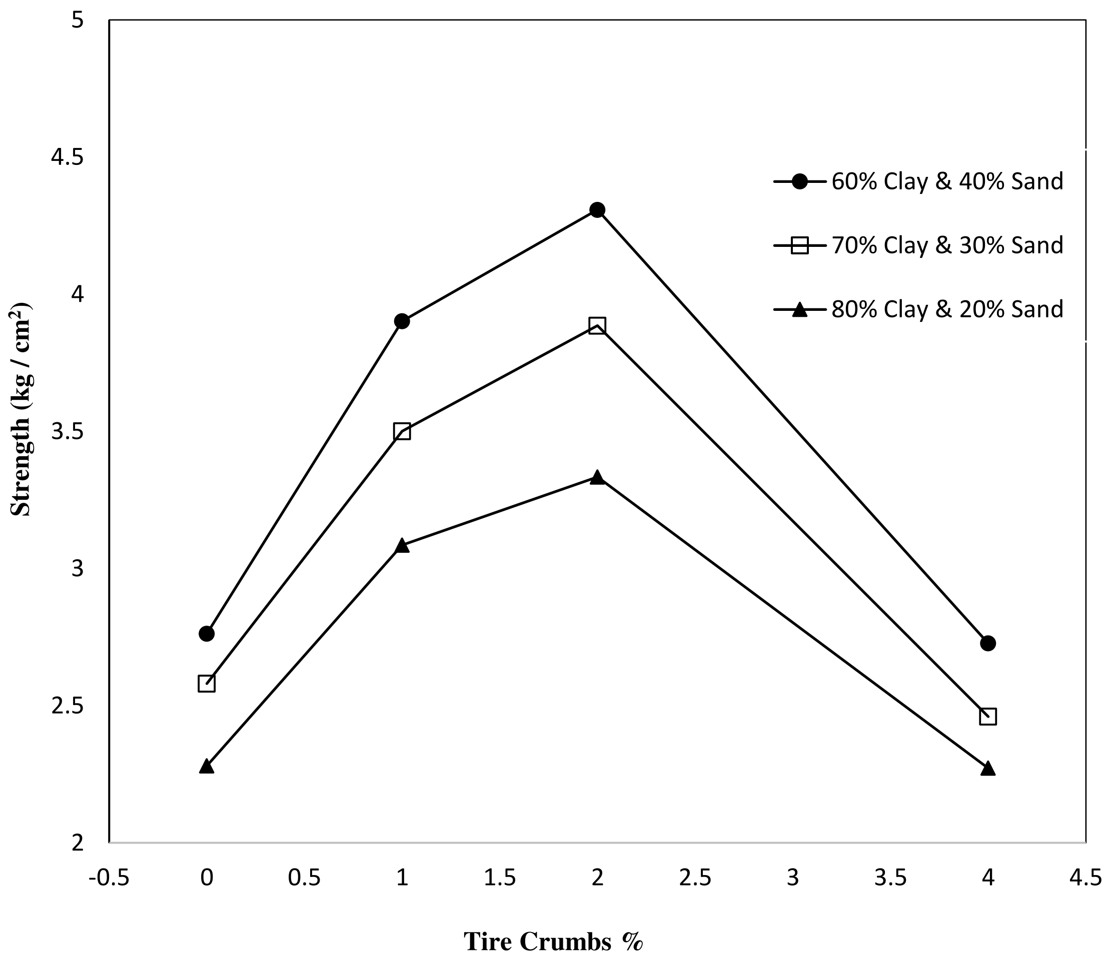



**Figure 4.9.** Optimum strength graph of tire-crumbs in kaolin clay-sand mixture

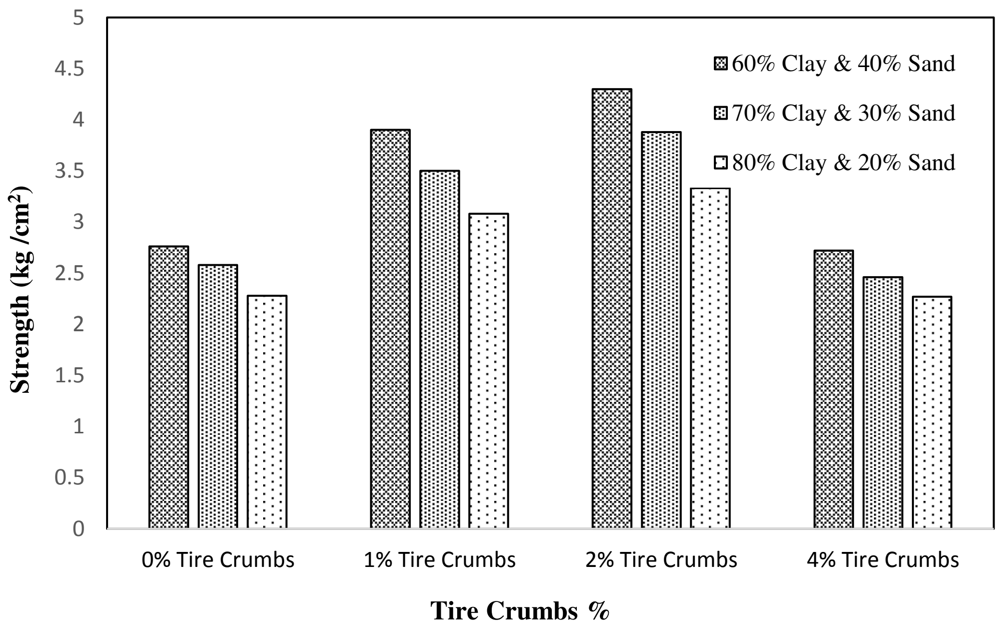



**Figure 4.10.** Kaolin clay-sand mixture comparative graph, with different ratio of tire-crumbs

Through the studies, it had founded that the adding sand to kaolin clay increase the shear strength, so the unconfined compressive strength values increase when sand percentage increases from to 0% to 20%, 30 % and 40% in the clay-sand mixture Figure 4.11.

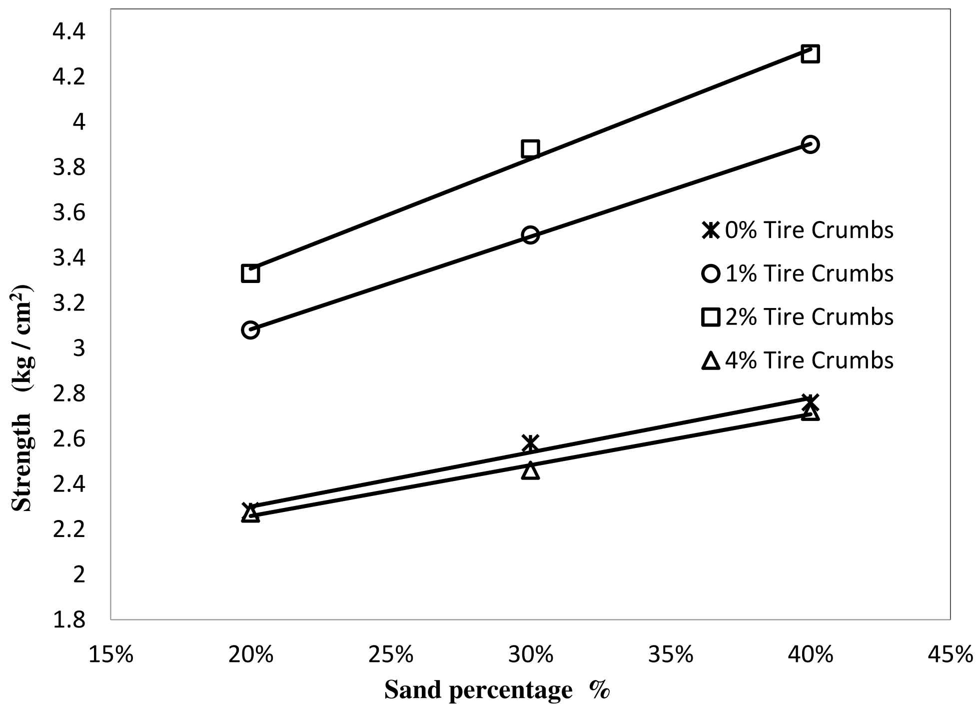



**Figure 4.11.** Shear strength and additive sand with tire crumbs, linear relationship

### 4.3.1 Unconfined Compressive Test Results (UCS) For Lime Powder Samples

The unconfined compressive strength ability for kaolin clay-sand mixture with lime powder has changed as follow.

The share strength values for 100 % kaolin clay, without lime had found 1 kg/cm$^2$, while with adding 5% lime powder after 28 days the strength parameters changes to 26.61 kg/cm$^2$. Similarly, the share strength values for 60% clay and 40 % sand mixture with 5 % lime after 28 days increased to 65.61 kg/cm$^2$ (Table 4.6).

As well, the share strength values in 70% clay and 30 % sand mixture with 5 % lime after 28 days increased to 49.31 kg/cm$^2$. Finally, the (UCS) values for 80% clay and 20 % sand mixture with 5 % lime after 28 days is find out 35.27 kg/cm$^2$ (Table 4.6).

**Table 4.6.** Unconfined compressive strength test values of kaolin clay-sand mixture after 28 days.

| **Kaolin Clay + Sand Mixture with lime Powder** | | | | |
|---|---|---|---|---|
| **Sample No** | **Mixture Type** | **Lime Powder Content (%)** | **Maximum strength (kg/cm$^2$)** | **Average Maximum strength (kg/cm$^2$)** |
| a | 100 % Kaolin Clay | 5 | 26.631 | 26.625 |
| b | 100 % Kaolin Clay | 5 | 26.624 | |
| c | 100 % Kaolin Clay | 5 | 26.619 | |
| a | 60% Kaolin Clay + 40% Sand | 5 | 64.100 | 65.612 |
| b | 60% Kaolin Clay + 40% Sand | 5 | 67.125 | |
| c | 60% Kaolin Clay + 40% Sand | 5 | 65.610 | |
| a | 70% Kaolin Clay + 30% Sand | 5 | 42.594 | 49.313 |
| b | 70% Kaolin Clay + 30% Sand | 5 | 49.328 | |
| c | 70% Kaolin Clay + 30% Sand | 5 | 56.018 | |
| a | 80% Kaolin Clay + 20% Sand | 5 | 35.777 | 35.277 |
| b | 80% Kaolin Clay + 20% Sand | 5 | 35.901 | |
| c | 80% Kaolin Clay + 20% Sand | 5 | 34.153 | |

Figure 4.12. Shows a compressive shear strength chart between different ratios of kaolin clay-sand mixture with 5% lime powder after 28 days.

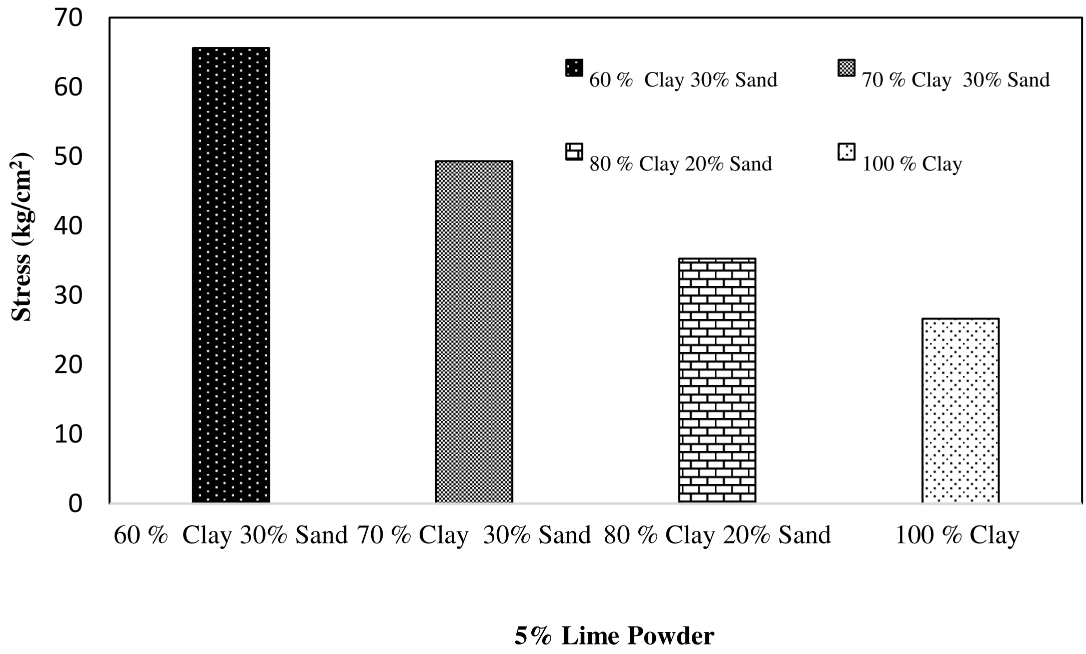



**Figure 4.12. Shear** strength chart of kaolin clay-sand mixture with 5% lime powder after 28 days.

## 5. CONCLUSIONS

In this study, a series of unconfined compressive strength (UCS) tests were conducted on clay-sand and tire crumbs mixtures and clay-sand and lime powder mixtures.

Each sample in the first class of studies were mixed with 60% kaolin clay, 40% river sand and (0%, 1%, 2%, and, 4%) tire crumbs by total weight. The sand and kaolin clay varied to (70% sand and 30% kaolin clay) and (80% sand and 20% kaolin clay) mixed with (0%,1%,2%,and,4%) tire crumbs. In the second class of studies, the same ratio of sand and kaolin clay mixed with just 5% of lime powder. Before starting, the (UCS) test the optimum moisture content ($w_{opt}$) for all the sample obtained by standard Proctor tests.

The maximum dry density and optimum moisture content of clay, tire crumbs, clay-sand mixture with tire crumbs and clay-sand with lime powder mixtures were determined by standard Proctor test in the laboratory (ASTM D698 – 07). Afterward unconfined compressive test (UCS) carried out on samples, which prepared at Proctor density. The following observations and conclusions can be made:

The maximum dry density ($\gamma_{dmax}$) of clay-sand mixture increase while the sand content increases, as well as the optimum moisture content of kaolin clay-sand mixture decreases while sand percentage increases. In contrast, adding tire crumbs to the clay-sand mixture decrease the maximum dry density ($\gamma_{dmax}$) and increases the optimum water content ($w_{opt}$)

Liquid limit (LL) and plastic limit (PL) of kaolin clay-sand mixture decreases, while sand percentage increases, but the plasticity index (PI) have significantly increases.

Throughout studies, it has founded that adding up to 2% tire crumbs to the clay-sand mixture increases the shear strength parameters. After 2%, adding tire crumbs decrease the unconfined compressive strength. However, the shear strength values for 60% kaolin clay and 40% sand mixture after adding 2% tire crumbs have 56 % increases, while with adding 4% tire crumbs to the samples the strength have 1.3% decrease. Similarly, the shear strength value for 70% kaolin clay and 30% sand mixture increased up to 50.6% with adding 2% tire crumbs, whereas by adding 4% tire crumbs to the mixture the strength values had 4.3% decreases. In the same time, the unconfined compressive strength (UCS) values for 80% clay and 20% sand mixture have 46.2% increases with adding 2 % tire crumbs, and decreases to 0.4% while adding 4% tire crumbs.

The unconfined compressive strength (UCS) values decrease when sand percentage decreases from 40% to 30%, 20 % and 0 in the mixture.

The unconfined compressive strength (UCS) for kaolin clay-sand and lime powder mixtures had tested after 28 days and the strength values changes as fallows.

The shear strength values for 100 % kaolin clay, without lime had founded 1 kg/cm$^2$, but then after 28 days with adding 5% lime powder the strength values changes to 26.61 kg/cm$^2$. In the same time, the shear strength values in 60% clay and 40 % sand mixture with 5 % lime after 28 days increased 146.5% that shows the highest value among others samples. In addition, the shear strength values for 70% clay and 30 % sand mixture with 5 % lime after 28 days increased 85.32 %. While, the UCS values for 80% clay and 20 % sand mixture with 5 % lime after 28 days had raised up 32.57 %.

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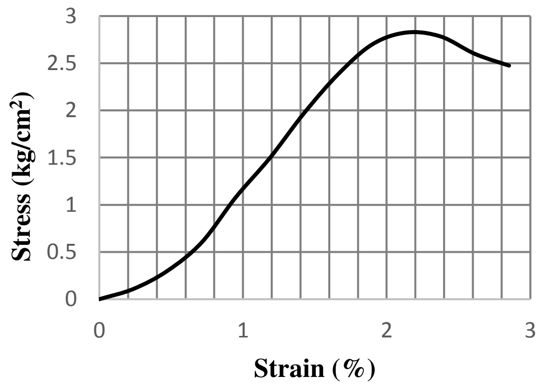
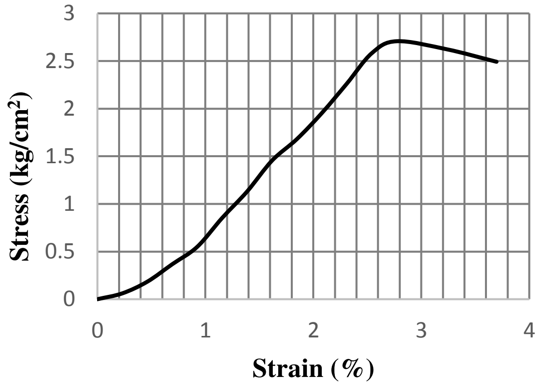
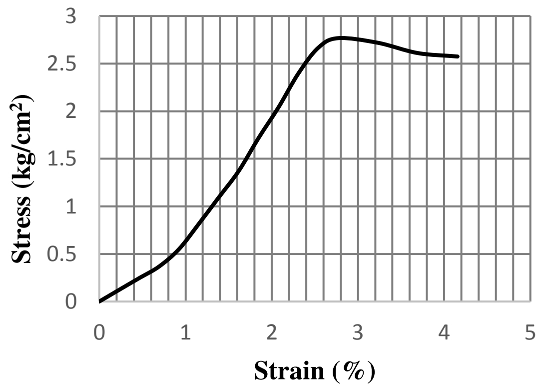
## APPENDIX-1 (Stress-Strain Axial Deformation Curves of Tire Crumbs Mixtures)



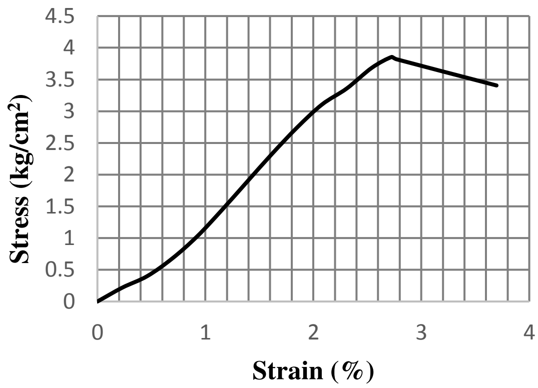

a) 1$^{ST}$. Sample (0% Tire crumbs)



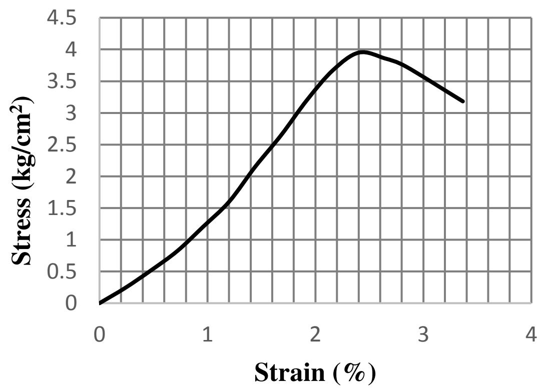

b) 2$^{ND}$. Sample (0% Tire crumbs)



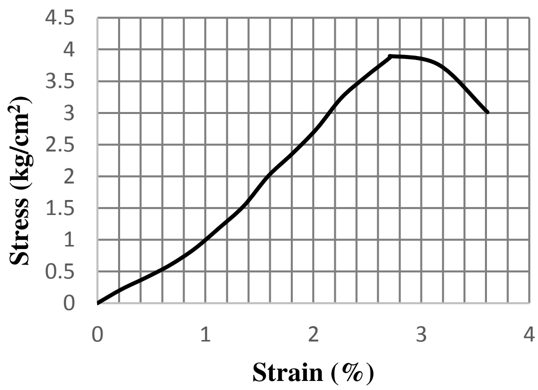

c) 3$^{RD}$. Sample (0% Tire crumbs)




d) 1$^{ST}$. Sample (1% Tire crumbs)

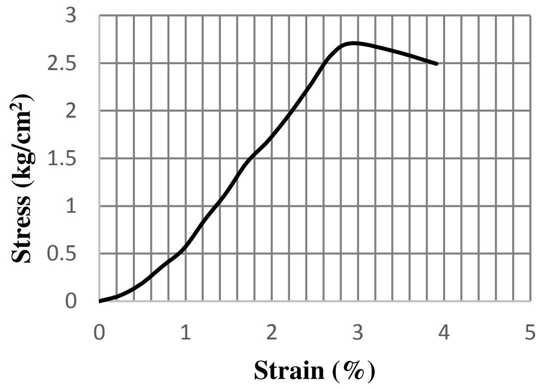



e) 2$^{ND}$. Sample (1% Tire crumbs)

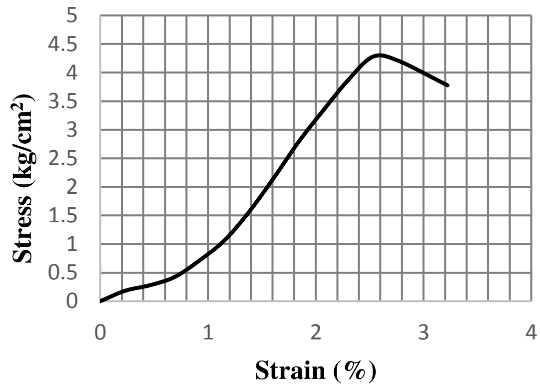



f) 3$^{RD}$. Sample (1% Tire crumbs)

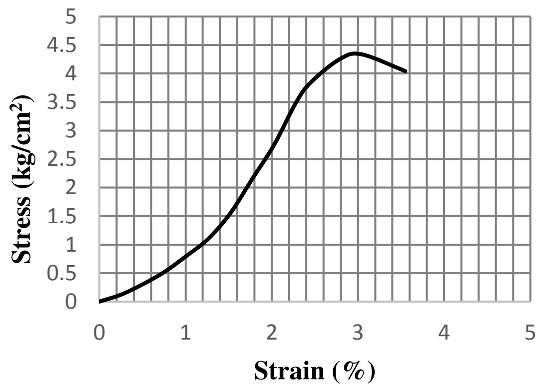
**Figure A.1.** a), b), c), d), e), f); The stress- strain axial curves for (60% Clay- 40% Sand) specimen with (0%,1% ) tire crumbs mixture**.**



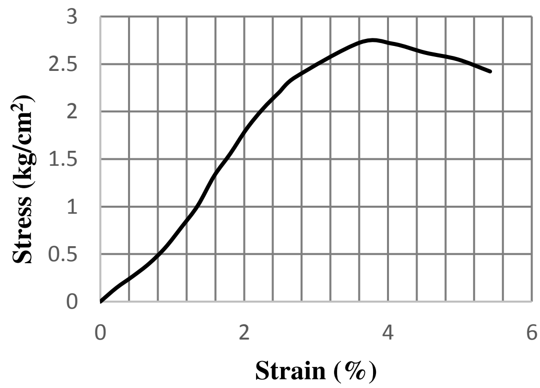

a) 1ST. Sample (2% Tire crumbs)



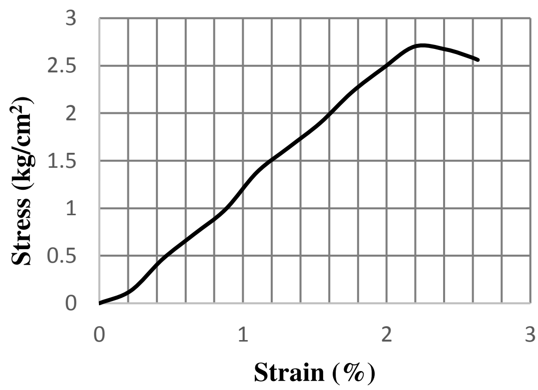

b) 2ND. Sample (2% Tire crumbs)



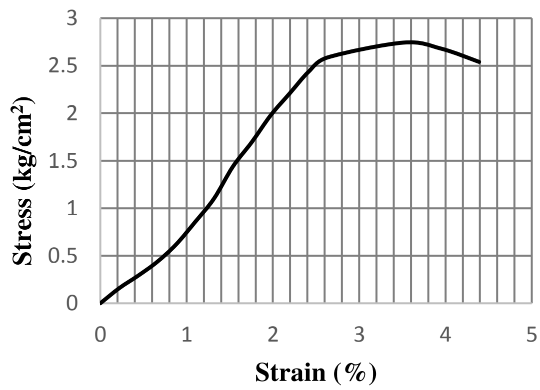

c) 3RD. Sample (2% Tire crumbs)




d) 1ST. Sample (4% Tire crumbs)

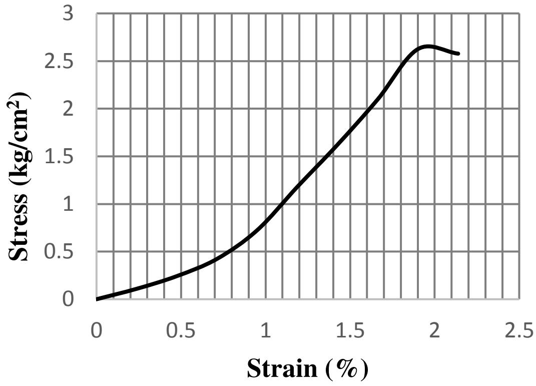



e) 2ND. Sample (4% Tire crumbs)

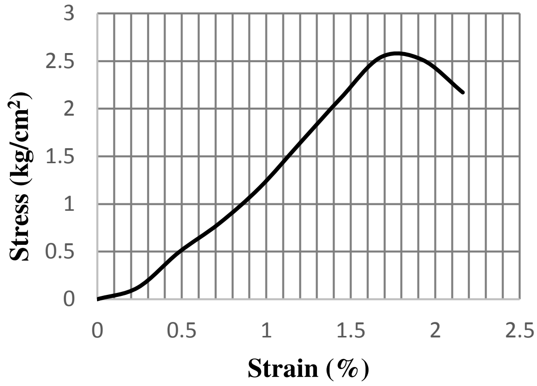



f) 3RD. Sample (2% Tire crumbs)

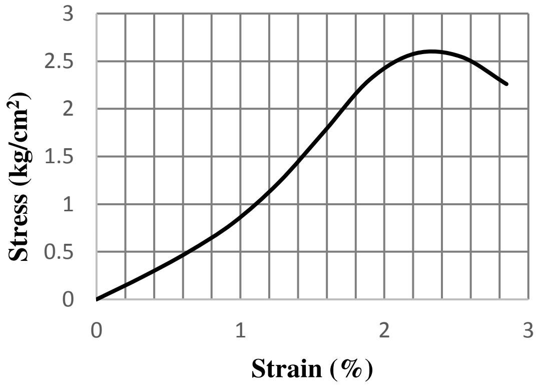
**Figure A.2**. a), b), c), d), e), f); The stress- strain axial curves for (60% Clay- 40% Sand) specimen with (2%,4% ) tire crumbs mixture.



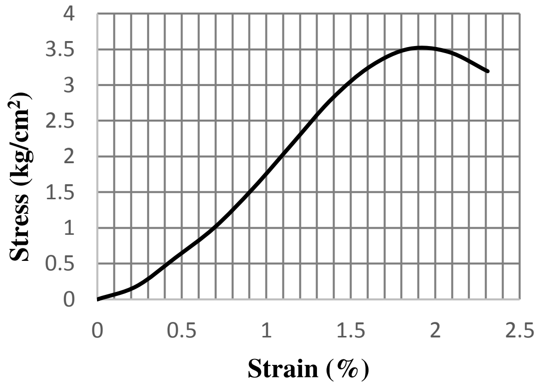

a) $1^{ST}$. Sample (0% Tire crumbs)



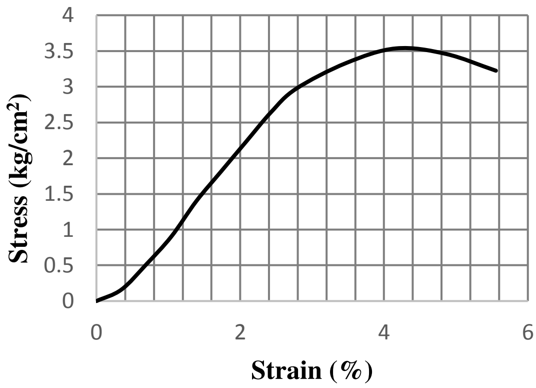

b) $2^{ND}$. Sample (0% Tire crumbs)



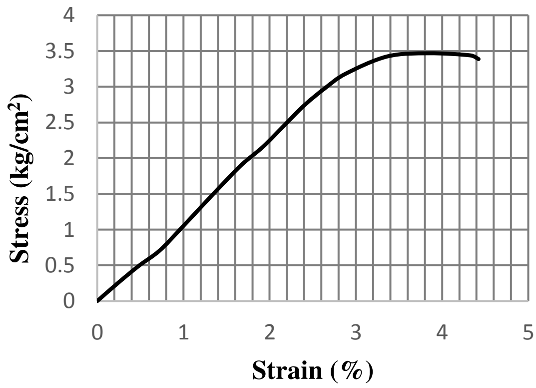

c) $3^{RD}$. Sample (0% Tire crumbs)




d) $1^{ST}$. Sample (1% Tire crumbs)

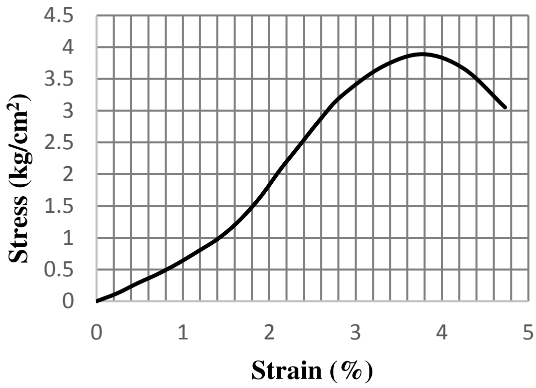



e) $2^{ND}$. Sample (1% Tire crumbs)

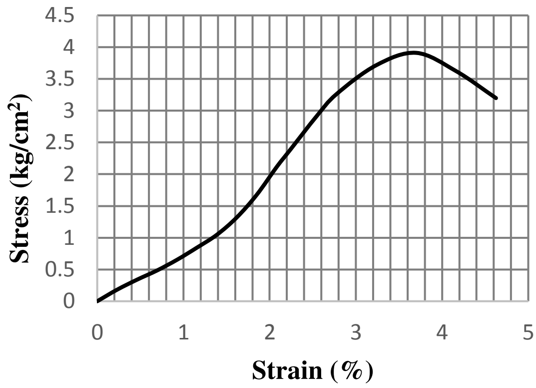



f) $3^{RD}$. Sample (1% Tire crumbs)

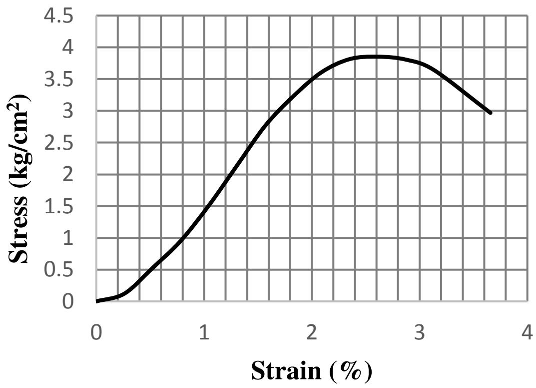
**Figure A.3**. a), b), c), d), e), f); The stress- strain axial curves for (70% Clay- 30% Sand) specimen with (0%,1% ) tire crumbs mixtuer.



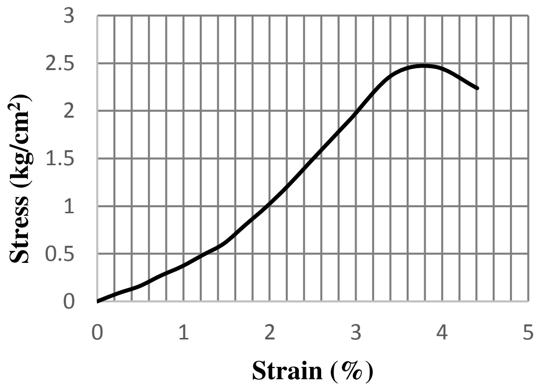

a) 1ST. Sample (2% Tire crumbs)



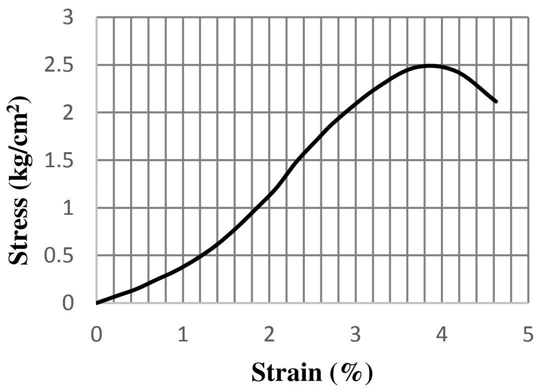

b) 2ND. Sample (2% Tire crumbs)



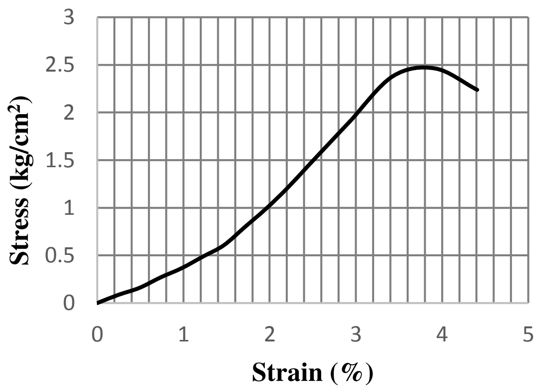

c) 3RD. Sample (2% Tire crumbs)




d) 1ST. Sample (4% Tire crumbs)

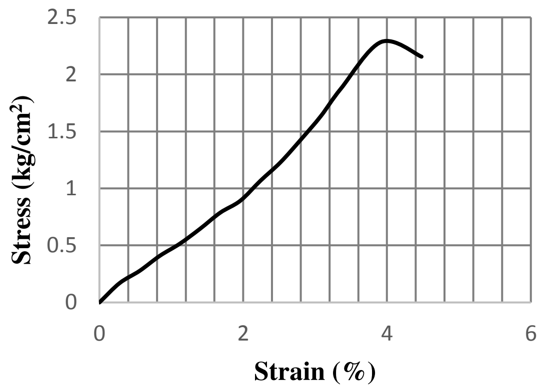



e) 2ND. Sample (4% Tire crumbs)

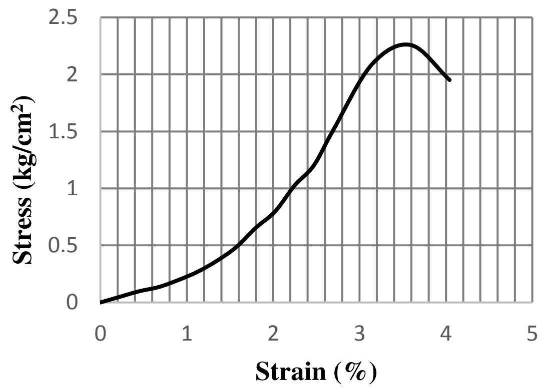



f) 3RD. Sample (4% Tire crumbs)

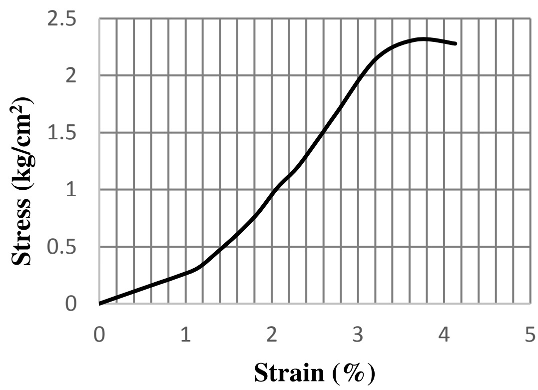
**Figure A.4.** a), b), c), d), e), f); The stress- strain axial curves for (70% Clay- 30% Sand) specimen with (2%,4% ) tire crumbs mixtuer**.**



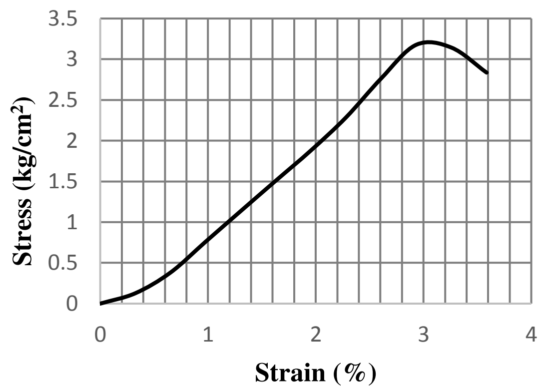

a) $1^{ST}$. Sample (0% Tire crumbs)



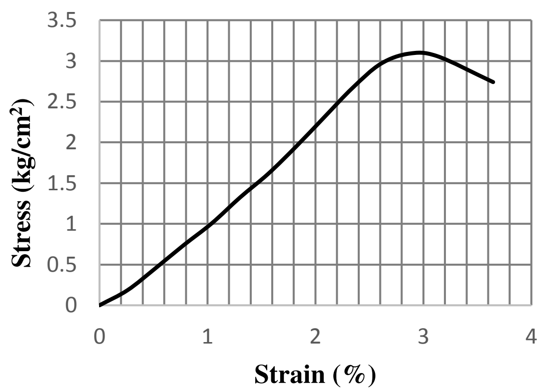

b) $2^{ND}$. Sample (0% Tire crumbs)



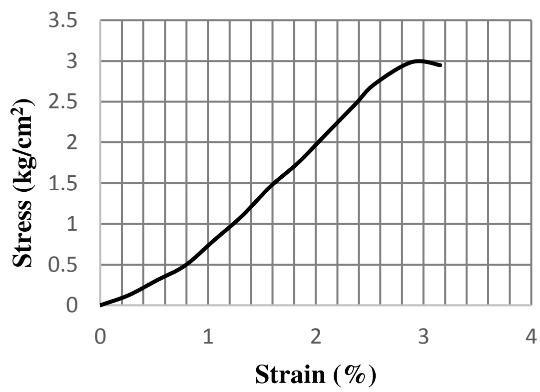

c) $3^{RD}$. Sample (0% Tire crumbs)




d) $1^{ST}$. Sample (1% Tire crumbs)

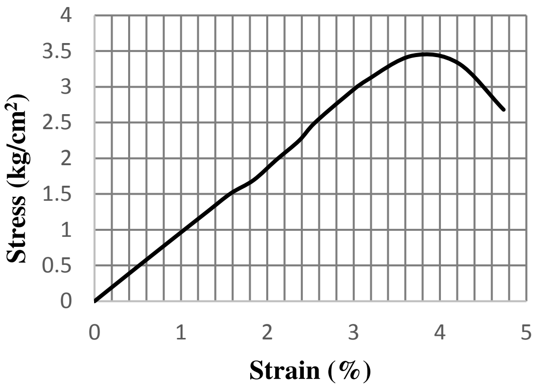



e) $2^{ND}$. Sample (1% Tire crumbs)

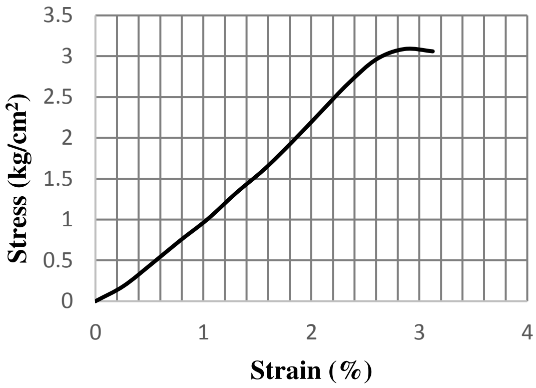



f) $3^{RD}$. Sample (1% Tire crumbs)

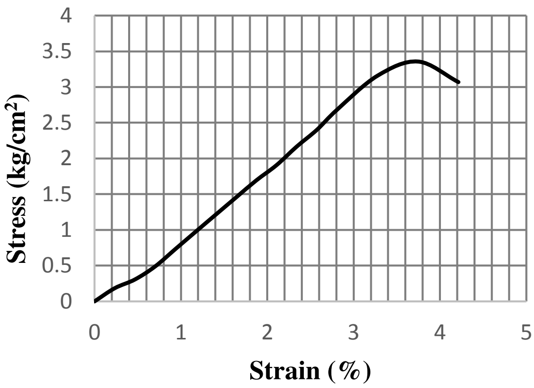
**Figure A.5.** a), b), c), d), e), f); The stress- strain axial curves for (80% Clay- 20% Sand) specimen with (0%,1% ) tire crumbs mixtuer.



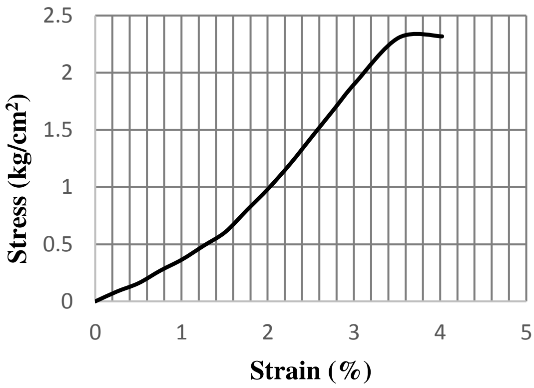

a) 1ST. Sample (2% Tire Crumbs)



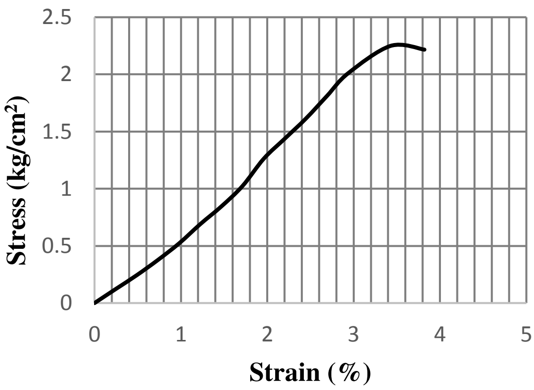

b) 2ND. Sample (2% Tire Crumbs)



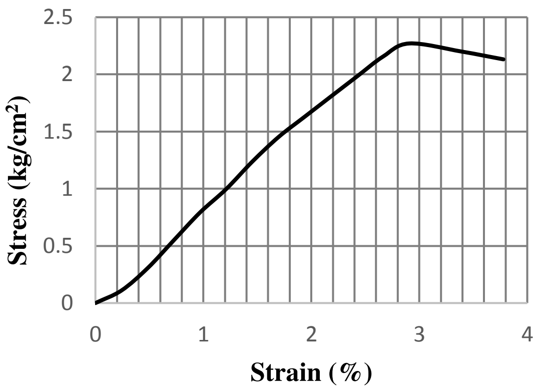

c) 3RD. Sample (2% Tire Crumbs)




d) 1ST. Sample (4% Tire Crumbs)



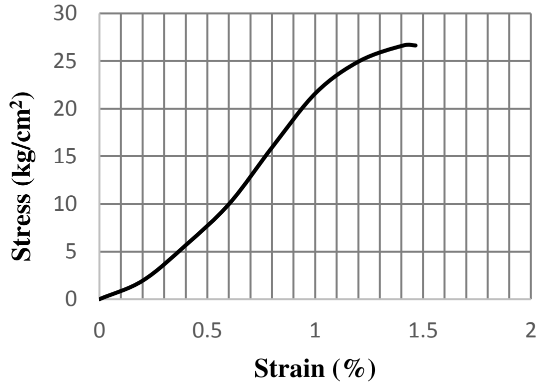

e) 2ND. Sample (4% Tire Crumbs)



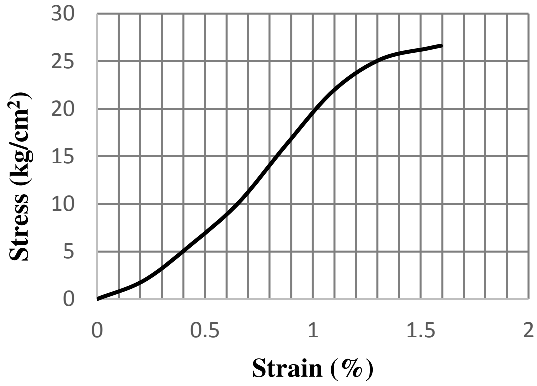

f) 3RD. Sample (4% Tire Crumbs)

**Figure A.6.** a), b), c), d), e), f); The stress- strain axial curves for (80% Clay- 20% Sand) specimen with (2%,4% ) Tire Crumbs mixtuer.

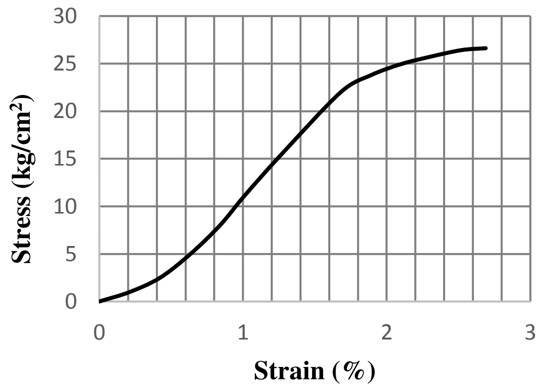
## APPENDIX-2 (Stress-Strain Axial Deformation Curves of Lime Powder Mixtures)



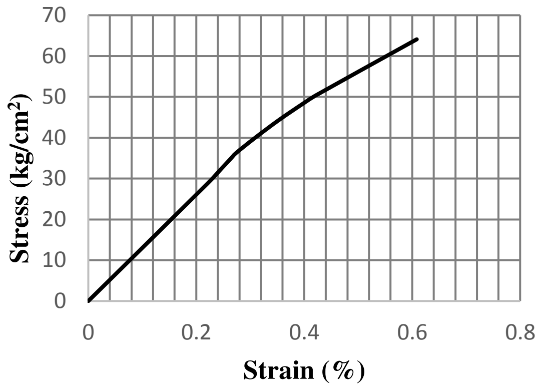

a) 1$^{ST}$. Sample (Pure kaolin clay, after 28 days)



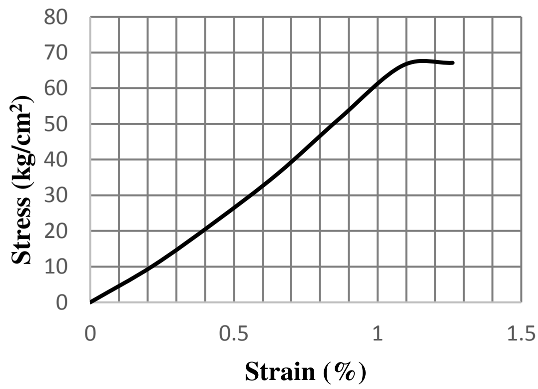

b) 2$^{ND}$. Sample (Pure kaolin clay, after 28 days)



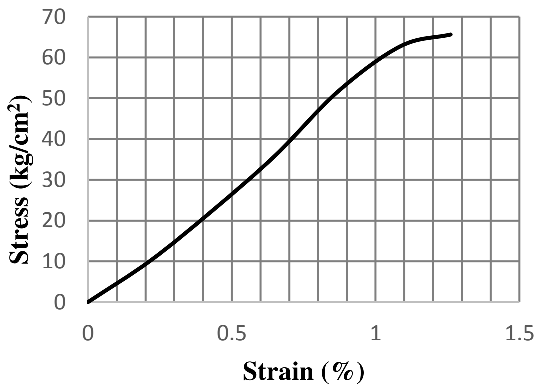

c) 3$^{RD}$. Sample (Pure kaolin clay, after 28 days)




d) 1$^{ST}$. Sample (60 % kaolin clay & 40% Sand, after 28 days)

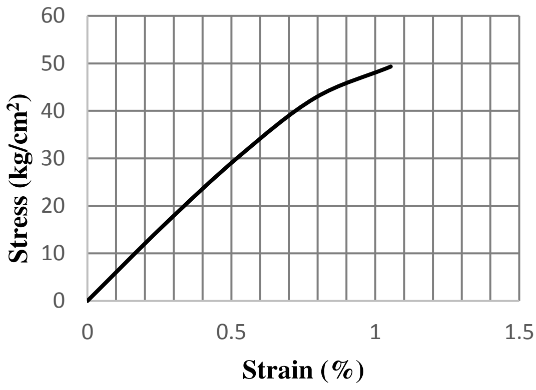



e) 2$^{ND}$. Sample (60 % kaolin clay & 40% sand, after 28 days)

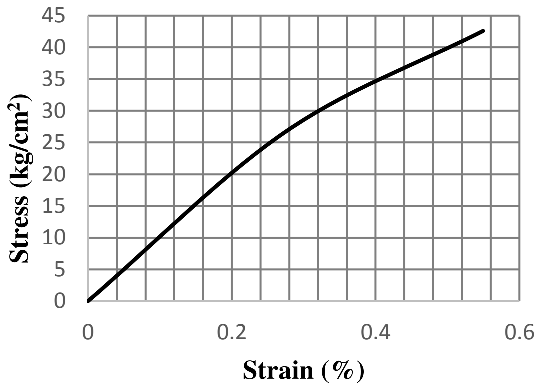



f) 3$^{RD}$. Sample (60 % kaolin clay & 40% Sand, after 28 days)

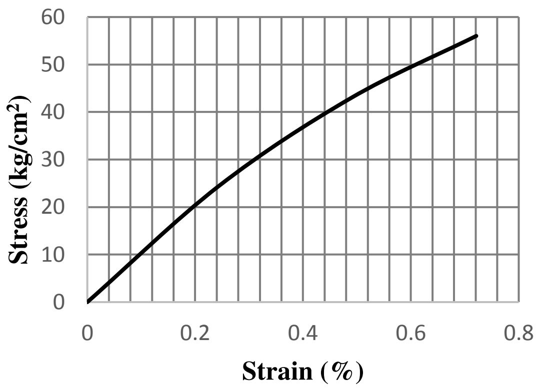
**Figure A.7**. a), b), c), d), e), f); The stress- strain axial curves for (pure kaolin clay and 60% clay- 40% Sand) specimen with 5% lime poweder.



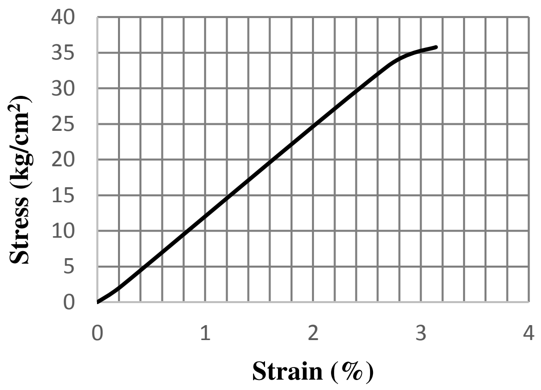

a) 1ST. Sample (70 % kaolin clay & 30% sand, after 28 days)



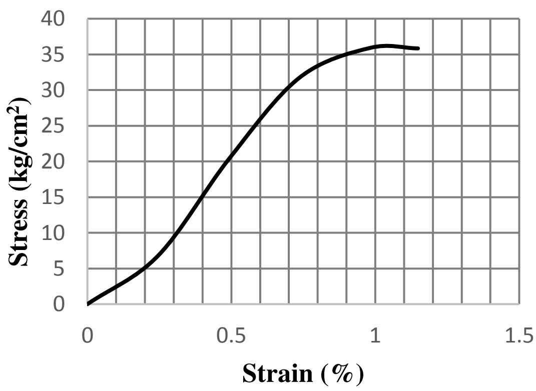

b) 2ND. Sample (70 % kaolin clay & 30% sand, after 28 days)



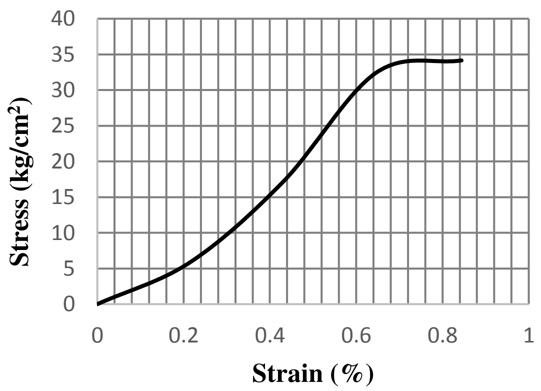

c) 3RD. Sample (70 % kaolin clay & 30% sand, after 28 days)




d) 1ST. Sample (80 % kaolin clay & 20% sand, after 28 days)




e) 2ND. Sample (80 % kaolin clay & 20% sand, after 28 days)




f) 3RD. Sample (80 % kaolin clay & 20% sand, after 28 days)

**Figure A.8.** a), b), c), d), e), f); The stress- strain axial curves for (70% clay- 30% Sand and 80% clay- 20% Sand) specimen with 5% lime powder**.**

**APPENDIX-3**, **(UCS System Calibration)**

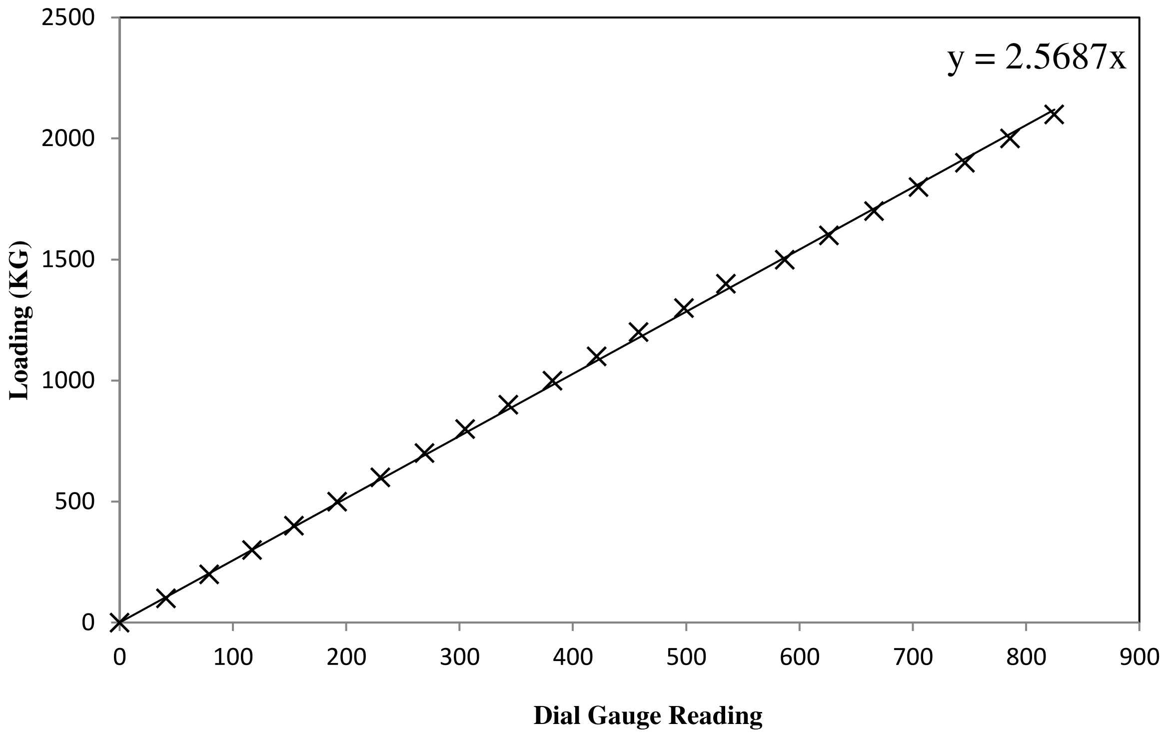



**Figure A.9.** Unconfined Compressive Test Mechanic Calibration Graph for 28kN Ring.

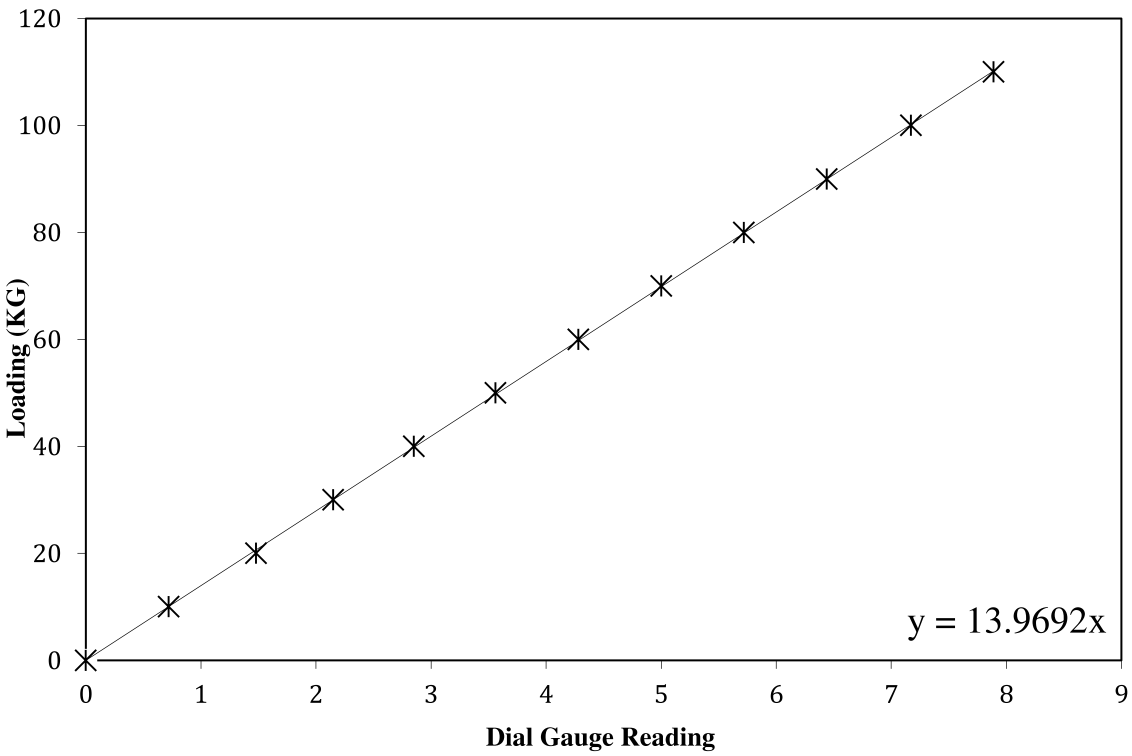



**Figure A.10.** Unconfined Compressive Test Mechanic Calibration Graph for 2kN Ring.

## RESUME

I got born in Parwan, Afghanistan in 1989. After that, I completed my primary, secondary and high school educations in Kabul. Consequently, I graduated from Kabul Polytechnic University Faculty of Geology 2012; from 2017 onwards, I am continuing my graduate education in Geotechnical Department of Department of Civil Engineering at Firat University Institute of Science and Technology.