DECEMBER 2019 M.Sc. in Civil Engineering SALAMA OMAR ALOSMAN

REPUBLIC OF TURKEY
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IMPROVING SOME GEOTECHNICAL PROPERTIES OF ORGANIC SOIL BY USING WASTE STONE POWDER

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IN
CIVIL ENGINEERING

BY
SALAMA OMAR ALOSMAN
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This is to certify that we have read this thesis and that in our consensus opinion, it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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Salama Omar ALOSMAN

ABSTRACT

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In general, improving the quality of life and the progress of industrial revolution adversely affect the urban environment. Recycled waste materials are now widely utilized in geotechnical usage, particularly in soil improvement fields. In this study, stone powder (SP) was used to investigate its effects on the stabilization of organic soil. The waste powder was blended with organic soil at different percentages of 10%, 20%, 35% and 50% by dry weight of mixture. A series of tests such as fall cone test, modified compaction test, unconfined compressive strength test and consolidation test have been performed on the mixtures. The results showed that the liquid limit, plastic limit and plasticity index decreased with the addition of stone powder. For compaction test, the maximum dry density increased by the addition of 50 % of SP and the optimum moisture content has decreased with adding 50% of SP. In addition, the unconfined compressive strength increased with adding stone powder with comparing to untreated soil and the maximum obtained value of unconfined compressive strength was at using 10% of SP. The compression index (c_c), swelling index (c_s) and the initial void ratio decreased with increasing SP ratio. The volume compressibility m_v decreased with adding more SP. Inversely, the consolidation coefficient (c_v) increased with the SP addition.

Key Words: Organic Soil, Stone Powder, Soil Stabilization.

ÖZET

TAŞ TOZU KULLANARAK ORGANİK ZEMİNLERİN BAZI GEOTEKNİK ÖZELLİKLERİNİN İYİLEŞTİRİLMESİ

ALOSMAN, Salama Omar
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Genel olarak, yaşam kalitesinin iyileştirilmesi ve sanayi devriminin ilerlemesi kent ortamını olumsuz yönde etkilemektedir. Geri dönüşümlü atık malzemeler artık geoteknik kullanımda, özellikle de zemin iyileştirme alanlarında yaygın olarak kullanılmaktadır. Bu çalışmada organik zemin stabilizasyonu üzerindeki etkilerini araştırmak için taş tozu (SP) kullanılmıştır. Bu atık toz, organik zemin ile karışımın kuru ağırlığına göre %10, %20, %35 ve % 50 oranlarında karıştırılmıştır. Bu karışımlar üzerine düşen koni deneyi testi, modifiye sıkıştırma testi, serbest basınç dayanımı testi ve konsolidasyon testi gibi birçok test yapılmıştır. Sonuçlar, likit limit, plastik limit ve plastisite indeksinin taş tozu ilavesiyle azaldığını göstermiştir. Modifiye kompaksiyon testi için, % 50 SP ilavesiyle maksimum kuru yoğunluk artmış ve %50 SP ilavesiyle optimum nem içeriği azalmıştır. Bunu ilaveten, yapılan serbest basınç dayanımı testi %10 SP ilavesiyle hazırlanan örneklerde en yüksek değeri vermiştir. Sıkışma indeksi (c_c), şişme indeksi (c_s) ve ilk boşluk oranı, artan SP oranı ile azalmıştır. Hacim sıkıştırılabilirliği (m_v) daha fazla SP eklendiğinde azalmıştır. SP ilavesiyle konsolidasyon katsayısı (c_v) artmıştır.

Anahtar Kelimeler: Organik Zemin, Taş Tozu, Zemin Stabilizasyonu.

To my life partner, who supported me and believed in my ability to reach this level of knowledge, thanks to God for having husband like you.

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I would like to dedicate this dissertation to my beloved parents, my father and my mother, they sacrificed their life to raise me to be a better person. I hope I have made you both proud.

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LIST OF SYMBOLS

LL	Liquid Limit (%)
PL	Plastic Limit (%)
PI	Plasticity Index (%)
WC	Water Content
s_u	Undrained Shear Strength (kPa)
G_s	Specific Gravity
γ_{drymax}	Maximum Dry Unit Weight (kN/m^3)
w_{opt}	Optimum Water Content (%)
c'	Effective Cohesion (kN/m^2)
φ	Angle of Internal Friction (°)
e_o	Initial Void Ratio
c_c	Compressibility Index
c_s	Swelling Index (%)
m_v	Volume Compressibility (cm^2/g)
c_v	Consolidation Coefficient (cm^2/cm^3)

LIST OF ABBREVIATIONS

OS	Organic Soil
SP	Stone Powder
MDD	Maximum Dry Density (kN/m^3)
OMC	Optimum Water Content (%)
UCS	Unconfined Compressive Strength (kPa)
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
EDX	Energy Dispersive X-Ray

CHAPTER 1

INTRODUCTION

1.1 Introduction

In general, improving the quality of life and the progress of industrial revolution adversely affect the urban environment. Throughout the world, waste production in 2002, which reach 12 billion tons (11 billion tons, which is equivalent to industrial waste), is expected to reach 25 billion tons in 2025 (Yoshizawa et al., 2004; Pappu et al., 2007). In term of expansion rate, Turkey is considered as one of the first three economies in the world. The construction is a paramount part of economy. Therefore, for a prospective economy, manufacturing and using of natural resources should be attached by environmental protection (Mymrin 1997). The most substantial step to achieve this balance is that the waste produced in one area could be reused for other areas, other industries, or for more beneficial purposes such as soil stabilization, concrete, aggregate and new materials (Karaca 2009; Gürer et al. 2004; Güllü and Girisken 2013; Zega et al. 2010;).

Recycled waste materials are now widely utilized in geotechnical usage, particularly in the soil improvement. In this study, the waste material which was selected to investigate its effect on soil stabilization was stone powder, this waste has a considerable negative impact on air, animals, water, flora and human health. So, we studied stone powder effects on organic soil to see if this waste material can fertilize and strengthen the organic soil with better properties.

In geotechnical engineering, soil stabilization is a mechanism to stabilize some engineering properties of soil, for example mechanical strength, permeability, compressibility and plasticity. Stabilization of soil could change some mechanical and physical properties of soil.

The Stabilization of soil is vastly applied in several civil applications such as railway, road construction, subbase and subgrade work. Subgrade in general consists of natural soils available locally. Pavement strength and implementation rely on the load-bearing capacity of the subgrade soil. In the condition of weak soil at construction site, the weak soil can be taken away or subrogate with another more strong soil. The studying of pavement design relies on the soil strength of the subgrade that influences the thickness of the pavement. So, in the end rising the construction cost. Additional studies were conducted on the use of waste for soil refinement of various projects like road pavements and highways (Kamie et al., 2007; Ahmed et al., 2010, 2011; Ugai and Ahmed, 2009; 2011, Khoury and Zaman, 2007, Karakus, 2011). Stabilization of soil by waste was adopted not just for change geotechnical properties but also to be a profitable method from economic and environmental point of view. However, the use of waste materials as chemical stabilizers may inversely affect some soil properties and this will cause harm to the environment, restricts plant growth and mutates the groundwater goodness (Alazigha et al., 2016; Chen et al., 2014). So, we have to be aware to which waste material we have to use and we must conduct a lot of lab experiments according to which criteria this soil is needed for.

Particularly in the city of Gaziantep in Turkey, a large amount of waste is accumulated every day, so it is necessary to analyze and re-examine "WASTES" to "WEALTH". Stone powder is one of this waste material which subjected to several laboratory tests to obtain valuable materials could be used as ground improvement and building materials through this study.

1.2 Problem Statement

Organic soil are classified as a complex soil due to high compressibility, low permeability, low bearing capacity and deformation. Organic soil does not only tend to expand and increase in its volume, but also to shrinks and decreases in its volume when general moisture conditions are pliable to fluctuate. The behavior of organic soil and clayey soils is inversely proportional to the excess of water, the plasticity develops with limited water amount (Grim, 1953; Das and Sobhan, 2013). The performance of organic soil is good and can be handled at optimal water content, but when its amount exceeds the optimum level , the stiffness and strength are significantly reduced (Dhakal, 2012). Organic soil is undesired option for engineers to construct on it. The organic soil is weaker than inorganic soils, it does not provide the right conditions for construction projects. When such soils are existed, they are usually avoided, drilled, or added with a certain percentage of another waste material in order to obtain better properties. The waste material could solve the problem of organic soil by enhance the strength of subgrade and sub base layers of pavement. Using the waste material in soil stabilization also gives the engineers the chance to construct structures on weak soils providing the decrease in cost and energy.

1.3 Objectives

The prime target of this thesis is improving organic soils to get better soil conditions. This improving is beneficial to us as civil engineers because we always try to avoid this kind of organic soils also the using of waste material is a friendly manner to save our environment from dust. In addition the large amount of energy is used in producing products from raw materials, recycling minimizes the need for raw materials then minimizes the producing of energy. In this study we used stone powder which is available easily from Gaziantep municipality so we can both decrease the amount of disposing in landfills and improve our soil economically. Several laboratory tests were conducted on soil mixtures to see the impact of stone powder on soil improvement.

1.4 Outline

This research has been summarized in five chapters, Chapter 1 describes the background and the purpose that this study carrying of for. Chapter 2 contains the history of several studies related to current research in the name of literature review. All material properties and the methods that implemented during the experimental work are included in Chapter 3.Dealing with the results obtained from laboratory tests are included in chapter 4. At the end, Chapter 5 includes the conclusions from the conducted tests through this study and the recommendations for next studies and researches.

CHAPTER 2

LITERATURE REVIEW

2.1 General

As this type of recycling has been developed in the past few years, the effect of stone powder on the stabilization of organic and expansive soils has been studied in several papers. Limited references on the types of recycling relevant to our research are listed at the last of this chapter. The main idea of "organic" soils gives a considerable range of differences related to the type of studied soil. Organic matter content which means the high percentage of organic matter into the soil is considered as the most significant characteristic of organic soil. A lot of efforts were concentrated on expansive soils and clayey soil that have similar problems to organic soil and on the soils that have high percentage of organic content. Several names are given to this type of organic soil such as “peats" ("Musk" in Canada, "Histosol" in the soil science literature) and this soil is unique in its special characteristics. Studying this kind of soil helps to identify and analyze the effect of the content of organic matter and the engineering problems which it brings to soil. The waste materials in this literature review chapter were all waste materials near to stone powder content that we used in our experiments such as lime, gypsum and other quarry stone powder wastes.

2.2 Origin of Peats

Peat mainly consists of breaking down plant material with much smaller mineral deposits. Near to 95 percent of the deposits of whole peat are composed from plants that degrade under aerobic case which is opposite to anaerobic cases that are composed in cold climate and an excess of water, degradation rates are several thousand times faster under aerobic conditions. While aerobic conditions promote growth and degradation, anaerobic conditions cause stopping of the surface growth and slow down subsurface degradation.

As a result of mentioned cases, this leads to difficult growth degradation style of moss and fibrous sedge plants. This sedge plants have more resistant to degradation than moss that cause naturally strengthen of weak moss plant. These strengthening impacts are more comprehensible, also if there are trees and shrubs in the area, soil variability is much more important (Landva and Pheeney, 1980). Despite this peat formation explaining is simplistic, it is the fundamental component to understand peats and organic soils.

2.3 Organic Soil

The general definition of organic material, all with carbon, is called "organic". Organic soil is soil that contains a certain amount of organic matter made from plant debris. It means that it is still in degradation process and needs to be fresh, so it has unique smell, texture and color (Huat et al., 2014). Organic matter in the soil consisting of organism such as remains of animals and living plants. Marine flora and fauna particularly, give a share in the formation of organic matter in the soil. Organic matter such as peat and coal is created through the process of plant transformation Larsson (1980). Stevenson (1994) mentioned that organic soil materials retain complete organic matter in the soil, light pieces, trash, water soluble organics, stabilized organics which give the stiffness of organic soil. Chikyala (2008) pointed out the ability of organic matter to fall into two major classes. Transfer of non-living organisms and living organisms to organic soil mainly due to bacterial activity. For organic fashioning, the deposition rate must be faster than the degradation rate. McDonald (1993) stated that the accumulation of organic matter takes about 500 years for 30 cm of organic soil accumulation.

2.4 Classification and Index Properties

The most used standard for soil classification is the Unified Soil Classification System (USCS) because it is the oldest system also the variations of this system are probably the most vastly used format of soil classification. This system is the best system for engineering requirements. It was developed from the system suggested by Casagrande (1948), it is called the airport classification system. Which is consisting of applying group names and group symbols to the soil, relating on composition, Atterberg's limitations and the distribution of particle size. This system studies the organic soils in three classifications: peat, organic silts and organic clays. Organic clays and organic silts meet the requirement that they have been classified as clays or silts, except that the liquid limit is decreased by 75% after oven drying from its value before oven drying.

The peats are defined by the USCS system as follows: “Soil consisting originally from plant tissue at several juncture of degradation, with an organic smell, near to brown color and composition changing from fibrous to amorphous (ASTM D2488-93). Particle size distribution makes the classification of organic clay and silt more complicated, because the peat classification using this system depending only on the mentioned definition before. So, for more certainty it requires other systems to be used for better specification for identifying peats. There are a lot of classification system for peat and organic soil, the most widely used ones to describe and classify peat are these three classification systems which are: Von Post, Radforth and ASTM. The classification system of Rad Force depending on organic content percentage which should be more than 80% (ie> 80%). The visible features and identifications of texture and plant content is the way which this system deal with, despite of there is no particular mention is made to algae type plant material. The peat is presumed to be completely organic, although water and wind transport colloids may be exist (Hwang et al, 2004). The system offers 17 various categories depending on the size of fiber presented in soil and the composition of the plant (see Table 2.1). These characterizations do exclude exact nor specific data about the particular direction and quality properties of the fibers. Most essential instruments utilized in this order are apparent distinguishing proof by hand test in the field, may be also by microscopic photographs and x-rays of undisturbed examples. The makers realize that in spite of the fact that a peat class may incorporate a few kinds of peat, no particular sort can have a place with different classifications (Radforth, 1969). Not at all like the Radforth characterization system which gives watchful classes, the peat grouping which utilizing the Von Post system is conducted during task of an arrangement of code letters relying upon the organic issue kind, deterioration level , wealth of assigned fiber sizes, water content, and plenitude of wood and bush remainders .This system gives some numerically explicit records to use in system, explicitly for detachment of fiber sizes and the assessment of water content (Landva and Pheeney, 1980). In contrast to the past order classification system, The Von Post system comprises from big number of potential outcomes for different classification systems. For instance, the total Von Post designation for a peat comprising of gently disintegrated, very wet sphagnum greenery peat, with some sedge filaments littler than 1 mm might be assigned by: SCH3B5F2R0W0. This equivalent soil utilizing the Radforth framework would only need a designation of classification 2 or 3 (Hwang et al, 2004).

In spite of the presentation of some quantifiable characteristics contrasted with the Radforth classification, the Von Post system additionally uses visual-manual identification strategies. These strategies are utilized especially for assessing the level of deterioration. The level of deterioration which otherwise called level of humification which is the most noteworthy commitment from the Von Post framework. This system is combined with fiber content in the ASTM standard for peat classification and is additionally the beginning for the terms “amorphous” and “fibrous” peat. An "amorphous” peat is what is in cutting edge phases of deterioration and has a basically granular surface, while a fibrous peat compares to generally beginning periods of decay in which stems and plant parts are as yet unblemished. These two textures might be contrasted with jam and fiberglass matting, respectively. The more granular sorts of peat more often than not compare to those containing a lot of greenery plants, while the fibrous peats would contain more sedge and "woodier" type plant materials (Landva and Pheeney, 1980).

In spite of the general benefits of the recently referenced characterization frameworks, the most widely recognized and the simplest framework to use for classification of peats is that utilized by the American Society for Testing and Materials (ASTM) because this system gives a significantly more institutionalized structure and its quantitative nature lessens the vulnerability related with assigning the soil classification. The definition for peat as expressed by ASTM seems to be: "a normally happening very natural substance got basically from plant materials. Peat is recognized from other natural soil materials by its lower ash content (under 25% ash by dry weight), and from other phytogenic material of higher position (that is, lignite coal) by its lower calorific incentive on a water soaked premise (ASTM D 4427-92)." More arrangement of peat is drawn nearer through the quantitative portrayal of fiber content (identified with level of humification), ash/organic substance, absorbency, acidity and botanical substance (see Table 2.2). In spite of the fact that this framework is not especially nitty gritty regarding herbal cause, the assignment of plant types is took into consideration gave that a noteworthy level of the peat begins from a specific plant. Regardless of this, the ASTM classification system appears to give a considerably more succinct and quantitative methodology than the recently referenced systems given the well-characterized and quantitative classifications.

Notwithstanding, the grouping frameworks mentioned before and archived in the writing, the Indiana Department of Transportation (INDOT) offers its rule to recognize peats and organic soils in Section 903.05 of the INDOT Standards Specifications. This expresses an "organic soil" contains between 19-30% organic materials, while a "peat" contains a measure of organic material more prominent than 30%. This grouping is significant since it have been gotten from the AASHTO standard T 267.

2.5 Strength of Organic Soil

Shear strength of soil is one of the fundamental issues because of its consequences for structure footing or any parts contact with soil. This factor was so significant especially during the pre-and post-development stages, since it used to get values of slope stability and footing of soil, if this values exceed the value of ultimate shear strength this lead up to soil fail or deformation. The theories of elasticity and stress-strain relationship were used for developing the failure condition as Huat et al (2014). Strain estimations of soil relied upon numerous elements like soil composition, measure of connected burden, void proportion, past pressure history, and furthermore the technique of applied stress as indicated by Anggraini (2006). Organic soils are found in some geological zones, as a result of their high water substance and organic solids, it shows mechanical characterizations to some degree unique than the typical conduct of inorganic soil as indicated by Edil and Wang (2000). On the other hand, understanding the conditions of shear strength and in-situ stress is very important in the design of structures on organic soils (Edil and Wang, 2000). As typical the shear strength parameter changes with both of consolidated soil and plasticity concurring (Larsson, 1980; Edil and Wang, 2000). Arman (1969) found that adding fibrous organic materials increases the shear strength of the soil. Mitchell and Soga (2005) inferred that the organic materials act in soil as reinforcement which lead to increase in strength value. Mesri and Ajlouni (2007) demonstrated that the friction angle of organic soil or peat soil is in the scope of 40º to 60° contrasted with the value of friction angle of soft clay which is under 35°. The increasing in friction angle value does not mean there will be increasing in the value of shear strength of the soil, as not all fibers are solid because of the water and gas containment. Regardless of the fiber content of organic soil, they mentioned it has an impact on the friction angle as expressed by Edil and Dhowian (1981). The impact of studying the reinforcement of fiber and its orientation provide a considerable intricacy in the explanation of fibrous peat. Using some scale impacts could unnaturally lead to raise the reinforcement mechanism of the fibers (Wang and Edil, 2000). For instance, a 2 cm long fiber might be basically insignificant in the field, but the existence of comparable fibers in triaxial specimens may artificially provide high value of strength; whereas the concentrate on non-idealistic soil use and testing has gained increasing attention lately in the field of geotechnical engineering.

2.6 Consolidation Behavior

The origin of organic soil which is result of forming peats gives soil some properties such as the abundant of water and low content of minerals, the way that the soil is founded near the surface grants this soil low density with slightly porous fabric which is characterized by high porosity in the field followed by high permeability (Fox and Edil, 1996). Therefore, it is very essential to comprehend the behavior of consolidation to construct and design on this type of soil. Consolidation is the gradual decrease of the volume of fully saturated soil under sustained loading, mainly due to the release of water from soil pores. The most used and simple form of consolidation test is the oedometer test which includes soil specimens that are laterally constrained then axially loaded by applying a static force. The deformation that happens to soil specimen and the elapsed time establish the primary and essential data of test. The obtained data can be utilized in expect the magnitude and the rate of settlement in normal field situations, Therefore, it is pivotal information for nearly all geotechnical applications (ASTM D2435). In spite of this simplistic way of describing the consolidation test, the genuine test and consequent translation may turn out to be very included and complex. Meyer and Bednarik (2008) have set up a study to estimate the expected soil reinforcement due to consolidation loading. This study showed the obtained results of oedometer test of soil consolidation from the Ostrów Grabowski site of Szczecin harbor. For starting the test an appropriate specimens were gathered from the site, loaded first and unloaded and reloaded. The basic parameter of this test is elastic modulus. The results plainly display that while the stress-strain relationship is clearly non-linear under initial loading, the relationship tends to be linear after re-loading. Malinowsk and Szymański (2014) investigated the likelihood of deriving the partial differential equation integration by nonlinear flow characterization on organic soil model, for example, the organic soils. This soil has high compressibility, so the outcomes in permeability fluctuations. The modeling of deformation that happens because of applied load on structure is very essential because even if the stress is really small the effect may be very high, a lot of constructions such as road embankments, dikes, flood control embankments and dams are frequently situated on organic soil, So it is a crucial step to model and specify all deformations of organic soil. Therefore, from this study, a non-linear theory on the consolidation of organic soils has been presented, taking into account the difference in porosity and the flow rate.

2.7 Collapse Characterization

Volume change is a standout amongst the most significant issues that the specialists and engineers face in huge regions of the world, particularly in regions with continental and subcontinental climates. Volume changes, particularly the collapsed situation, cause critical material misfortunes because of genuine harm to structures, bridges, roads, airports and other designing structures. Organic soils are most likely to collapse as the water content increases. Potential solutions or obtainable alternatives may be included in; (a) subtractive folding soil, (b) compaction techniques, (c) wetting avoidance, (d) chemical stabilization or injecting, (e) pre-wetting, (f) planned wetting, (g) deep dynamic, (h) pile or pillar foundations, and (i) differential sedimentation resistance foundations (Houston et al. 2001). Ayadat and Hanna (2005) have experimentally investigated the performance of stone columns condensed with geofabrication reinforcement installed in a collapsible soil layer and flooded. According to the experimental results, it can be concluded that unreinforced sand columns in the collapsible soil do not make a significant contribution to the soil performance. Stone sections conceivable to use in transmit balance burdens to suitable bearing strata under layer soil collapsible. As indicated by the consequences of the test think about, it is may be the reason that unreinforced sand sections in collapsible soil did not contribute huge in execution of the soils.

2.8 Improvement of Organic Soil Techniques

As the weak and problematic lands become scarce day after day and the construction on them is more costly and expensive. The engineers are compelled to construct and design construction structures that do not meet the coveted soil conditions. Construction of levees is increasingly needed even on organic soil deposits. These deposits may have just simply avoided before. The next discussion examines the most typical strategies that were utilized practically when the problems of swelling and compressible organic soils are existed (Hwang J et al, 2004).

The most established evident technique for dealing with compressible soils is to evacuate the undesired soil and supplant them with more effective fills. This method is unbelievably viable and straightforward strategy. This strategy demonstrates for becoming useful if the quality of fill is accessible and the studied soil is a moderately thin surface layer but if the peat layer did not removed very well, the compression of filler material may be difficult and cause undesirable displacement. In the event that the peat layer is not totally expelled, the compaction of fill materials might be troublesome which lead to undesirable removals (Sasaki, 1985). The use of surcharge is another regular technique that may guarantee that settlements because of ensuing development are restricted. In this strategy, a layer of extra charge fill is put over the organic soil then afterward expelled after the proper preloading time. This state is called over consolidated state bringing about littler settlement with high shear quality. The deterrent of this used strategy is the fact it requires a very adaptable and extensive development period (Sasaki, 1982). Another method to solve problematic soil issues is chemical treatment. The method of chemical modification is about changing the content of soil by adding some binding components to soil. The implementation of this way for mass stabilization is by staunch binder into problematic soil to compose a stabilization column or block (Åhnberg and Holm, 1999). The most well-known binding materials utilized are: quick lime, fly ash, gypsum coal slag, Portland cement and other pozzolanic materials. All these binding compounds act to raise the pH value of soil environment above 12.4 value, which permits the alumina and silica of the soil to become available for pozzolanic reaction (Esrig, 1999). All these binding compounds act to raise the pH of soil environment up to the value of 12.4, which permits the alumina and silica to become ready for the pozzolanic reaction (Esrig, 1999).

2.9 Admixtures in Pavement Designs and Strength Properties

There is a significant role for the admixtures and they can utilized quickly in the applications of roadway for different asphalt "layers." There are a lot of advantages to utilize waste material in subsurface asphalt layers, for example, cost reserve funds in materials through a decrease in layer thickness, decrease in decay rate of asphalt layers, upgrade seepage characteristics and reduce in maintenance (Nicholson, 2014).

The most significant uses of utilizing lime as well as concrete admixtures on the asphalt is in the curing of subgrade soils or as the capacity and quality of the subgrade soil is upgraded, the entire quality and ability of the completed roadway are improved. The increasing hardness, strength and permanency feasible with common admixtures take into consideration reducing the thickness necessities for subsequent base layers and the plan graphs. The used rules to describe the minimum thickness of pavement are in general based on the strength limit of stabilized soil layer (Nicholson, 2014). On account of expansion, lime can be utilized to stabilize calcareous major aggregate and clay-contaminated major aggregate as per pavement thickness structure. The advantages of adjustment by utilizing lime for calcareous bases have been built up in South Africa, France, and United States. Then again, it appears that the utilization of lime has the likelihood to improve numerous territories worldwide where calcareous aggregate are considerably more predominant than higher-quality. The hydrated lime and igneous rocks are additionally here and there utilized in asphalt mixtures as it can extensively improve the cohesion when it responds with contained fine substances.

2.10 Stabilization of Soil by Waste Materials

The waste material that might be added to the soil for stabilization is extensive and has variability in nature, morphology and quality. These ordinarily run from naturally happening soils to chemical extracts and also recycled products. There are a portion of the exploration works which performed by scientists that have been clarified beneath:
Al-Rawas et al. (2005) had investigated the utilization of lime, cement, the mingling of cement and lime, (artificial pozzolan) Sarooj and warmth treatment to stabilize the studied soil. These used stabilizers were blended with the soil in various extents for getting test samples. The test outcomes demonstrated that the swelling pressure and swelling percent are going to decrease with adding more content of used stabilizer in the soil samples but for sarooj the opposite happened they noticed increasing in swelling pressure particularly when the substance of Sarooj was 6% and 9%. However the best results were noticed by adding 6% of lime to the soil samples which reduced the swelling pressure to zero.

Praveen Kumar et al. (2006) demonstrated the California Bearing Ratio (CBR), static and cyclic triaxial test on four waste materials, for example, fly bed, stone powder, coarse sand and River Bed Material (RBM) that utilized in the layers of subbase of an adaptable pavement. The CBR value of stone powder demonstrated the highest value compared with other waste materials. Yet the attitude of stone powder under dynamic status in triaxial test was low with comparison with other materials. However, the CBR value of fly ash was the lowest one but it has better stress-strain characteristics than stone waste.

Solanki et al. (2007) clarified microstructure and building properties of the dirt blended with waste cement kiln dust. They demonstrated that the obtained values of unconfined compression strength, hard modulus and modulus of elasticity of cement kiln dust blended with soils more noteworthy than concrete kiln dust alone. According to the scanning electron microscope photomicrograph, chemical reactions observed as a result of the reaction occur in the soil voids. This suppresses the increase in elastic modulus and strength.

Ene and Okagbue (2009) investigated the pyroclastic rock dust to see its effect on the properties of expansive and problematic soils. Several tests were conducted on different mixtures of pyroclastic dust such as; proctor compression test, CBR test, plasticity test, linear shrinkage test and shear strength test. The outcomes indicated that there were increasing in the values of optimum water content, maximum dry density, shear strength and CBR value with the increase of pyroclastic dust. In opposite for linear shrinkage and plasticity the results showed decreasing in their values with increasing proportion of pyroclastic dust. At 8% of pyroclastic dust the maximum values of CBR were obtained.

Brooks (2009) had modulated expansive soil by using fly ash and Rice Husk Ash (RHA). The performance of stress-strain of compressive strength limitations showed that the strain step by step increased and when fly ash increased from 0 to 25% the stress failed. Likewise there was raising by 97% of unconfined compressive strength, and CBR improved by 47% when the percentage of Rice Husk Ash (RHA) raised from 0 to 12%.

Karakus (2011) had thought about the impact of utilizing Diyarbakir basaltic waste for Stone Mastic Asphalt (SMA). In spite of the fact that it is a new technique in Turkey, that the asphalt improved by stone mastic for street development this technique has been utilized in the USA Europe 40 years ago. The composition of this waste is mainly from aggregate and inorganic fillers of 93% to 94%, bitumen of 6% to 7% and added substances. Training on the utilization of basalt residue and aggregate waste is proposed to be done likewise in the regions of concrete and generation chemical substances.

Gandhi (2012) succeeded in stabilizing expansive and roadbeds soil by adding bagasse ash. The effect of adding Bagasse ash was gradually in drying the saturated soil and providing an initial rapid strengthening. This is worthwhile when building in wet and unstable ground conditions. The results showed that the swelling potential of expansive soil was decreased by replacing some of the amounts previously held to assess its potential to see the possibility of use it in industry. Different tests were conducted using different bagasse content 0%, 3%, 5%, 7% and 10% respectively .The tests are liquid limit, plasticity index, plastic limit, shrinkage limit, swelling pressure and free swelling. From these tests it has been found that as the proportion of bagasse ash increases, there were decreasing in the mixture properties and values.

Sabat and Bose (2013) had applied several tests trying to improve expansive soils by using quarry dust and fly ash. The Fly ash were added at this percentages (0%, 5%, 10%, 15%, 20% and 25%) and quarry dust were added using five percentages (10%, 20%, 30%, 40% and 50%) by dry mass of the soil. The mixtures were mixed in such a way that their total percentages is 100%, like soil: fly ash: quarry dust: 70:10:20 etc. The reports demonstrated that the ideal content of fly ash-quarry dust is 45%.The MDD has increased and the OMC has decreased with increasing the content of flay ash and quarry dust. Maximum value of UCS was at 45% of fly ash-quarry dust mixes and after that it decreased. For swelling pressure, if the proportion of the fly ash quarry dust mixture is 45%, we can see that the expansion pressure is 12 kN / m^2. It is safe to build pavements on inflatable soil subgrades stabilized with this proportion of fly ash quarry dust mix.

Cabalar et al. (2014) had announced that tire buffing could be utilized for stabilizing soil on the chance that it blended with a specific amount of lime. The tire buffing substance in the blends were 0%, 5%, 10%, and 15%, and lime proportions were 0%, 2%, 4%, and 6% by dry weight of the blend. Full-scale laboratory tests were performed on specimens containing CBR, UCS and swelling tests. Studies have shown that as tire buffing increases, CBR, UCS and swelling values decreased. The results showed have also shown that the CBR reaches its high value when the lime content that utilized in the blend is 4%, implies that the ideal measure of lime substance is 4%.

Modarres and Nosoudy (2015) studied the technical and environmental effects on lime and coal waste to stabilize clayey soil. They distinguished that CBR at OMC effectively increased by utilizing lime and waste of coal. The immersed samples of hydrated lime had essentially better impacts on balancing out swelling properties. In addition, the acquired results showed that the value of compressive strength and CBR containing coal waste powder is less than soil which included lime activated by coal waste powder.

Deivanai and Arumairaj (2016) described the stabilization of subgrade of soil by reinforced this subgrade by using natural fiber. They have specified that as the fiber content increases, the soil's soaked or unoaked CBR increases. The increase in CBR of reinforced soil is also confirmed to be equivalent to 1% for coir fibers, 1.25% for bamboo fiber content and 0.75% for jute and sisal fibers. This stabilization lead to reduce the thickness of pavement as result of increasing CBR value.

2.11 Improvement of Organic Soil Using Waste Materials

The utilization of waste materials to stabilize organic soils has two preferences; (i) Environmental and financial advantages of material disposal that adversely affect the environment, and (ii) the building advantage of improving soil properties with geotechnical issues.

Kolay and Taib (2011) examined the likelihood of utilizing the class F pond ash (PA) from a coal-terminated which is gotten from thermal station in stabilization of organic soil. The specialists investigated the impact of different content of PA (i.e. 5%, 10%, 15% and 20%) on the compaction and uniaxial compressive strength UCS features of organic soils. The outcomes of this study showed that MDD of organic soil increased opposite to OMC which decreased with increasing PA content. The UCS value was increasing with more curing time and it increased obviously with the increase of PA percent in soil mixtures. The compressive strength results of organic soils showed that the PA can possibly be utilized as a stabilizer for organic soil. Furthermore, the utilization of PA in soil adjustment helps in decrease pond volume and accomplish environment amicable and possible growth of resources.

Mukri et al (2013) researched the impact of utilizing the waste paper sludge ash (WPSA) as a stabilizer material for organic soil. The organic soil was blended at five different extents WPSA at 2%, 4%, 6%, 8% and 10% of dry weight of mixtures for deciding the ideal content of (WPSA) to settle organic soil at the greatest value of compressive strength. This examination showed that the waste paper sludge ash (WPSA) has the ability to stabilize soil and give an effective outcome to upgrade the organic soil strength for long terms. The strength has increased at nearly 7% of (WPSA) to reach the value of 290 kPa.

Rahgozar and Sabreian (2016) completed an exploratory examinations on the impact of utilization of shredded tire chips on bearing capacity and stability of organic soils. Organic soil samples from Chaghakhor Wetland (Chaharmahal and Bakhtiari province in Iran) were blended with sand at a constant dosage (400 kg/m^3) and different rates (0%, 5%, 10%, 15%, and 20%) of shredded tire chips. Each of UCS, Φ and c' were estimated to these different mixtures. The outcome of unconfined compressive strength test and direct shear test showed up that the sample with shredded tire chips has influenced on improving some properties of organic soil. Blend with 10% of shredded tire chips showed the most noteworthy value of UCS; the mixture with 15% of tire chips demonstrated the most elevated ductility; and including 20% of shredded tire chips gave the most astounding value for the cohesion and friction angle.

2.12 Improvement Technics Using Stone Powder

The crushing of stone gives stone powder which is a problem for the effectiveness of disposals. A large amount of dust is generated in crushed stone. They are often regarded as waste in the area. They were thrown here and there without any interest (Ahmed and Yusuf, 2009). While landfills are normally utilized for transfer of stone residue in Bangladesh, quick urbanization has made it progressively hard to discover appropriate landfill destinations (Lin and Weng, 2001). A few endeavors are seen obviously in various specialists' movement (Mahzuz et al., 2009; Kameswari et al., 2001; Sanchez et al., 2003; Shih and Lin, 2003) to discover appropriate use and transfer of waste.

There are various examinations on the utilization of marble powder in the arrangement of sub-bases in highway development and the re-use of waste as aggregate in asphalt pavements (Akbulut and Gurer 2006; Misra. And Grupta 2009; Üstünkol and Turabi 2009).

Soosan et al. (2001) announced that the smashed residue displays high shear quality and is valuable as a geotechnical material. This study clarified that the stone powder is quantifiable which contains pozzolanic, and furthermore coarser in it. However, waste materials, for example, fly ash, have a solitary pozzolanic property and do not contain progressively granular particles.

Soosan et al. (2005) investigated the impact of quarry dust on three types of soil (Cochin marine clay, Red Earth and Kaolinite) to get better geotechnical properties of the studied soil for construction of highway. He observed that, the more percentage of quarrying dust improves some test values like CBR value of the soil, the ideal mixture is 40% of quarry dust and 60% of soil.

Khan (2005) studied and applied several tests on waste marble powder to be used as stabilizer in cohesive soils stabilization and then to use this stabilized soil in the dam cores construction, he found that the permeability of treated samples decreased and the durability increased.

Akbulut and Guler (2006) have shown that the marble from quarries is possible to be reused as aggregate for lightweight and medium trafficking asphalt binder layers.

Suresh et al. (2009) studied the effect of Polypropylene fibers and stone powder on the properties of Black Cotton Soils. The scientists carried out the tests using stone powder percent (0%, 1%, 2%, 3%, 4% and 5%) and the percent of Polypropylene fibers were (0% 0.3%, 0.6%, 0.9% and 1.2%). California bearing ratio, compaction characteristics, unconfined compressive strength tests were applied on the stone powder mixture alone, Polypropylene fibers alone and stone powder with Polypropylene fibers together. The result showed that the optimum percentage of stone powder and Polypropylene fibers which provided the best properties to soil are 3% stone powder and 0.6% Polypropylene fibers. The subgrade strength properties were improved by adding either the optimal percentage of stone dust (3%) and the optimal percentage of fibers (0.6%), or the optimal percentage of the combination to the black cotton soil.

Üstünkol and Turabi (2009) examined the utilization of waste, for example, glass powder, fly ash, phosphogypsum and glass in filling asphalt concrete as effective filler, and decided the worthy qualities for filler materials.

Mahzuz et al. (2010) studied the effect of using stone powder in mortar and concrete instead of using sand. The fundamental concern is finding an alternative to sand. Replacing sand with stone powder is a good way to help in both solid waste reducing and waste recovery. Research investigations uncovered that the strength of concrete with adding stone powder and stone chip increased about 15% higher than the concrete made of typical sand and block chip. The strength of concrete made of stone powder and block chip increased about 10% higher than that of the concrete normal sand and stone chip concrete. The most noteworthy compressive strength of mortar found from stone powder which is 33.02 MPa, reveals that the best mortar can be set up by the stone powder. The concrete from stone powder indicates 14.76% higher compressive strength than that of the concrete made of ordinary sand. Then again, concrete from stone powder and block chip output higher compressive incentive from that of ordinary sand concrete and block chip.

Ramadas et al. (2010) did a series of experiments to see the effect of fly ash and stone powder on some characteristic of expansive soil. The different mixtures of stone powder and fly ash which utilized in the study were 10%, 20%, 30%, 40% and 50% and 10%, 15%, 20%, 25%, 30% respectively. From the outcomes, it has been seen that at ideal rates, i.e., 30% stone powder and 25% fly ash, it is discovered that the swelling of the expansive soil is nearly controlled and furthermore seen that there is a salient improvement in the strength of soil. The mix of fly ash and stone powder is more powerful than the using of stone powder alone in controlling the swelling problems. The blend of 25% fly ash and 30% stone powder is observed to be an appropriate measure to decrease the swelling and increase the strength of the tested samples.

Çimen (2010) appeared that the marble powder and pumice stone blended with a modest quantity of lime can be utilized for filling, and also marble powder and pumice alone or pumice and marble powder blended with lime can be utilized for improving clayey soils of high plasticity one.

Ali and Koranne (2011) expressed that there is a most extreme improvement in strength properties for the mix of fly ash and stone powder when they are mixed together instead of adding each of them separately.

Al-joulani (2012) did several tests to study the effect of stone powder mixing with lime on the fine soils. He added 10-30% of lime and stone powder by dry weight of soil. The conducted tests were CBR, compaction and direct shear. The outcomes showed that the addition of 30% lime and stone powder has good effects on CBR ratio and it increased the angle of friction and decreased the cohesion.

Sivrikaya et al. (2014) studied the effect of recycled waste from natural stone processing plants to stabilize the different type of clayey soil. He used three type of stone powder to see the most efficiency one .He used calcitic marble, dolomitic marble and granite powder as additives for soil stabilization. The results showed that the liquid limit and plasticity index of three type of treated samples decreased and the plastic limit increased. Furthermore the outcomes showed that all wastes of stone powder are good stabilizer for clayey soil; however, the waste of dolomitic Marble powder showed the best properties for soil stabilization because of its composition.

Sabat and Muni (2015) investigated the using of limestone dust (LSD) for the stabilization of highly expandable clay. The LSD utilized in this analysis was squashed into residue and purchased from marked. It was added to the soil in various extents 0, 3, 6, 9 and 12% of the dry unit weight of soil. As a result, at 12% content of limestone powder, there was increasing in plastic limit from 31% to 36%, decreasing in liquid limit from 60% to 42% and plasticity index from 29% to 6%. Besides, 9% is the ideal substance of LSD that gave greatest incentive in UCS, CBR, internal friction angle and cohesion. Furthermore, the creators set up that when the content of LSD increased the swelling pressure, maximum dry density and hydraulic conductivity decreased.

Table 2.1 Peat classification according to Radforth System (Radforth, 1969).

Predominant Characteristic	Category	Name
Amorphous-granular	1	Amorphous-granular peat
	2	Non-woody, fine-fibrous peat
	3	Amorphous-granular peat containing non-woody fine Fibers
	4	Amorphous-granular peat containing woody fine fibers
	5	Peat, predominantly amorphous-granular, containing non-woody fine fibers, held in a woody, fine-fibrous framework
	6	Peat, predominantly amorphous-granular containing woody fine fibers, held in a woody, coarse-fibrous framework
	7	Alternate layering of non-woody, fine-fibrous peat and amorphous-granular peat containing non-woody fine fibers
Fine-fibrous	8	Non-woody, fine fibrous peat containing a mound of coarse fibers
	9	Woody, fine fibrous peat held in a woody, coarse-fibrous framework
	10	Woody particles held in a non-woody, fine fibrous peat
	11	Woody and non-woody particles held in fine-fibrous peat
Coarse-fibrous	12	Woody, coarse-fibrous peat
	13	Coarse fibers crisscrossing fine fibrous peat
	14	Non-woody and woody fine fibrous peat held in a coarse fibrous framework
	15	Woody mesh of fibers and particles enclosing amorphous-granular peat containing fine fibers
	16	Woody, coarse-fibrous peat containing scattered woody chunks
	17	Mesh of closely applied logs and roots enclosing woody Coarse-fibrous peat with woody chunks.

Table 2.2 Peat classification (ASTM System).

ASTM Standard	Criteria	Designation
Fiber Content (D 1997)	>67 % fibers	Fibric (H1-H3)
	33%-67% fibers	Hemic (H4-H10)
	<33% fibers	Sapric (H7-H10)
Ash Content (D 2974)	<5% ash	Low Ash
	5%-15% ash	Medium Ash
	15-25% ash	High Ash
Acidity (D 2976)	pH < 4.5	Highly Acidic
	4.5 < pH < 5.5	Moderately Acidic
	5.5 < pH < 7	Slightly Acidic
	pH > 7	Basic
Absorbency (D 2980)	w > 1500%	Extremely Absorbent
	800% < w < 1500 %	Highly Absorbent
	300% < w < 800 %	Moderately Absorbent
	w < 300 %	Slightly Absorbent
Botanical Composition	If a single botanical name is to be used (sphagnum peat, etc.), it is required that at least 75% of the fiber content of that peat be derived from the designated type of plant material.	

CHAPTER 3

MATERIAL AND EXPERIMENTAL METHODOLOGY

3.1 Introduction

This section represents all laboratory tests which have been conducted to accomplish the targets of this study. All mixtures and soil samples were prepared according to the American Society of Testing and Materials (ASTM). In addition the engineering characterization and classification of used organic soil will be explained in this chapter. The stone powder is considered as additives and stabilizer for organic soil. It was chosen because the fact that we wanted to reuse it in valuable and useful way by recycling them to save the accumulation of landfills and to improve our soil. The accessibility to get stone powder is easy because they are available in huge amount and by free. A series of tests were done on soil mixtures and untreated soil samples such as fall cone test, unconfined compressive strength test, modified compaction test, swelling test, collapse test, specific gravity, organic content, fiber content, water content. The properties of organic soil were changed when SP was increased at different rates with various selected ratios that listed in Table 3.1. The reason for test strategies is to know the impact of stone powder on the properties of organic soil.

Table 3.1 Description of samples

Description of samples	Organic soil (%)	Stone powder (%)
Clean soil	100	0
Soil with 10% SP	90	10
Soil with 20% SP	80	20
Soil with 35% SP	65	35
Soil with 50% SP	50	50

3.2 Materials

In this part, the properties and attributes of the clean soil and the added substances of stone powder were considered and examined.

3.2.1 Organic Soil

The used soil to be stabilized in this study was gathered from Sakarya-Turkey. The soil has been sieved to be passed through #18. The color of obtained soil is dark brown as appeared in Figure 3.1. Table 3.2 shows some technical specifications obtained through laboratory tests conducted on soil. All tests were performed according to specific standards, Atterberg limit test was performed according to ASTM D4318-2000 standard to calculate plastic limit (PL), liquid limit (LL) and plasticity index (PI).The classification of organic soil was according to ASTM D2487-2000 and AASHTO as SP-SM and A-2-7 respectively. The particle size distribution for organic soil is shown in Figure 3.2. According to the ASTM D1557-98-2000 the modified compaction test was performed to evaluate the value of optimum water content and the maximum dry density. Unconfined compression strength test was accomplished in accordance with ASTM: D2166 – 2013. Loading rate was 1mm/min utilized by numerous scientists, for example Kumar et al (2007) and Estabragh (2012) for different blenders of fly ash, clay soil and soil–cement, respectively.

Through the scanning electron micrograph (SEM) and the energy dispersive x-ray (EDX) investigation, the metallurgical specialists give a careful assessment of material properties and provide important knowledge of information to makers. SEM examination is an amazing insightful device which utilizes a concentrated beam of electrons to generate a high images of magnification of the surface topography of sample. Figure 3.3 shows the SEM picture of organic soil. The picture shows that the organic soil particles shape is flaky and its size is 2 μm. Energy Dispersive X-Ray (EDX) is a chemical microanalysis technique utilized in synchronism with scanning electron microscopy (SEM). The EDX technique detects x-rays released from the sample during bombardment by an electron beam to characterize the elemental composition of the analyzed volume. Features or phases as small as 1 μm or less can be analyzed. The energy determined from the voltage measurement of each incident x-ray is transmitted to a computer for show and further data evaluation. For determine the chemical composition of sample, the spectrum of x-ray energy versus counts is evaluated. The information about the chemical composition of organic soil are shown Table 3.3 and Figure 3.4.

Table 3.2 Some engineering propertied of the untreated soil.

Properties of organic soil	Results
Liquid limit (LL, %)	65
Plastic limit (PL, %)	47
Plasticity index (PI, %)	18
Su (kPa)	1.67
Specific gravity ($G_{s)}$	2.24
Classification (USCS)	SP-SM
Classification (AASHTO)	A-2-7
Maximum dry density (γ_{dmax}, g/cm^3)	1.25
Maximum Unconfined compressive strength (UCS, kPa)	228

Table 3.3 Chemical composition of untreated soil (by weight %).

Element	Ca	O	Al	Mg	Mn	Si	S	K	Na	Ti	C	Fe
Row soil	0.4	51.5	9.06	1.14	0.03	19.1	0.3	1.1	0.2	0.28	14.0	2.5

Figure 3.1 The soil used during the experimental study.

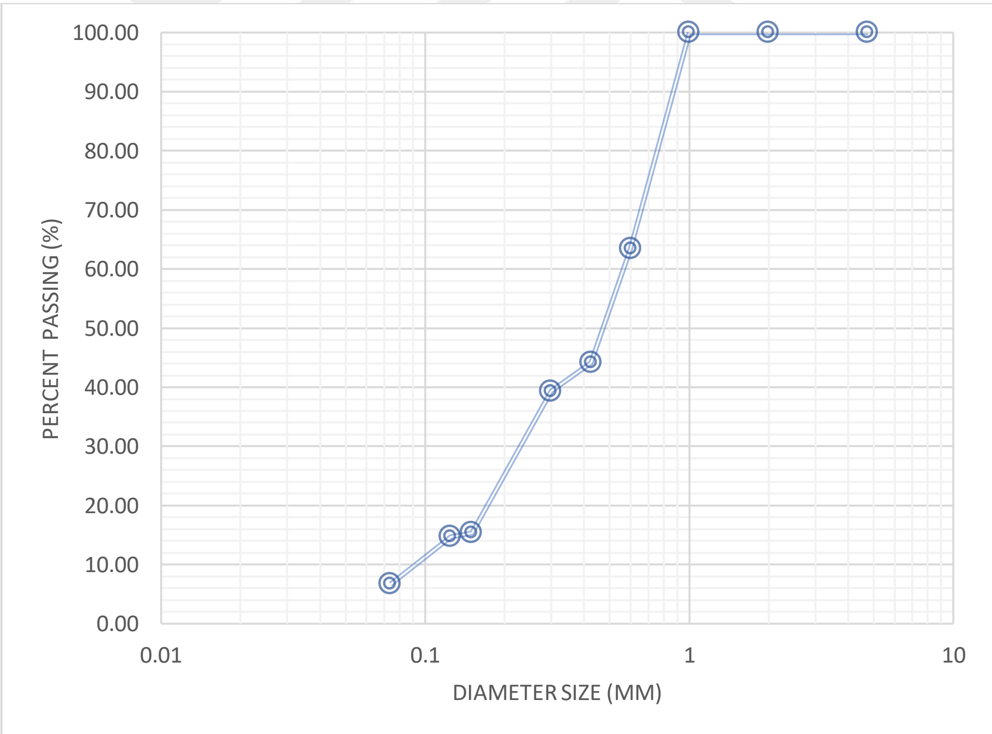

Figure 3.2 Particle size distribution of organic soil**.**

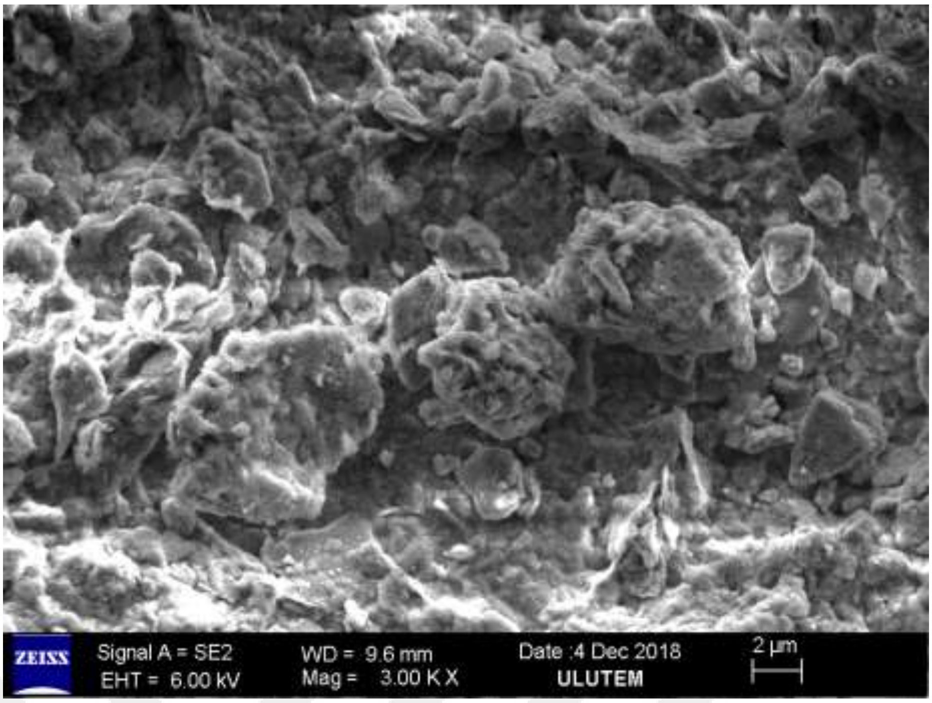

Figure 3.3 Scanning electron micrograph picture of the soil.

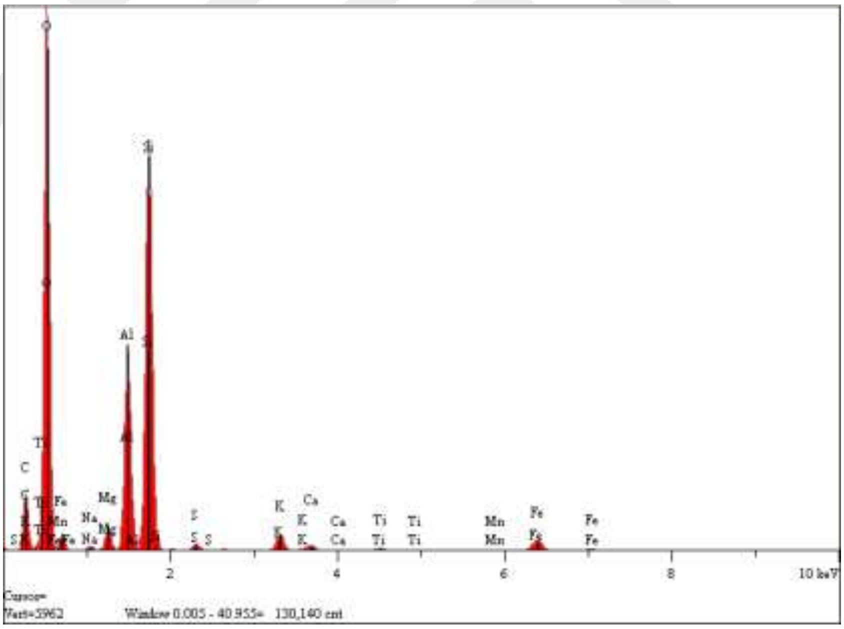

Figure 3.4 EDX analysis of the organic soil.

3.2.2 Stone Powder (SP)

The used waste material in this study was stone powder which is a fine white powder see Figure 3.5 this waste powder was obtained from aggregate and crushed stone from Gaziantep municipality, Turkey. What's more, the stone powder is inorganic and dangerous waste material that it tends to be utilized as an admixture to alter the building properties of natural soil and it is exceptionally affordable in expense. The image of microstructure of stone powder and the chemical composition of it are shown in Figure 3.6, 3.7, and Table 3.4 respectively. In addition the sieve analysis test was conducted to obtain the particle size distribution curves for organic soil with different percentages of stone powder and to see the effect of adding this waste material on soil classification. See Figures 3.8, 3.9, 3.10, 3.11and 3.12. In Table 3.5 the different in soil classification with different percentages of stone powder is shown.

Table 3.4 Chemical composition of SP (by weight %).

Elements	Fe	Al	Si	Mg	Ca	O	C
SP	0.34	0.32	0.68	0.27	38.32	38.32	9.47

Table 3.5 Soil classification for organic soil and after adding SP (by weight %).

Stone powder %	Classification (USCS)	Classification (AASHTO)
0	SP-SM	A-2-7
10	SM	A-2-7
20	SM	A-2-7
35	SM	A-2-7
50	SM	A-2-7

Figure 3.5 The used white stone powder.

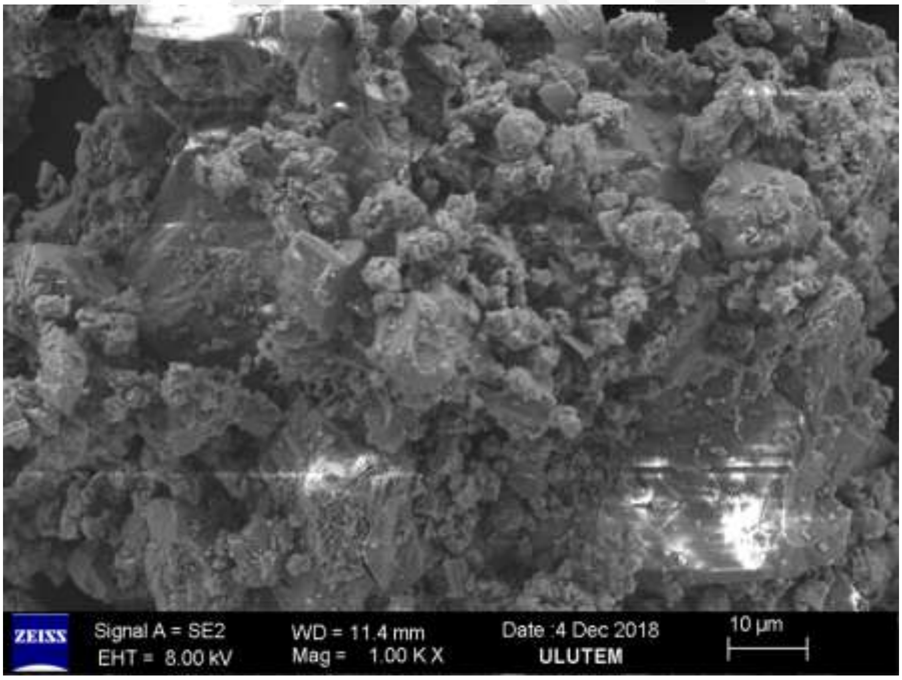

Figure 3.6 SEM picture of SP, soil particles size is 10 µm.

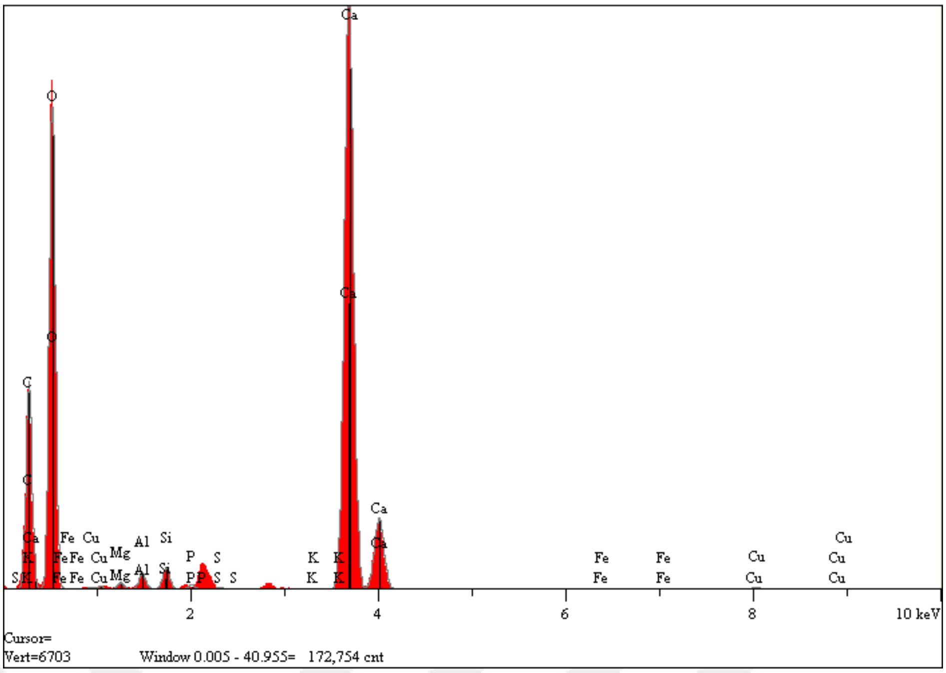

Figure 3.7 EDX analysis of the stone powder (SP).

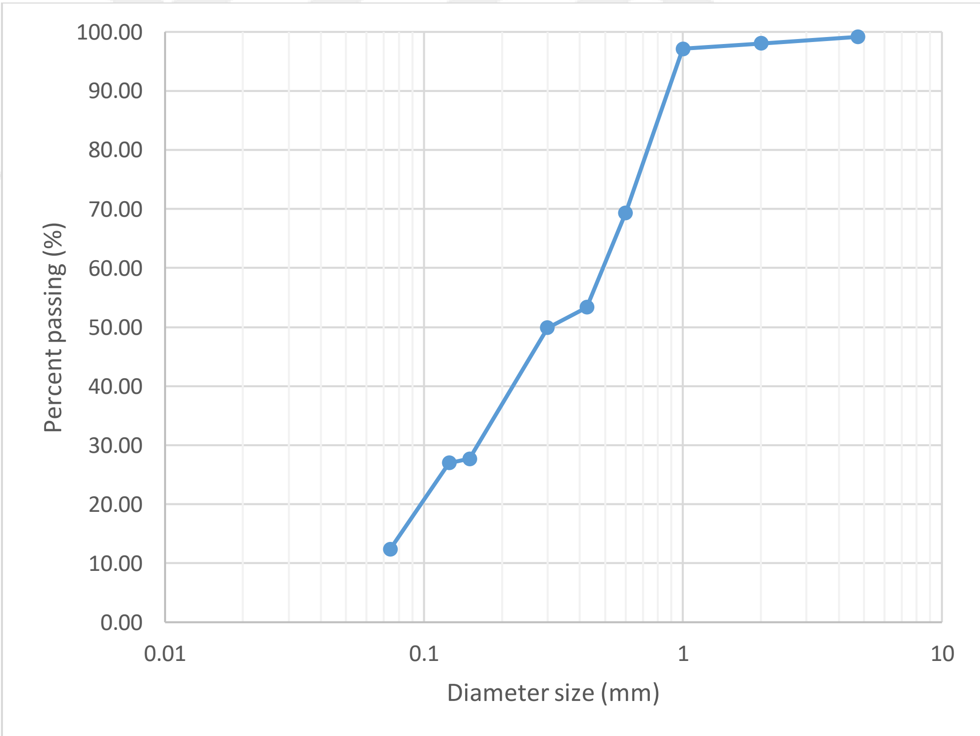

Figure 3.8 Particle size distribution of organic soil with 10% SP.

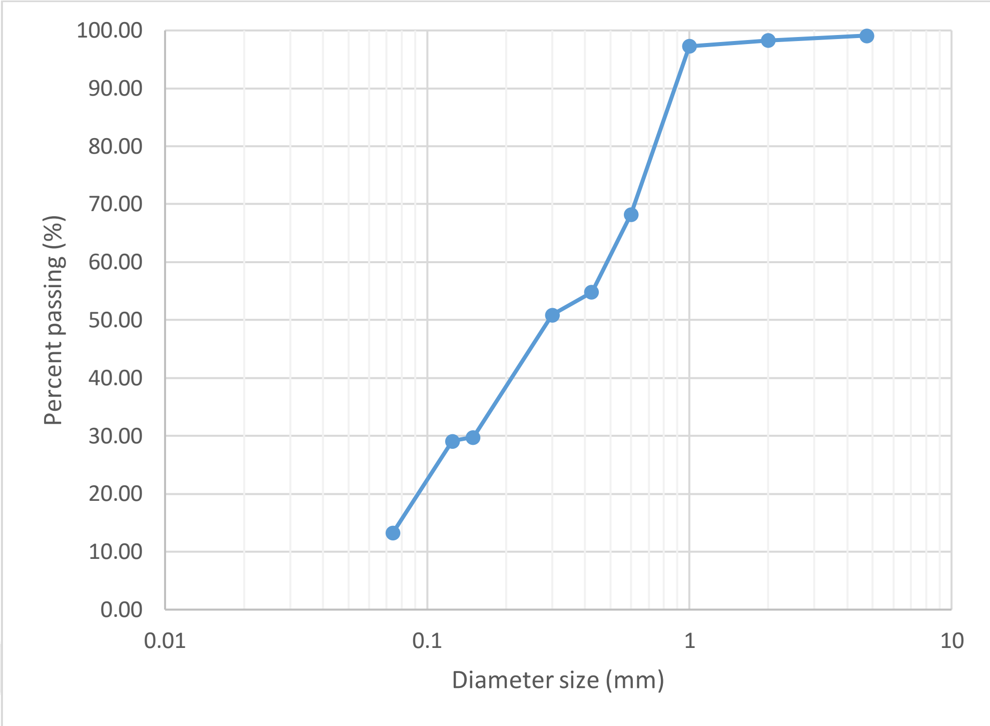

Figure 3.9 Particle size distribution of organic soil with 20% SP**.**

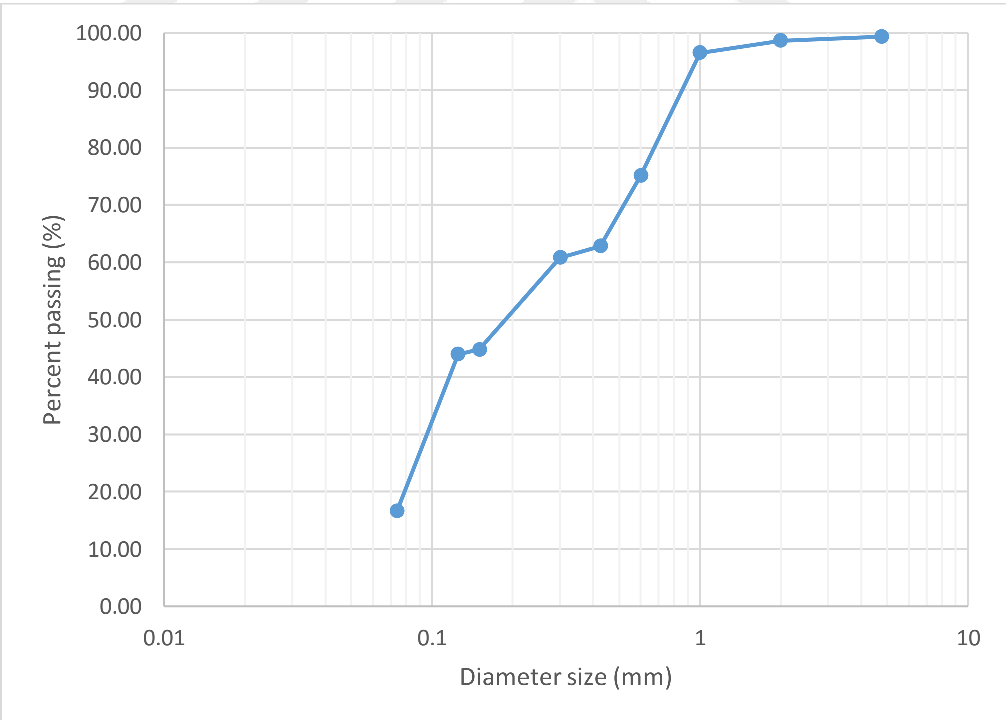

Figure 3.10 Particle size distribution of organic soil with 35% SP**.**

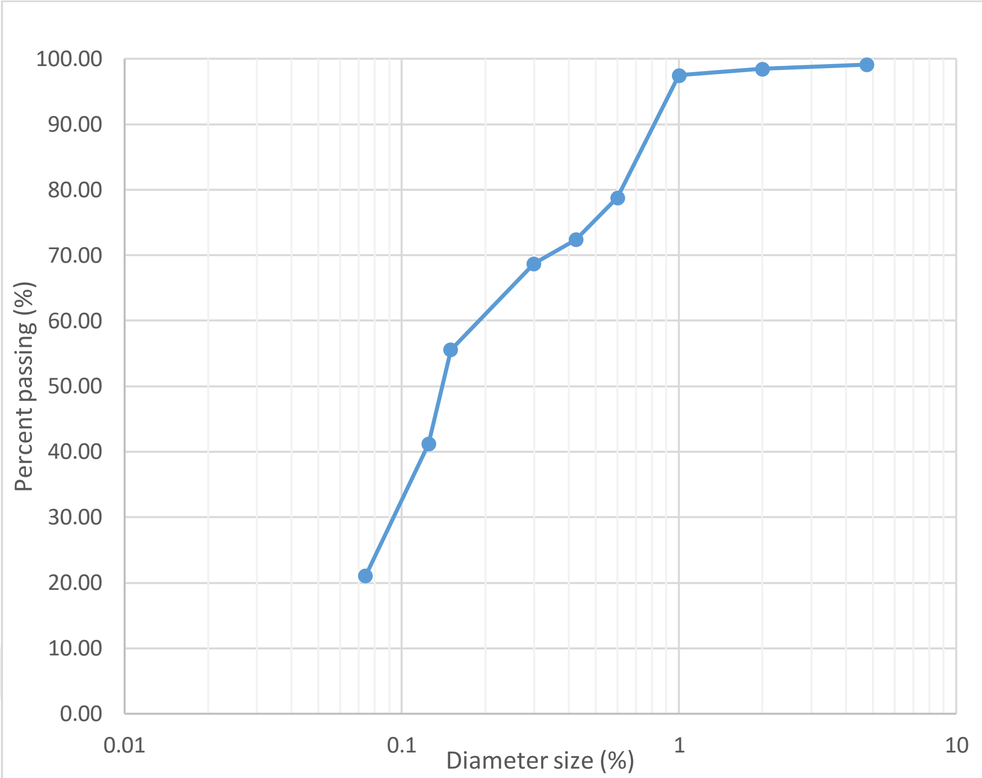

Figure 3.11 Particle size distribution of organic soil with 50% SP.

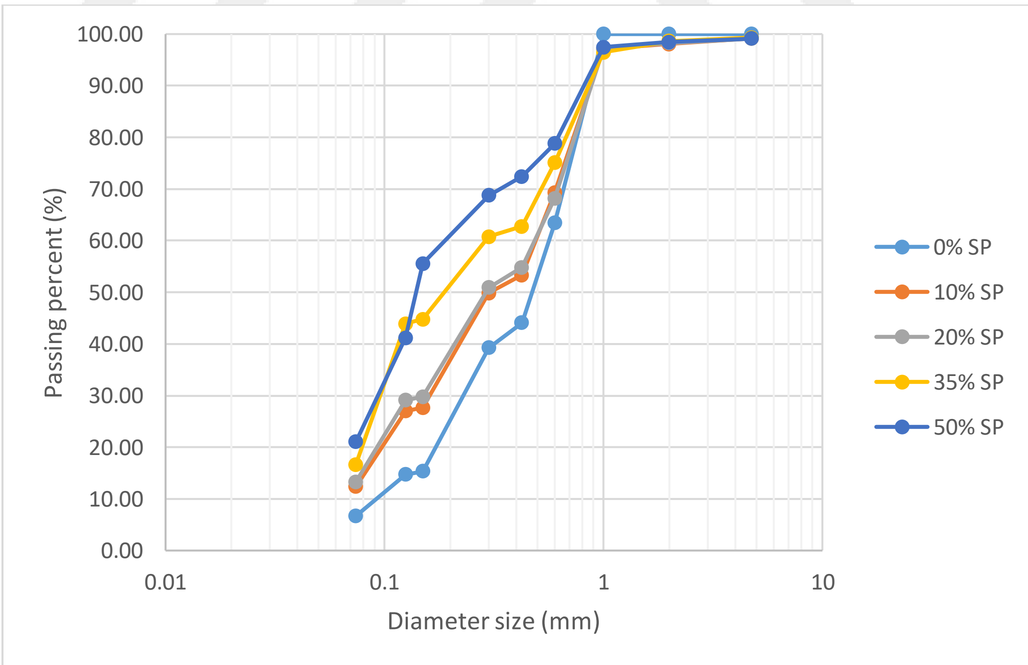

Figure 3.12 Particle size distribution of organic soil with different SP percentages.

3.3 Sample Preparation

The material was sieved with accordance to the specifications of all tests requirements, also the particle size was constrained as each test had a particular range for the size of particles. The organic soil and stone powder were placed in an oven prior to use in experiments, as all materials have to be dry prior to use in experiments. The degree of oven drying was 60 c for organic soil and it lasted for two days at oven because the organic content is exposed to be burned if we put it in higher degree. The preparation of mixtures was by blending stone powder with organic soil that passed through sieve # 18 (1 mm) at different percentages; 10%, 20%, 35% and 50% by dry weight of stone powder. All test samples were blended with five values of water contents to get the value of optimum water content and maximum dry density. Notwithstanding the measurements and the used molds of each test that completed during the laboratory experiments, every one of each sample arranged to be smooth and at flat level in surface and in perpendicular to the length of sample.

3.4 Experimental Methodology

3.4.1 Water Content

The measuring of water content of the studied soil was according to (British Standard (BS1377, Test 1(A)), temperature limitation should be taken into account to avoid the oxidation of organic substance. So the used temperature was 60° C for peat of organic soil. The mass of soil before putting in oven and after oven was obtained and then the water content of soil sample has obtained which was equal to 92%.

3.4.2 Organic Content

The test was conducted on organic soil. In order to calculate (OC) for this soil, the measurement used the technology of ignition in a muffle furnace at 440° C up to 4 hours, in according to ASTM D 2974-14 see Figure 3. 13 In the test, the oven-dried sample of tested soil was kept at a temperature of 440 ° C to ensure that the mass of the specimen did not change with additional heating. The residual 'ash' mineral content of the soil, where the mass of waste constitutes the organic matter (Figure 3.14). Organic content can be defined as a percentage of the original oven dry mass. 20% was the organic content of the used sample.

Figure 3.13 Muffle furnace with organic soil sample (Ibrahim, 2018).

Figure 3.14 Organic soil samples (1) before oven (2) after oven (Ibrahim, 2018).

3.4.3 Specific Gravity

Laboratory experiments were performed on soil (organic soil) that was tested to obtain precise value for specific gravity According to ASTM D 854. The performed test was by using distilled water. To remove trapped air from soil sample the heat was utilized during the test. The average getting value of G_s was 2.24.

3.4.4 Fall Cone Test

The utilization of cone penetrometers laboratory test to calculate the liquid limit of soil has been discussed in several researches (e.g. Terzaghi, 1927; Karlsson, 1961; Sowers et al, 1959; Sherwood & Riley, 1970; Garneau & Le Bihan, 1977; Littleton & Farmilo, 1977; Wroth & Wood, 1978; Wood, 1982; Houlsby, 1982; Whyte, 1983; Wasti & Bezirci, 1986; Wood, 1985; Farrell et al., 1997; Leroueil & Le Bihan, 1996). The Swedish Standard (SS 027120:1990), The British Standard (BS 1377:pt2: 1990) and the Canadian Standard (CAN/BNQ 2501-092-M-86) have studied the obtaining of liquid limit by using fall cone test. However, there are just few researches and studies (e.g. Wood & Wroth, 1978; Harison, 1988; Belviso et al., 1985) that concentrated on the using of fall cone test to determine plastic limit because at this stage the samples of soil are very stiff and hard to blend (Wasti and Bezirci, 1986; Whyte, 1983; Stone and Phan, 1996; Wood and Wroth, 1978; Harison, 1988). For 30 of British cone the depth of cone penetration corresponding to LL is 20 mm. The studies of Skempton and Northey (1953) showed that the value of undrained shear strength at plastic limit value is near 100 times of its value at liquid limit, so from undrained shear strength equation 3.1 it could be observed that the penetration of cone is 2 mm at plastic limit.

The fall cone test was performed according to BS1377 standard. ELLE type of fall cone test devices has been used in this experiment. The fall cone was manufactured by Wykeham France with angle of a 30° and its weight is 0.785 N. In addition there are another elements of fall cone apparatus including the cup of specimen which is 40 mm in height and its diameter is 55 mm. Figure 3.15 and Figure 3.16. This test was performed to determine the value of plastic limit (PL), liquid limit (LL), plasticity index (PI) and undrained shear strength for the clean soil and stabilized soil. A total of 60 samples have been used to calculate the mentioned values by using several ratios of SP with organic soil. About 100 gr of dry mass of organic soil was mixed with stone powder in different ratios (0%, 10%, 20%, 35% and 50%). The first mixture of all test ratios was starting without adding any water to study the soil samples in unsaturation and saturation status. The mixing was by using plastic bag trying to make the mixture more homogeneous. Then the mixture was putting into fall cone cup, taking into account the evacuation the voids from trapped air by compressing the cup which contains the soil on flat surface. After that, the sample surface was smoothed and adjusted to be flat. Then the first step was by manually moving the cone until it touched the center of sample surface. The cone was released by penetrating the sample by holding the bottom for 5 seconds. Then the penetration of cone was recorded by using the dial gauge which was connected to fall cone device. At last, a small amount was used and taken from each tested sample and waiting for 24 hours for drying the sample to determine the water content. The same steps have been conducted near to ten times with different values of water content for each sample.

After conducting fall cone test on the all used ratios of SP at different water contents values, the chart between the penetration of cone and water content has been plotted. After that according to the mentioned studies, the liquid limit and plastic limit are accepted to be at penetration values 20 mm and 2 mm respectively.

The calculating of undrained shear strength was by using equation 3.1 and the plot between undrained shear strength and water content has been plotted also.

$$s_u = k \frac{W}{d^2} \tag{3.1}$$

where;

s_u = the undrained shear strength (kPa);

k = constant depend on the angle of cone (0.85 for 30º angle) (Wood 1985);

w= Cone weight;

d = the depth of penetration (mm).

Figure 3.15 The fall cone apparatus during the test.

Figure 3.16 The stainless steel cone.

3.4.5 Compaction (Modified Proctor) Test

The definition of compaction test is repositioning the particles of soil to reduce the internal voids which are filled with air by applying external energy on the tested soil. In this test we used modified proctor test according to ASTM D1557-2000 standards, as shown in Figure 3.17. This test was conducted on the all mentioned ratios of SP and untreated soil to get the values of maximum dry density and optimum water content. About 9 samples for each mixture and ratio of SP were subjected to modified proctor test by using different ratios of water content until getting the wanted plot shape of optimum water content with the dry density of each mixture. Firstly, the dry weight of organic soil was blended with the various ratios of stone powder (0%, 10%, 20%, 35% and 50%) in a big pan to ensure a good mixing of the soil with the additive stone powder percentage then the water was added to each mixture starting by small amount just to ensure the blending until getting a uniform mixture in texture and color. Then the weight of mixed soil with the mold and its base was recorded by using electronic balance. The blend was placed into the mold at five equivalent layers, for each layer the number of blows was 31 by using a specific proctor hammer designed for modified proctor test with a rate not more than 1.5 seconds for each blow, The using of hammer was by guaranteed that this hammer was able to cover all surface of soil sample inside the mold. in addition to that the thickness of last layer should be higher than the top of mold and should be extended to the collar. In the wake of completing the five layers compacting, the collar removed carefully from the mold and the extra soil layer was cutted off by spatula. Finally, the weight of mold with the soil mixture inside it was recorded. For calculating the water content of each mixture a three small amount from the bottom, middle and top of the compacted sample was taken and was placed in the oven for 24 hours.

The used equation for calculating dry density is:

$$\gamma_d = \frac{\gamma_m}{1+(\frac{w_c}{100})} \tag{3.2}$$

where;

γ_d dry density (gm/cm^3);

γ_m is moist density (gm/cm^3);

w_c is water content (%).

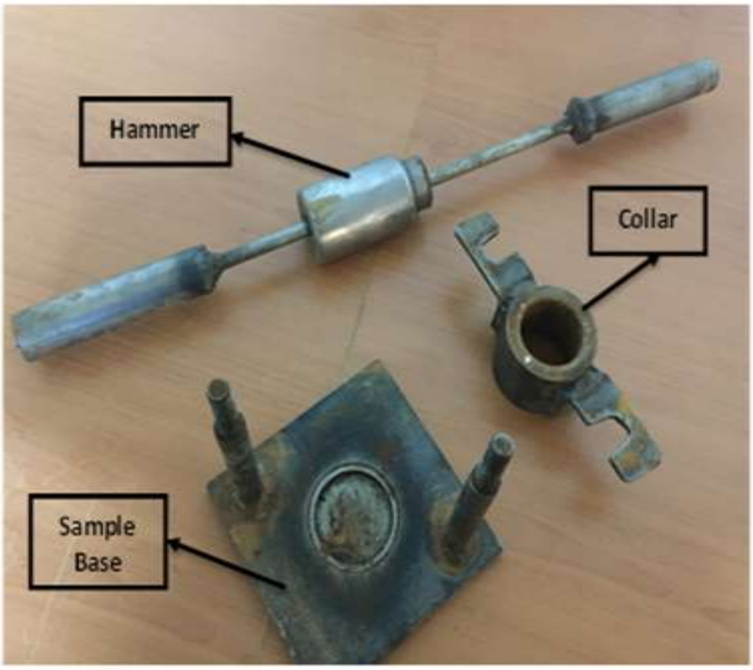

Figure 3.17 Compaction apparatus utilized through test.

3.4.6 Unconfined Compressive Strength (UCS) Test

The Unconfined compressive strength test was performed on the clean soil and all mixtures of the soil with different ratios of SP. This test is generally conducted to see the relationship of stress-strain of organic soil and SP mixtures. The tested samples for unconfined compressive strength arranged according to ASTM D2166-2000 in a similar method that used in compaction test, the acquired values of optimum water content of each mixture from compaction test were adopted to apply unconfined compressive strength test. The same mold of compaction test was used to UCS test, this mold is a cylinder, and its dimensions are 100 mm in height and 50 mm in diameter. Each sample compacted for 5 layers then the sample was expelled from the mold and substituted in the unconfined compressive strength device see Figure 3.18. The soil sample was engaged into the device and then an axial load was applied by 0.5 to 2% per minute axial rate of strain. The gauges of stress and strain were at zero reading, after that the compressor of the machine was turned on then the reading were recorded at 0.20 mm of deformation interval. When the sample collapses and reaches its maximum deformation the device was stopped. At the end of getting all wanted values the plot of unconfined compressive strength against strain was plotted; where the calculation of strain is by the equation.

$$\varepsilon = \frac{\Delta l}{L_o} \tag{3.3}$$

where;

ε Axial strain;

ΔL Deformation in length (mm);

L_o initial length of sample (mm).

The calculation of unconfined compressive strength was by the equation.

$$q_u = \frac{P}{A} \tag{3.4}$$

where;

q_u is the unconfined compressive strength (kPa);

P is the axial load (kN);

A is the cross-sectional area of the sample which could be calculated by equation.

$$A = \frac{A_o}{1 - \varepsilon} \tag{3.5}$$

where; A_o is the initial area before deformation in mm^2

Figure 3.18 Unconfined compressive strength machine.

3.4.7 Consolidation Test and Apparatus

Preparing the samples for consolidation test was by firstly sieve the sample through sieve 18 (1 mm in diameter) and then the stone powder ratios were added to organic soil and from compaction test results the obtained optimum water content was added to soil mixture. After that the prepared samples were compacted by CBR machine applying a constant rate equals to (0.02mm/sec or 0.05inch/min). This method and its following procedures were approved according to ASTM D2435– 2011 codes. The test samples were prepared by using specimen which is a ring its dimensions are 75mm in diameter and 20 mm in the thickness Figure 3.19. The beginning of the test was by preparing the soil samples of organic soil and stone powder mixtures to the test by placing this samples in the mold set of consolidation test, the samples were covered from bottom and top by two filter papers and then the mold was ready to be placed into consolidation device Figure 3.20. The water were added to soil specimen and of course with the keep of monitoring to dial gauge reading , the load were added to each time the gauge was moving and several days were waited until finishing the swelling of soil specimens. After that the start of consolidation test was by adding the load gradually starting from 10 kPa and then the load was doubled until applying the maximum load value of 800 kPa. At each stage of load adding there was a determined interval time until the ending of consolidation test or after 24 hours from the beginning of test of adding the first load. The values of added stresses were 10 kN/m^2, 20 kN/m^2, 50 kN/m^2, 100 kN/m^2, 200 kN/m^2, 400 kN/m^2 and 800 kN/m^2 during the loading and unloading stage. The unloading case is the second part of the test where the load were taken inversely starting from the maximum load until 100 kN and again after 24 hours the recording of dial gauge reading were recorded.

From the first stage we can calculate compressibility index value's according to the equation;

$$c_c = \frac{\Delta e}{\Delta \log p} \tag{3.6}$$

where;

c_c is the compressibility index;

Δe is the void ratio difference and

P is the applied stress.

From the second stage of test which is unloading case the swelling index c_s was obtained by the equation and it was represented by the rebound part of e-log p curve plot.

$$c_s = \frac{\Delta e}{\Delta \log p} \tag{3.7}$$

Figure 3.19 Apparatus of one-dimensional consolidation test.

Figure 3.20 Consolidation test device used during experiment.

CHAPTER 4

EXPERIMENT RESULTS AND DISCUSSION

4.1 Introduction

This study deals with the using of SP to investigate its effects on organic soil. In spite of there are numerous examinations about the best possible treatment of SP in the pavement applications, there are little-realized endeavors to investigate the impacts of this material on the organic soil. This chapter manages the investigation and discussion of the information acquired during the laboratory experiments.

4.2 Fall Cone Test

Fall cone test had been conducted to study the effect of SP on liquid limit (LL), plastic limit (PL), plasticity index (PI) and undrained shear strength of clean soil and treated one with different SP percentages. As mentioned before the test were conducted on saturated and unsaturated samples. (Wood and Wroth 1978; Whyte 1983; Wasti and Bezirci 1986; Harison 1988; Stone and Phan 1996; Feng 2000) said that the conducting of fall cone test at water content near to plastic limit is very hard to be conducted because the mixing of samples is difficult and the samples at that state are so stiff. Obtaining of liquid limit and plastic limit values are at the cone penetration 20 mm and 2 mm respectively and the plasticity index is the difference between them. Figure 4.1 shows the relation between water content and the penetration of cone at saturated and unsaturated states. The indication of results for the liquid limit, plastic limit and plasticity index calculations in saturation part showed that the obtained values of them decreased with the increasing of stone powder percentage. Furthermore, the (LL) decreased from 65% to 43% by the increasing of stone powder percentage up to 50%. The plastic limit decreased from 47% to 31% up to 50% of stone powder and also the plasticity index decreased from 18% to 12% up to 50% of stone powder. From Table 4.1 and Figures 4.2, 4.3, 4.4 and 4.5 we can

see the variation of obtained penetration with water content and the variation of stone powder with liquid limit (LL), Plastic limit (PL) and plasticity index (PI) at saturated state. One appropriate clarification for the reducing of plasticity index is that the more adding of SP to organic soil is like the replacing of plastic materials in organic soil with non-plastic materials from stone powder. From Casagrande chart we can see the changing of soil classification under line A from high plasticity to low plasticity Figure 4.6. Kalkan (2006) had studied the using of red mud waste in clayey soil stabilization, the result showed that the red mud waste has changed the soil group from high-plasticity soil group (CH) to low-plasticity soil group (MH). In addition (Ene and Okagbue,(2009)) Obtained the same results when they used pyroclastic dust for stabilizing expansive soil. They explained the reason of this reduction in LL, PL and PI because of that the pyroclastic powder particles cover and coat the clay, bonding them with each other, so the voids and the water constrained inside the void decreases. So the decreasing of the plasticity of organic soil with adding more stone powder improve the workability of soil and the soil become more friable.

Undrained shear strength is the shear strength of the soil when the excess pore pressures are not permitted to dissipate any more. The calculations for undrained shear strength were by using the equation 3.1 where the penetration of the cone by d distance in mm represents the undrained shear strength value. Feng, T. (2000) applied his study using the same equation. The obtained results about the relationship between undrained shear strength and water content at saturated state are shown in Figure 4.7, we can see that the undrained shear strength has increased with adding stone powder but the most increasing was at the using of 10% of stone powder. Cabalar and Mustafa (2015) studied also the impact of clay-sand mixtures on the undrained shear strength and liquid limit to see the effects of sand on changing some geotechnical properties of clayey soil.

Table 4.1 index properties of organic soil and treated one.

SP (%)	0	10	20	35	50
Liquid limit (%)	65	60	55	48	43
Plastic limit (%)	47	45	41	35	31
Plasticity index (%)	18	15	14	13	12

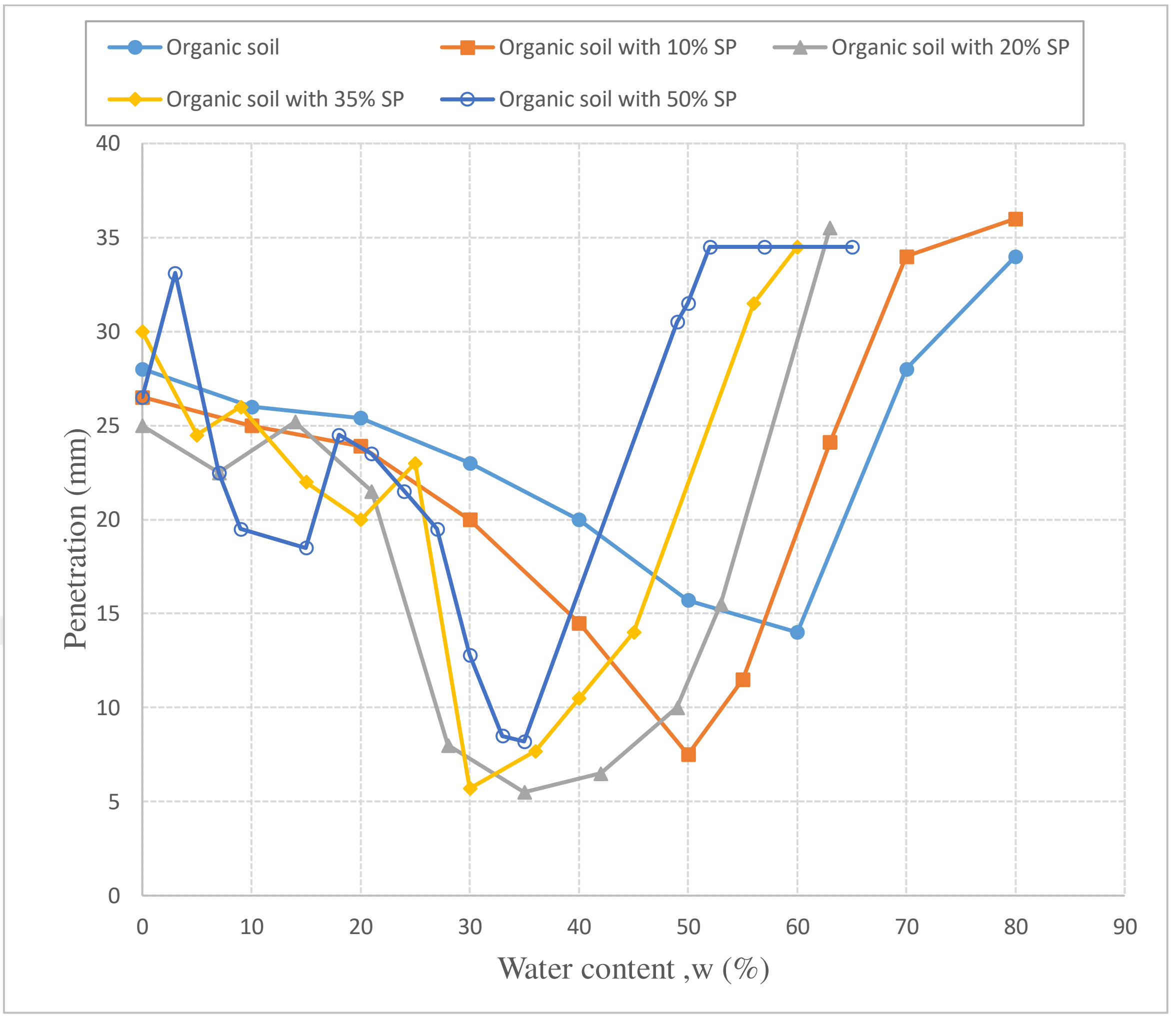

Figure 4.1 Variation of cone penetration with water content.

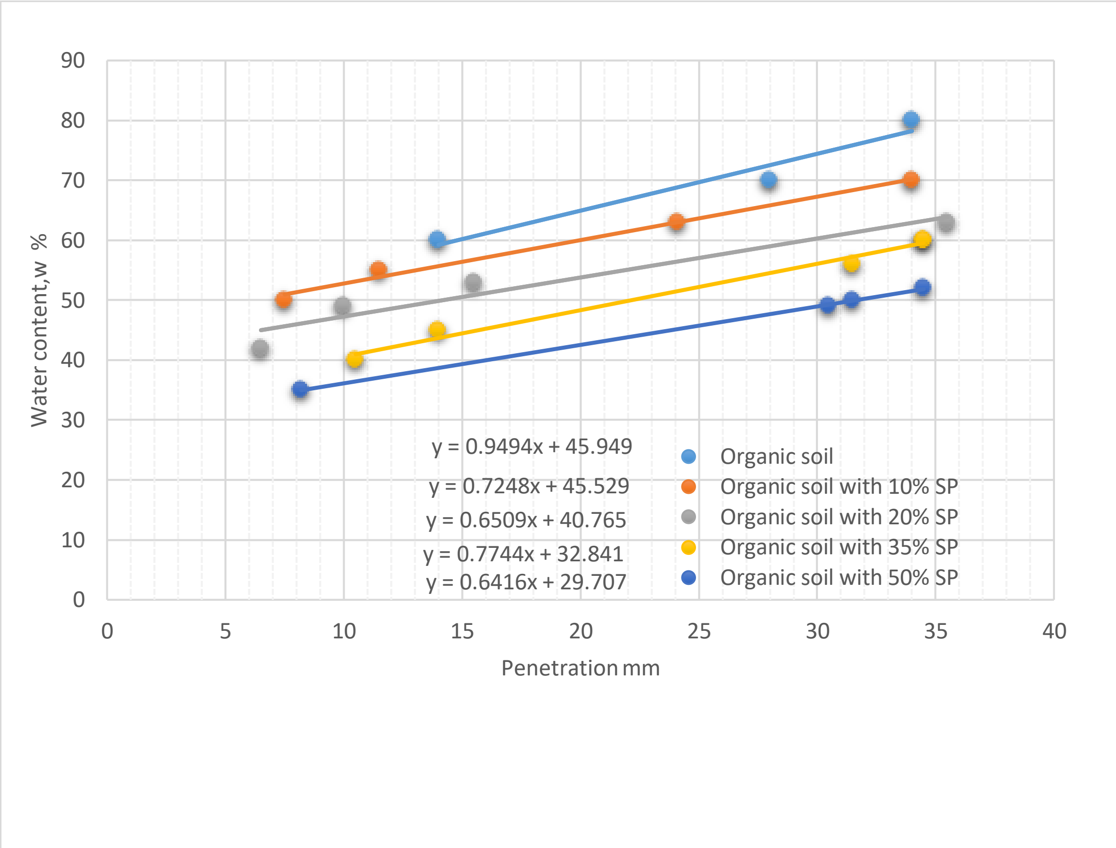

Figure 4.2 Relationship between the penetration and water content at saturated state.

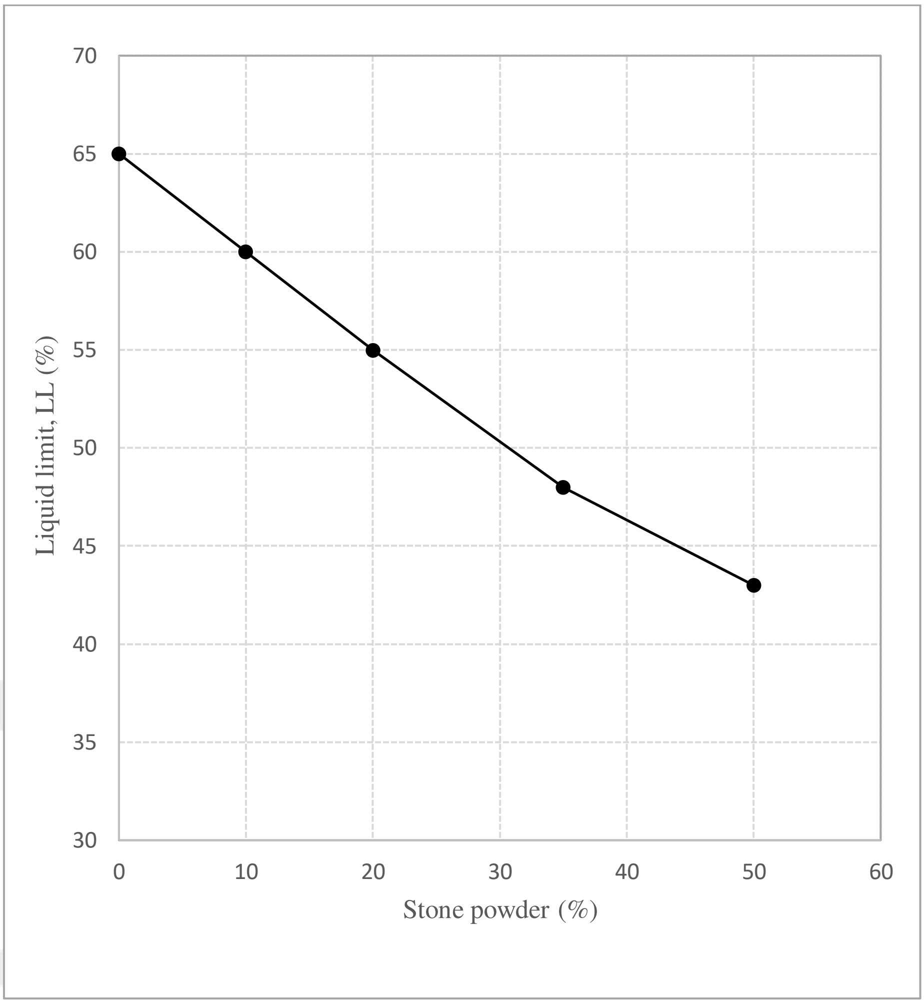

Figure 4.3 Variation of liquid limits with adding more SP percentage.

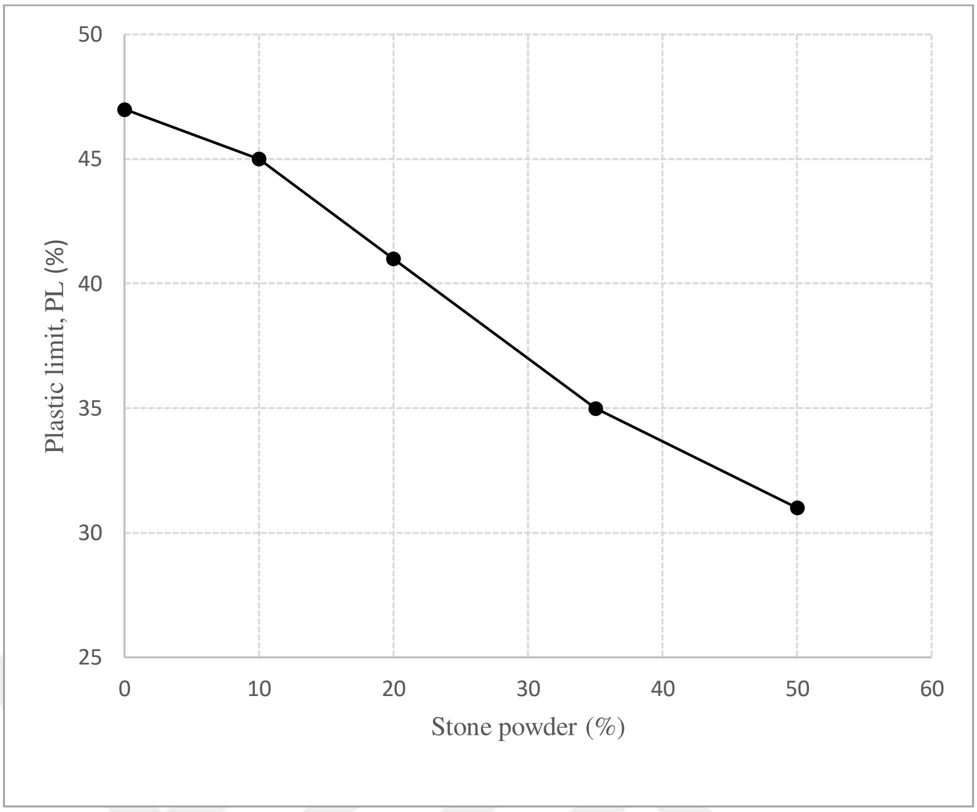

Figure 4.4 Variation of plastic limits with adding more SP percentage.

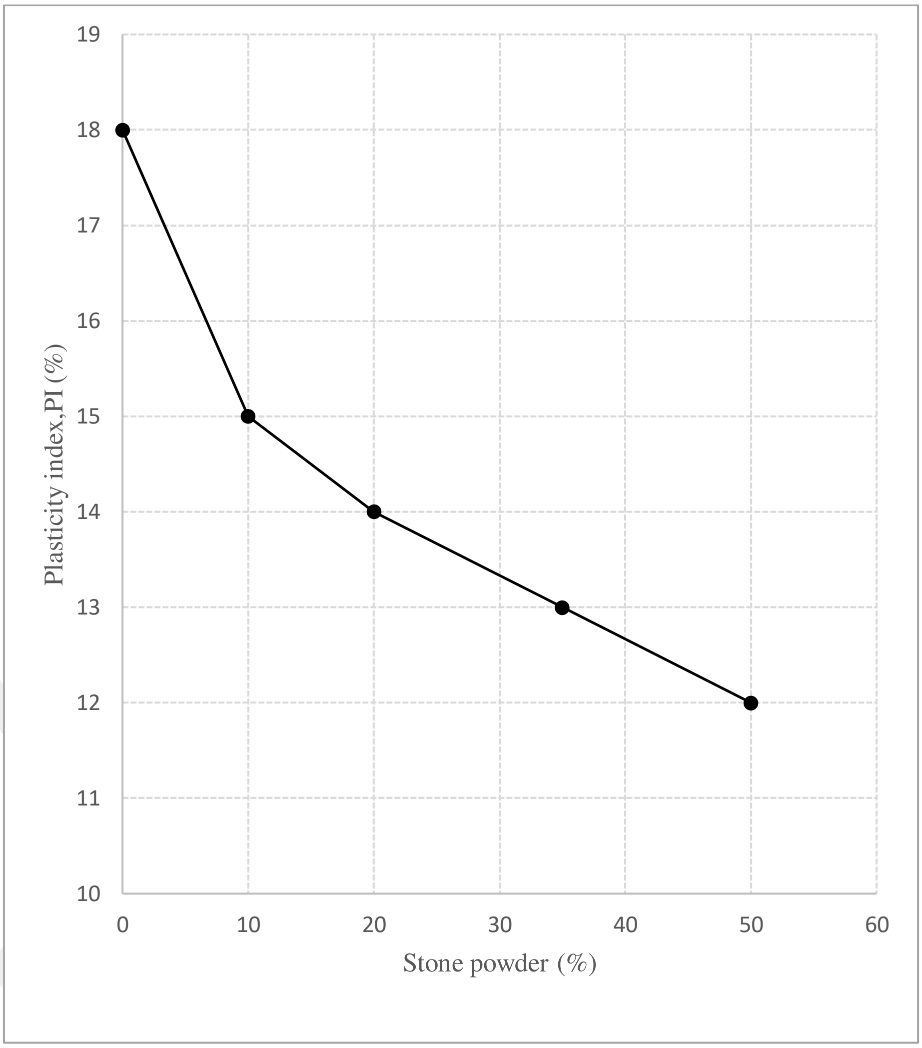

Figure 4.5 Variation of plasticity indexes with adding more SP percentage.

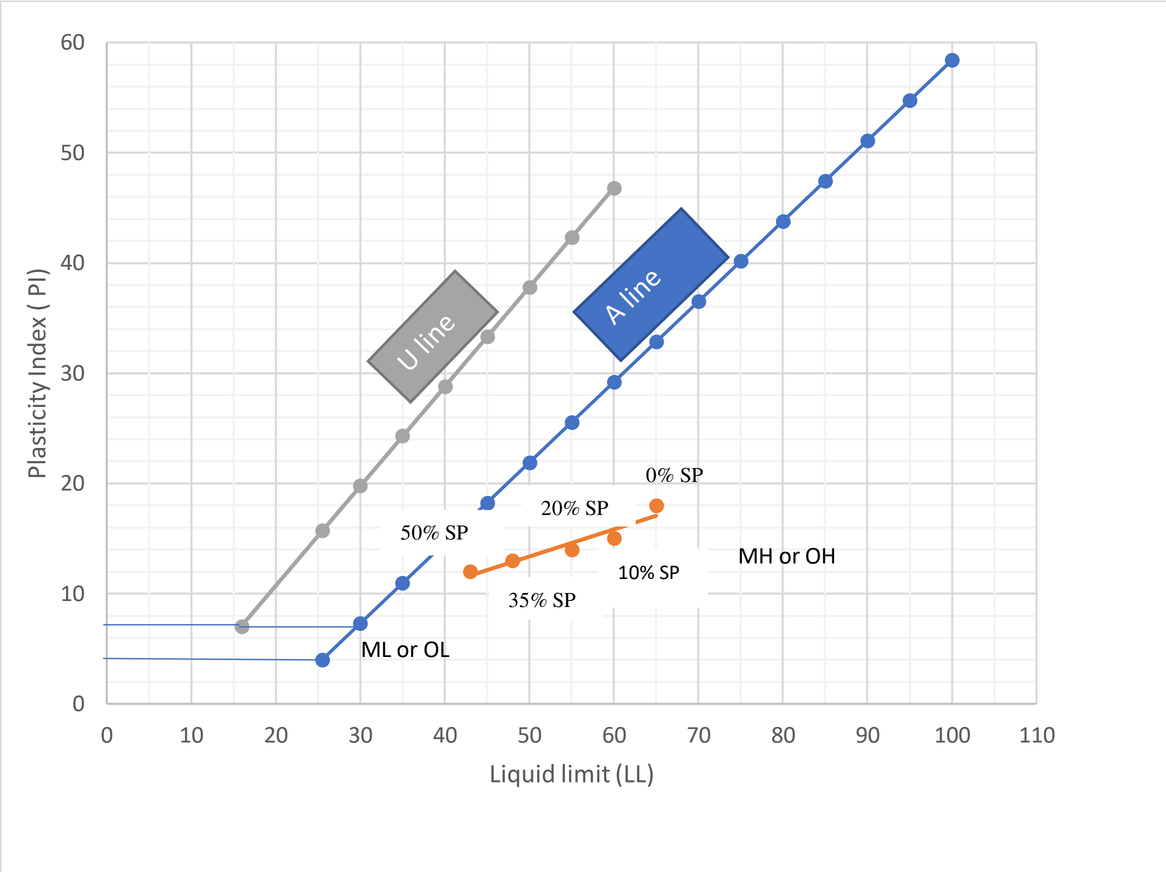

Figure 4.6 Plasticity chart according to studied soil.

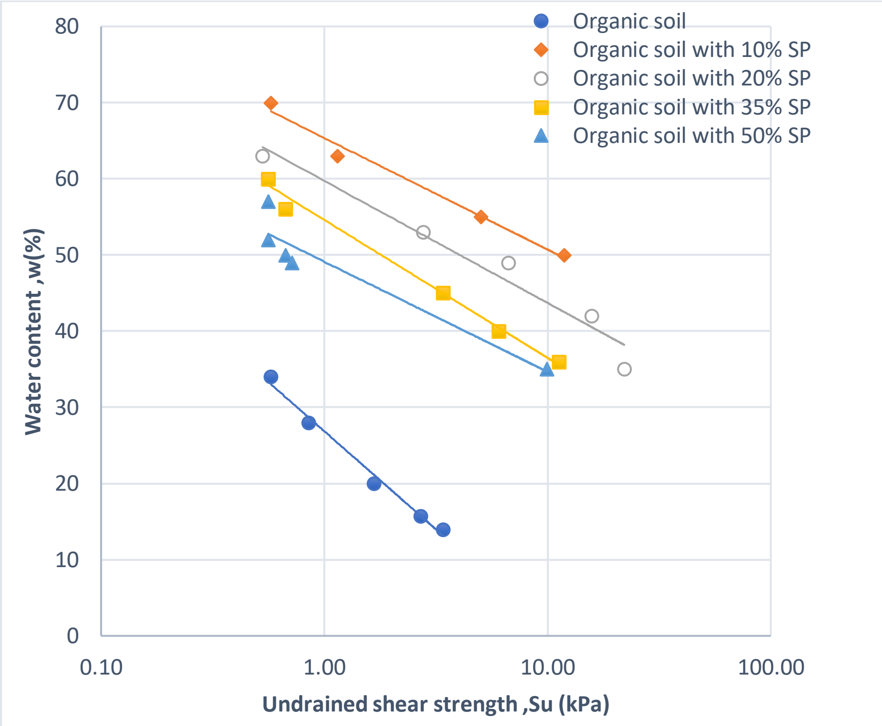

Figure 4.7 Relation of undrained shear strength and water content at saturated state.

4.3 Modified Proctor Compaction Test

Modified compaction test was conducted to investigate the variation of dry density and optimum water content with adding different percentages of stone powder and without stone powder. According to ASTM D1557 standard all compaction curves of all mixtures were obtained. The peak values of maximum dry density and optimum water content are shown in Table 4.2. Figure 4.8 shows the compaction behavior of organic soil at different ratios of stone powder, from the figure we can see that there is decreasing in the optimum water content from 30% for untreated soil to 15% for the maximum ratio of stone powder and for the maximum dry density the results showed that there was increasing in its value from 12.23 kN/m^3 for untreated soil to 15.26 kN/m^3 for 50% of stone powder. The analysis for optimum water content reducing are back to that the stone powder does not absorb a lot of water like organic soil and with adding more stone powder the replacement between organic plastic soil and non-plastic stone powder particles happens so the need for more water decreases. Another reason is due to that the surface of stone powder particles is rough and waste stone powder considered as heavy material. Chetia and Sridharan (2016) studied the effect of using quarry dust on compaction test results. The results showed similar result to our study, the optimum water content decreased by adding more amount of quarry dust and the maximum value was at 50% of dust. Phani Kumar and Sharma (2004) had used the fly ash to apply several laboratory test on using it in soil stabilization, the results indicated that at the using of 20% of fly ash the MDD have increased and the OMC have decreased. Al-Joulani (2012) investigated the effect of lime and stone powder on the fine soil stabilization. They said that the 30% of stone powder slightly reduced the values of MDD and OMC. While the using of 30% lime reduced the OMC and MDD by 13.5% and 19%, respectively. The reason for MDD increasing is that the specific gravity of stone powder is more than the specific gravity of organic soil and with adding more SP the void percentage decreases SP it causes increase in MDD value up to 50% stone powder. Figures 4.9, 4.10 and 4.11 show the variation of MDD and OMC with SP ratios.

Table 4.2 Variations of OMC and MDD with stone powder ratios.

Stone powder (%)	OMC (%)	MDD (kN/m^3)
0	30	12.23
10	25	13.17
20	22	13.63
35	16	14.67
50	15	15.26

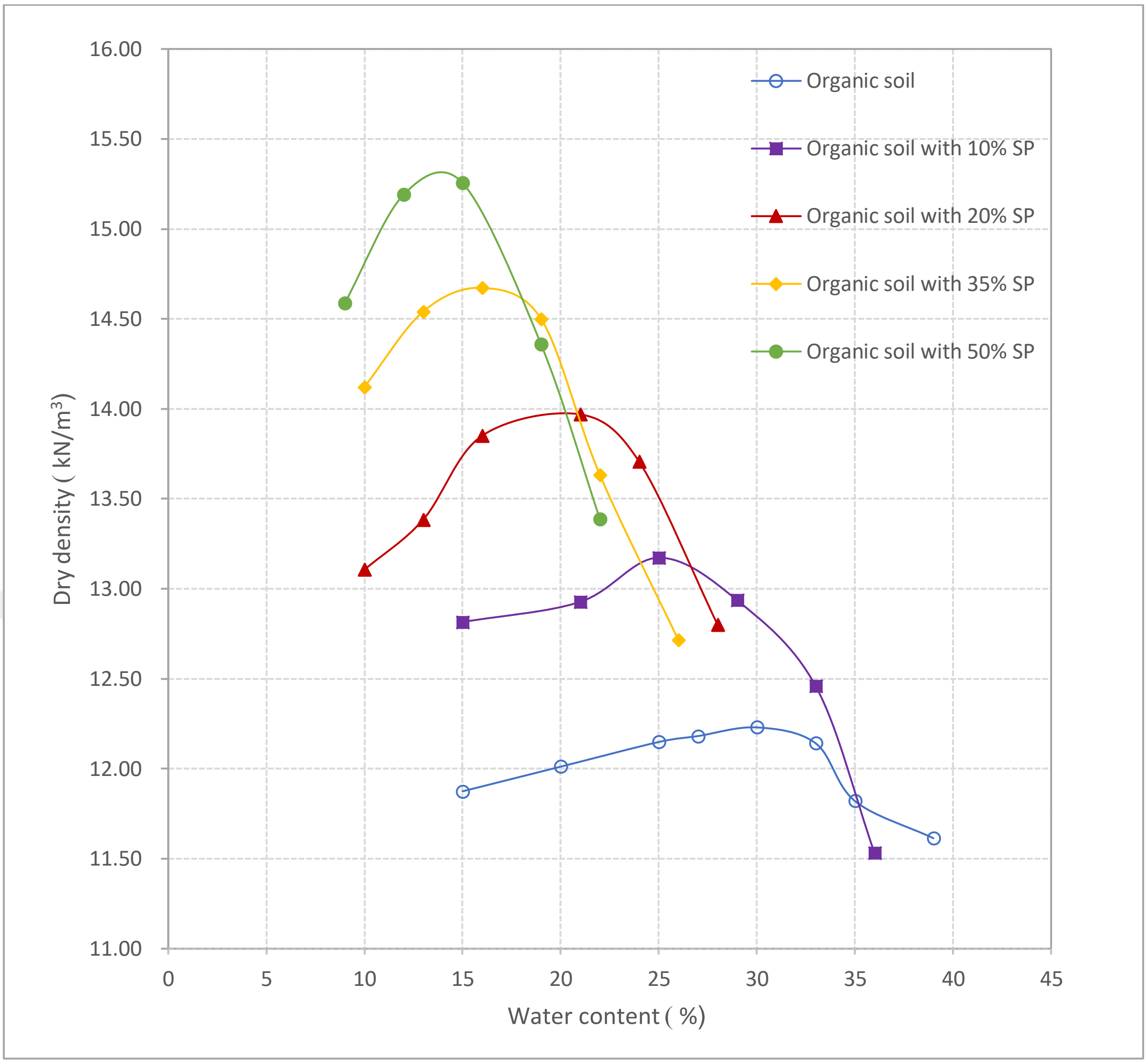

Figure 4.8 The effect of SP on MDD and OMC values.

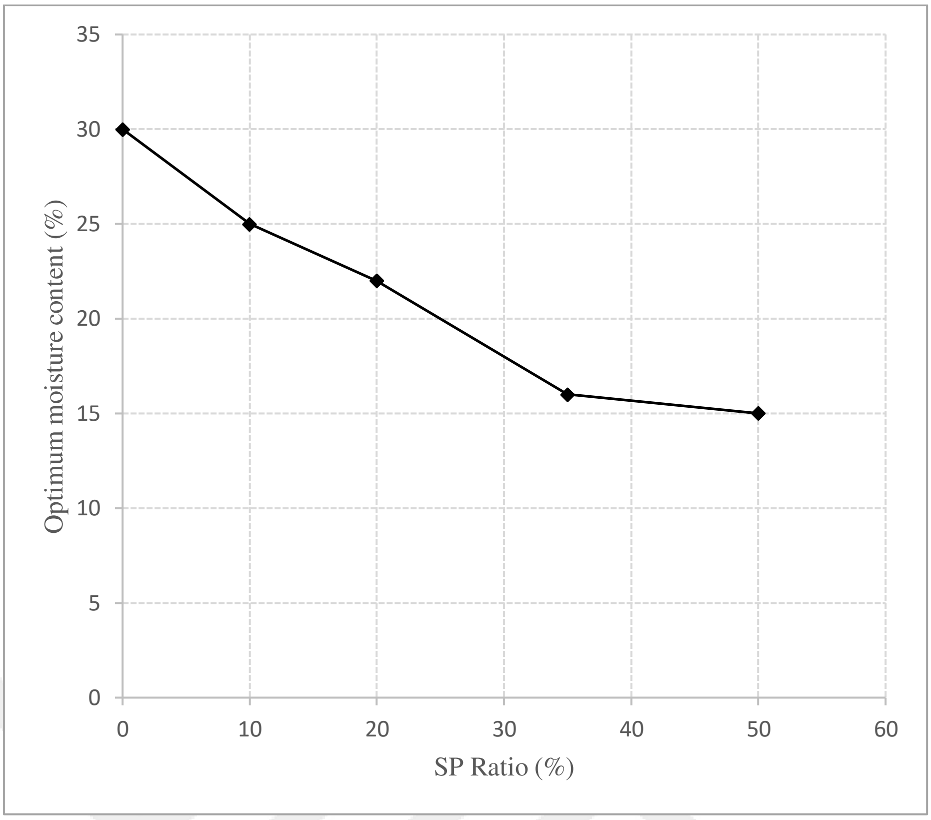

Figure 4.9 Effect of SP on OMC.

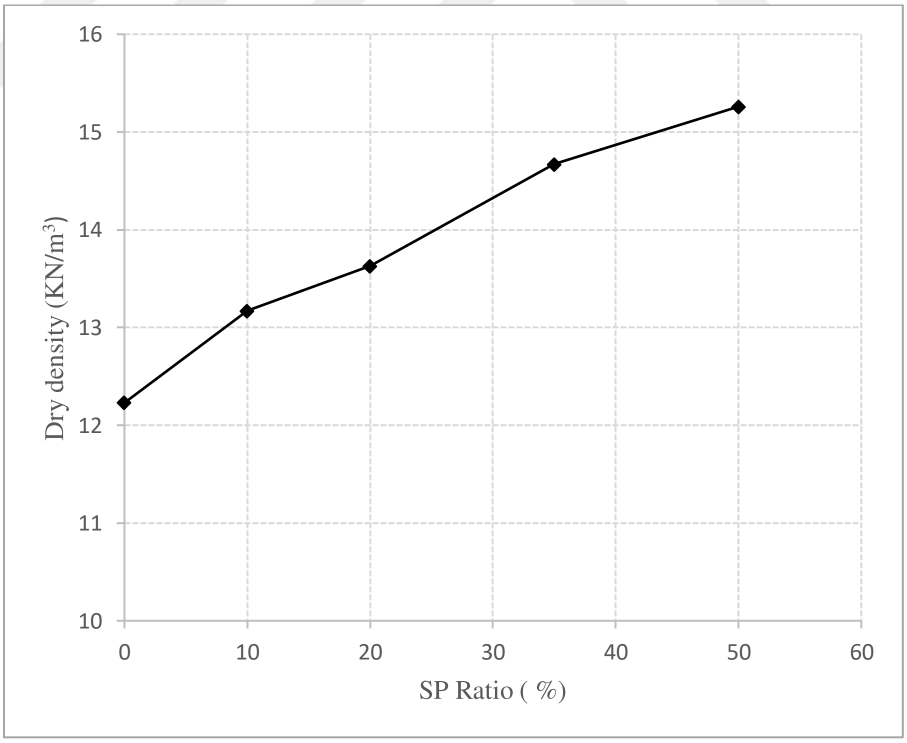

Figure 4.10 Effect of SP on MDD.

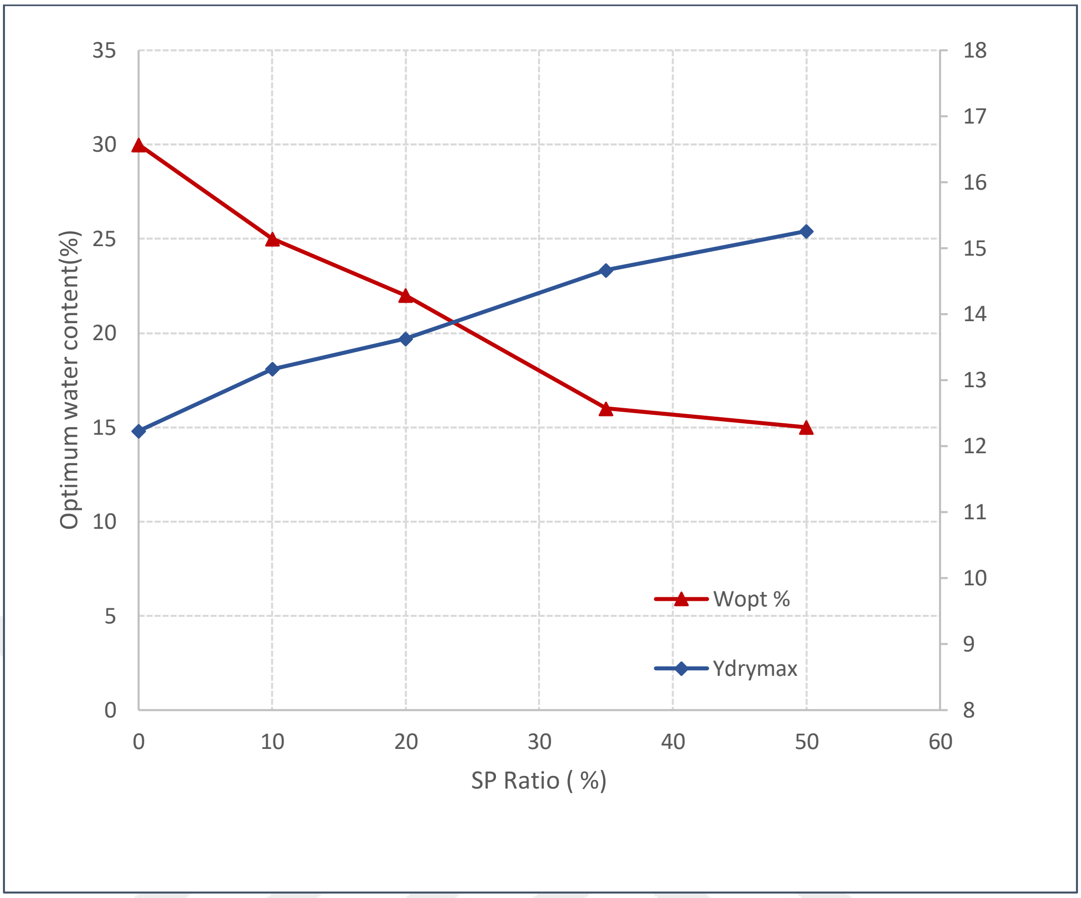

Figure 4.11 Variation of MDD and OMC with SP ratios.

4.4 Unconfined Compression Strength (UCS) Test

The organic soil sample and the stabilized mixtures of stone powder were prepared to unconfined compressive strength test. This test has been conducted after preparing soil samples immediately and compacting them according to MDD and OMC results obtained from compaction test. All obtained results are shown in Figures 4.12, 4.13 and 4.14. The plot showed that at the end of UCS test the compressive strength decreases with increasing strain rate. Moreover, the maximum value of compressive strength was at 10% addition of stone powder and also 35% of stone powder was at near value to 10% of stone powder. In general the compressive strength of mixtures with stone powder was higher than the compressive strength of untreated soil. One explanation of getting the maximum compressive strength value at 10% and 35% of SP is maybe increasing the cohesion between SP particles and organic soil at these percentages so the particles bonded strongly together. The maximum obtained values were 330.39 kPa and 291.23 kPa at 10% and 35% of stone powder, respectively. Thus the optimum and optimized ratio of stone powder is chosen to be at 10% of SP. Omar (2017) had studied also the effect of stone powder on silty soil. His results also showed variation in compressive strength values, it increased up to 30% SP and then decreased. The best mixture was by using 30% of stone powder and the value of compressive strength was 497.88 kPa. Attom and Al-Sharif (1998) had conducted unconfined compressive strength test on the soil samples mixed with burned olive waste. The 2.5% of burned olive was the optimum value for strength and after this percentage the strength decreased. Chetia and Sridharan (2016) studied the using of rock dust in the soil stabilization. The results showed that the addition of 20% quarry rock granted the soil sample the highest strength value and after that the UCS decreased. Roohbakhshan and Kalantari (2013) had also investigated the using of lime and stone powder in silty soil stabilization. The UCS increased with the increasing of stone powder and lime content.

The brittleness and ductility of materials do not have a big effect on the strength value. However the high ductility materials are more wanted and preferred for engineering properties. The area under stress strain curve from zero to peak value represents the energy absorption of soil. Figure 4.15 shows the changing in energy absorption value with stone powder ratios. The results showed decreasing in energy absorption with increasing SP ratio.

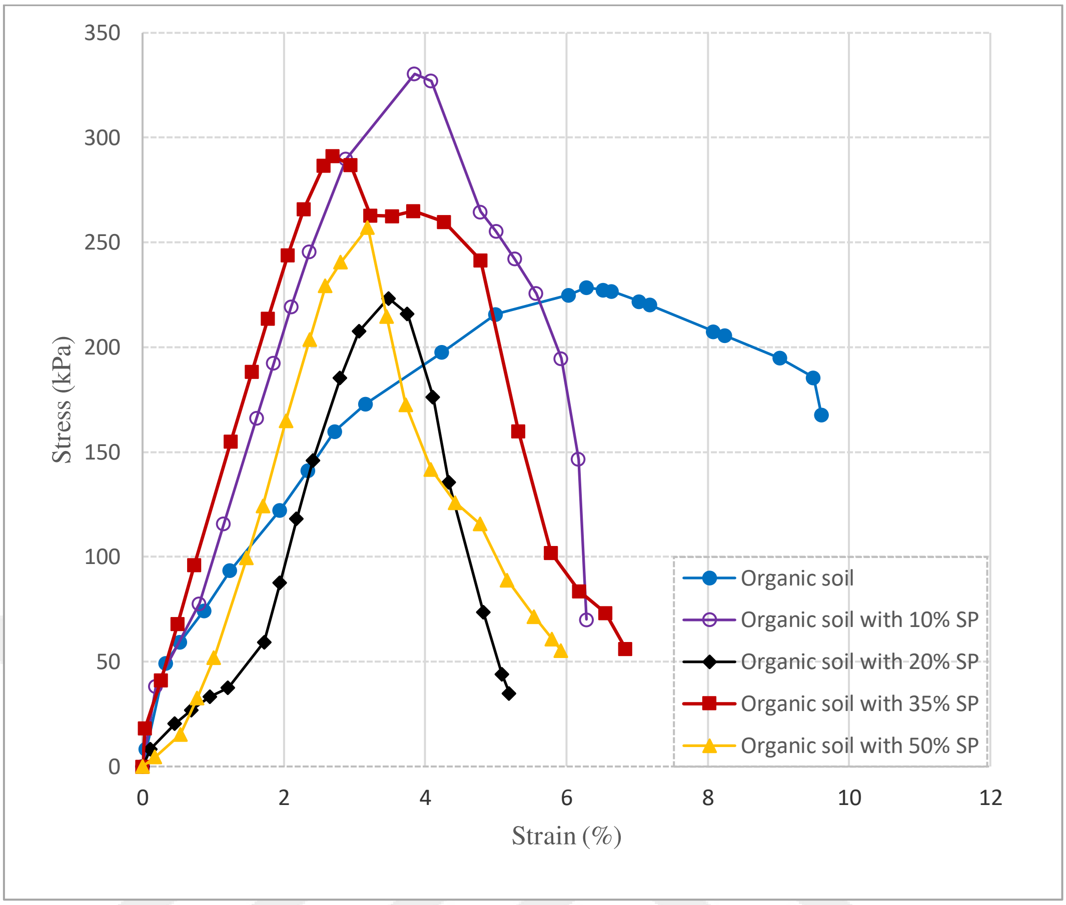

Figure 4.12 Effect of SP on the UCS.

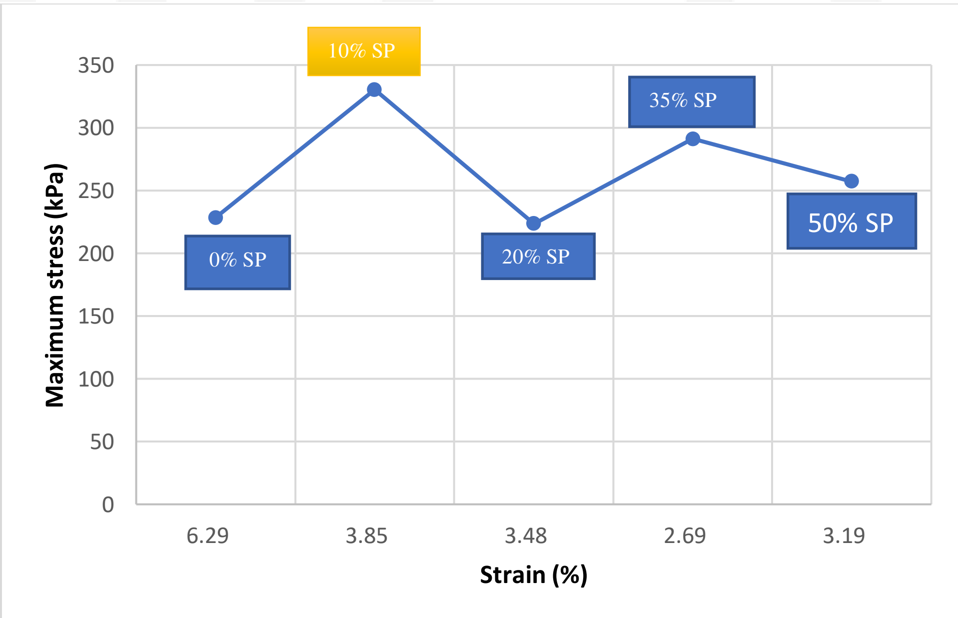

Figure 4.13 Maximum stress vs strain at SP ratios.

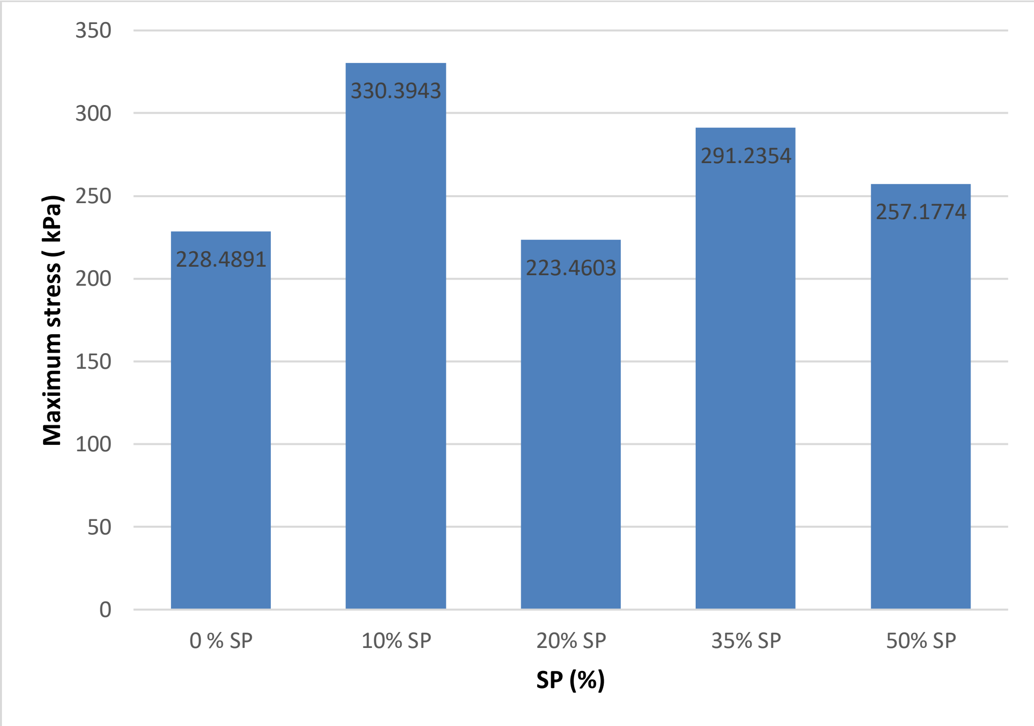

Figure 4.14 Maximum stress with SP ratios.

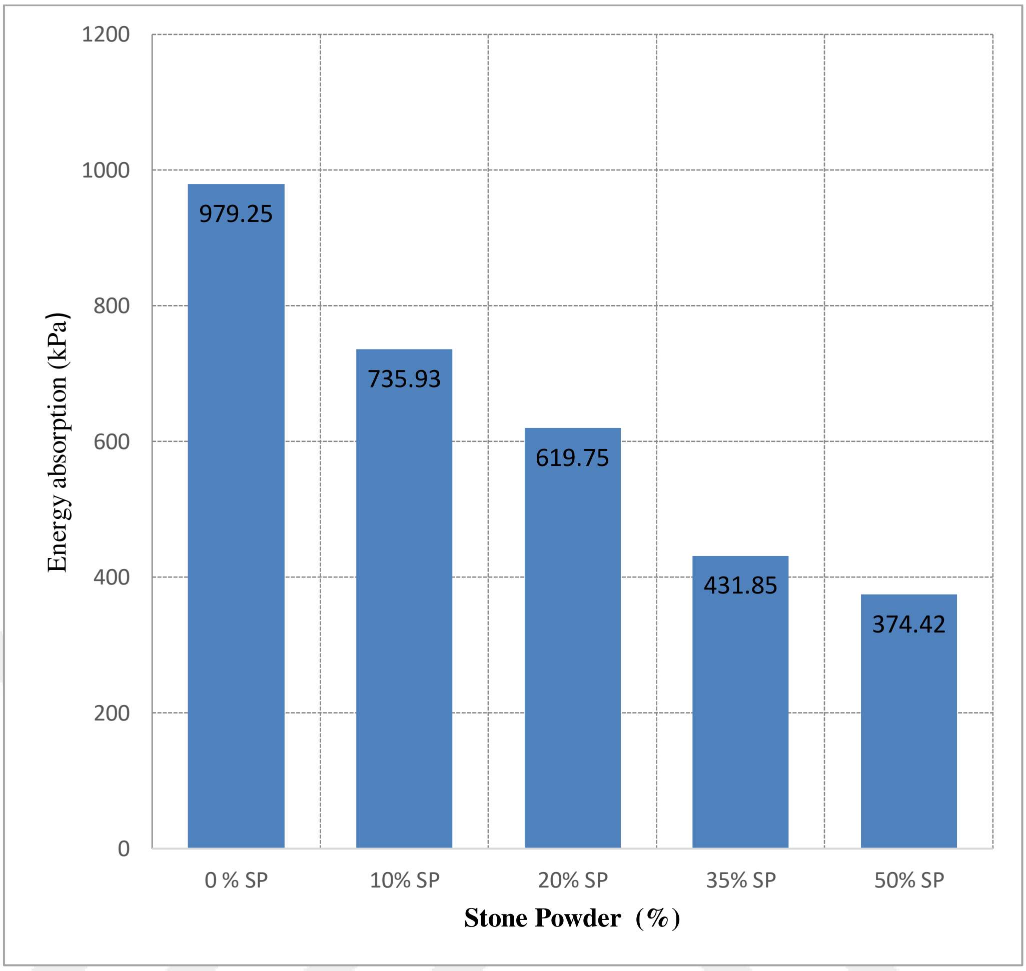

Figure 4.15 Effect of SP on the soil energy absorption.

4.5 Consolidation Test

The conducting of consolidation test was according to ASTM D 2435.The test was performed to obtain several values for untreated and treated soil mixtures. Some of the values that are acquired during consolidation tests are; initial void ratio (e_o), coefficient of consolidation (c_v), compression index (c_c), volume compressibility (m_v) and swelling index (c_s).

Secondary consolidation test is a difficult test to be conducted on such a problematic organic soil. The essential consolidation test starts when soil sample immersed totally in water and with the time moving the problem increases. In this study the effect of stone powder ration on organic soil have been studied. The compression curve (e-log p) of clean soil and stabilized one is shown in the Figure 4.16.

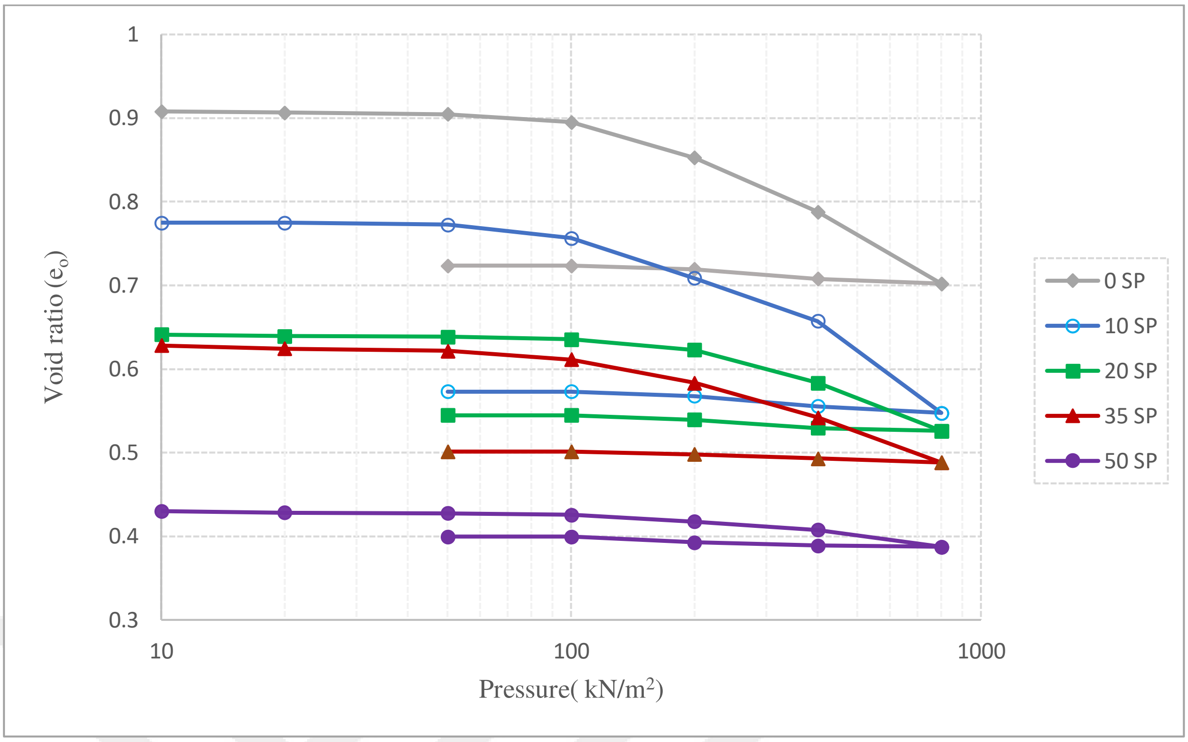

Figure 4.16 Compression curve (e-log p) of different SP ratio.

4.5.1 Initial Void Ratio

One of the significant characteristic of consolidation test is the initial void ratio. The calculation was for all mixtures of stone powder and clean soil. The value of initial void ratio decreased from 0.908 to 0.43 with adding more stone powder percentage see Figure 4.17. This reduction in the value of e_o is maybe because of that the absorption for water content reduced by the addition of SP ratio and the MDD of soil increased with adding more stone powder so a rearrangement for soil particles has happened. Several performed studies by Saeid et al. (2012) had explained the decreasing of initial void ratio for the stabilized soil with increasing fly ash ratio. Also Mohanty et al. (2016) got the same result where the e_o decreased with adding more dolochar waste material to the soil up to 30%.

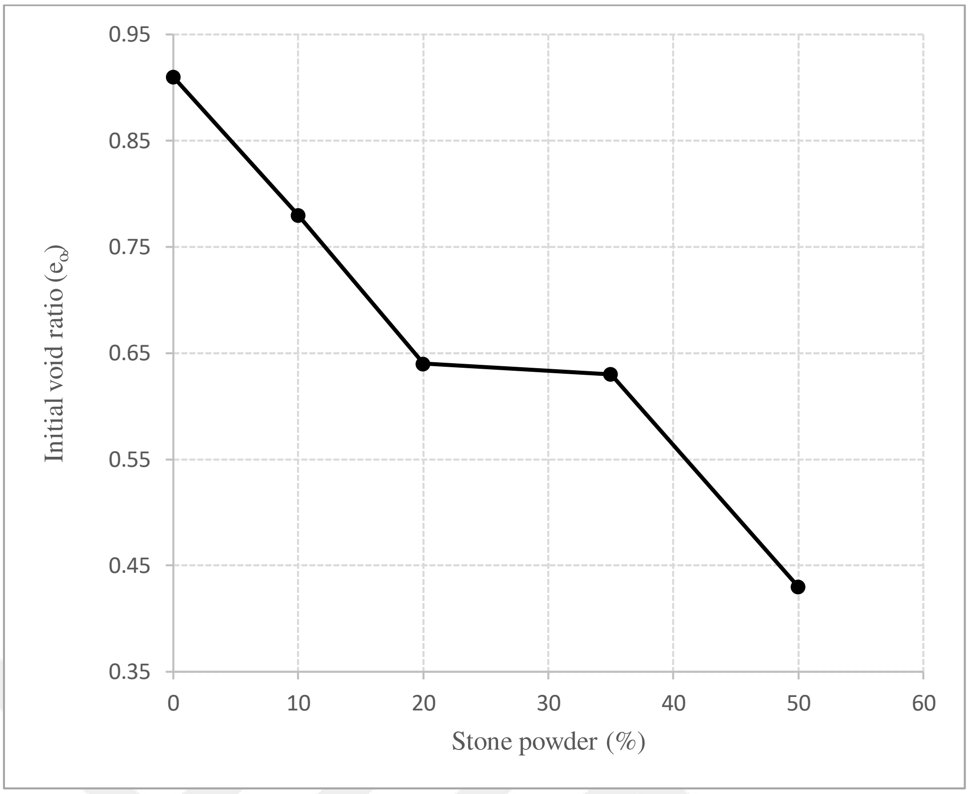

Figure 4.17 Variation of e_o with SP content.

4.5.2 Compression Index (c_c)

Several plots were illustrated to see the changing of c_c value under different pressure loading. The best plot was at pressure between (200 and 400) kPa. Figure 4.18 shows the relationship between c_c and SP percentage which refers to decrease in c_c value with more stone powder ratio. Figure 4.19 shows the variation of c_c also with pressure between 200 kPa and 800 kPa. The value of c_c has increased at 10% of SP and then decreased. The best plot was at pressure between 200 kPa and 400 kPa where c_c value was decreasing with adding SP up to 50%. It decreased from 0.004 to 0.001. This reduction is result of the increasing of non-plastic content in soil mixtures which gained from increasing SP ratio. This decreasing is good for soil because it decreases the settlement of soil so it is very good in pavement construction. Kumar and Sharma (2004) studied the impact of fly ash on expansive soil stabilization and non-expansive one. The same results were obtained for both non-expansive and expansive soil mixtures. Decreasing in c_c was acquired with the addition of more fly ash amount.

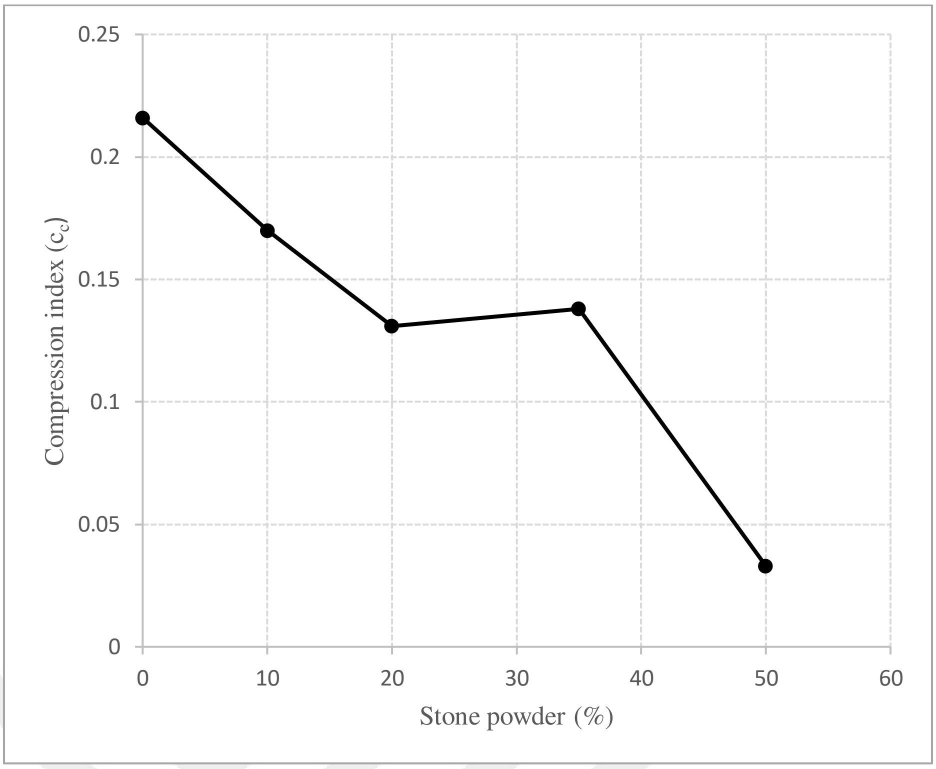

Figure 4.18 Relations between Cc and SP at pressure increment (200-400) kPa.

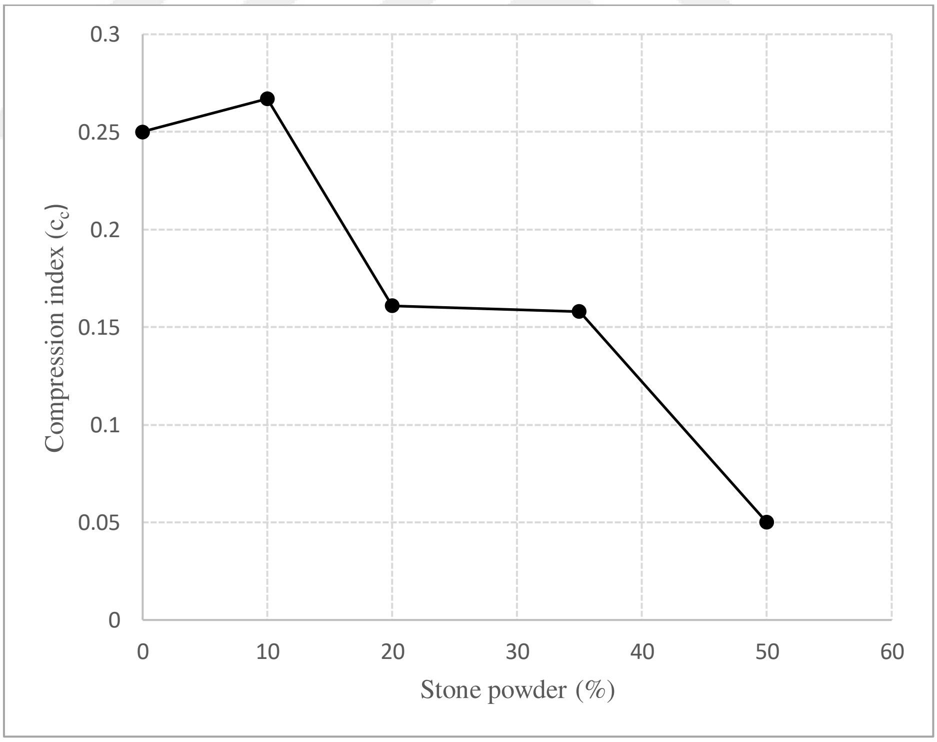

Figure 4.19 Relation between Cc and SP at pressure increment (200-800) kPa.

4.5.3 Swelling Index (c_s)

The calculations for c_s value are represented at unloading case, in the pressure between 200 kPa and 400 kPa as shown in the Figure 4.20 the c_s decreased from 0.0377 to

0.013. It is increased little at 10% SP and then decreased. Another case for c_s value at pressure between 200 kPa and 800 kPa which shows also decreasing in c_s value after adding 10% of SP but the decreasing was at the end of test near the value of c_s for untreated soil see the Figure 4.21. As seen from the plots we can see there is some increase in c_s value, this small increase may be because of the increasing of MDD value with adding more stone powder and the general decreasing in c_s values is explained by the increasing of non-plastic materials and the decreasing of water content with adding more SP ratio. Ramadas et al. (2010) had studied the stabilization of soil by adding sand at different percentages. The value of swelling index had decreased at 30% of sand by 71% with comparison with the clean soil. Afolagboye and Talabi (2013) used tyre ash to stabilize lateritic soil. They used 8% of tyre ash and studied its effect on consolidation properties. As result, the adding more percentage of tyre ash reduced the value of swelling index and compression index.

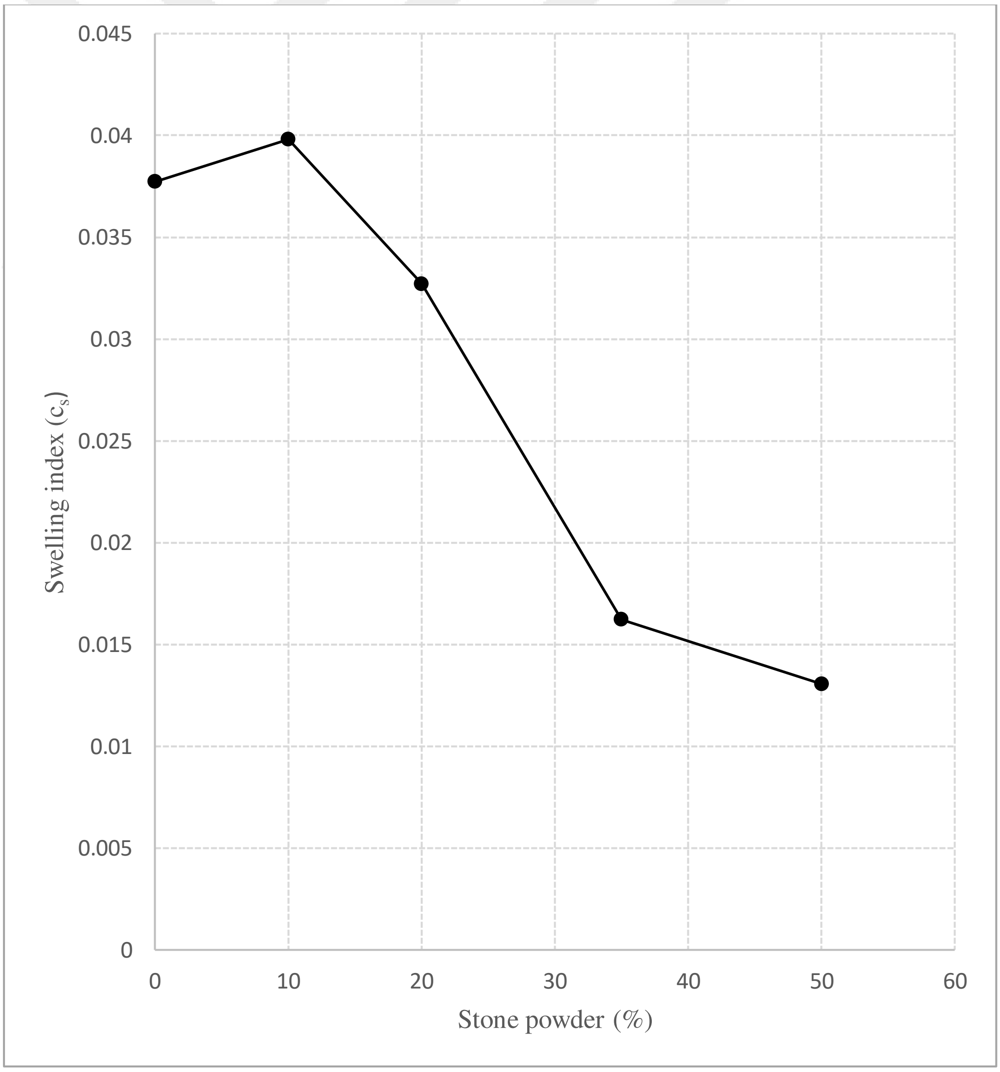

Figure 4.20 Relation between c_s and SP at pressure increment (200-400) kPa.

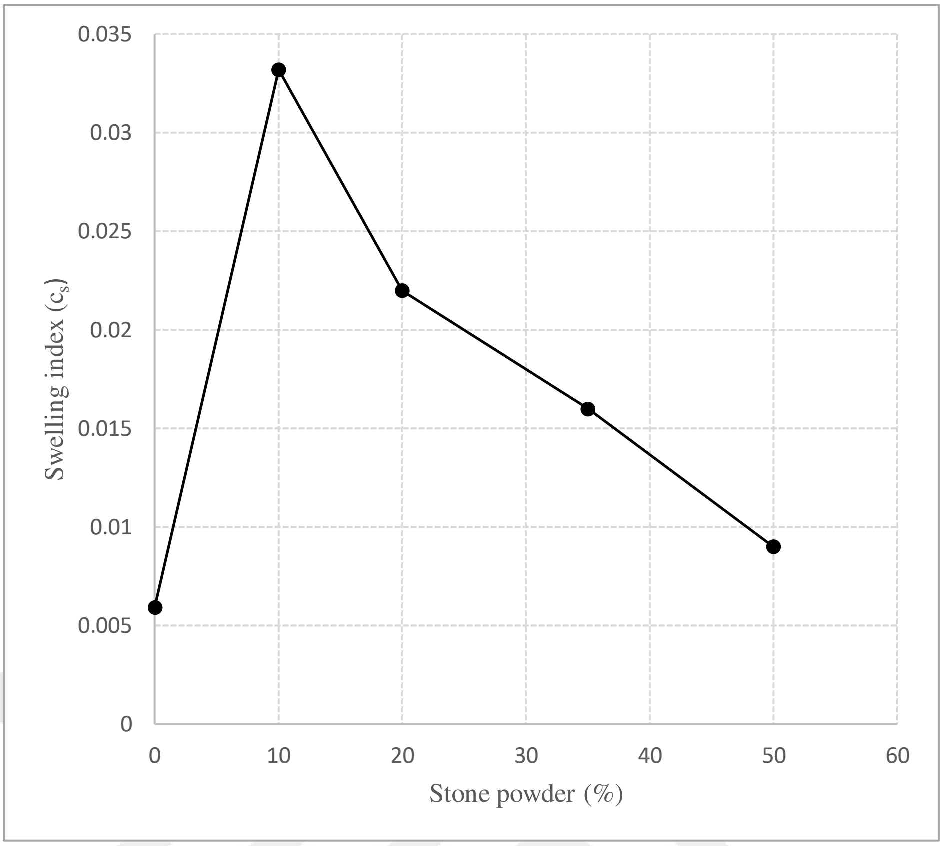

Figure 4.21 Relation between c_s and SP at pressure increment (200-800) kPa.

4.5.4 Coefficient of Consolidation (c_v)

The relationship between consolidation coefficient and the applied stresses for different ratios of SP has explained by using one dimensional consolidation test. The results showed that the value of c_v at the pressure 400 kPa has increased from 0.001766 cm^2/sec to 0.0071 cm^2/sec until adding 35% of SP and then it decreased at 50% of SP. It was equal to 0.001766 which is the same value of c_v for untreated soil see Figure 4.22.On the other hand, at pressure value equals to 800kPa, the c_v has increased from 0.0023 cm^2/sec to 0.0078 cm^2/sec at 20% of SP and then decreased to 0.0018 at 35 % of SP see the Figure 4.23. The increasing in c_v value may be is result of decreasing m_v value when more stone powder added to soil mixture. Chang and Cho (2014) studied the effect of biopolymer on soil improvement .The noticed that as the biopolymer content increases, the swelling index increases and inversely the coefficient of consolidation decreases. Jain and Puri (2013) investigated the effect of stone dust on consolidation coefficient. The results showed that there was increasing in c_v value from 0.101 x 10^{-2} to 1.005 x 10^{-2} cm^2/sec with the increasing of stone dust content and the increasing was at different effective stresses. However, the consolidation coefficient also decreased when the effective stress increased at same Stone dust contents.

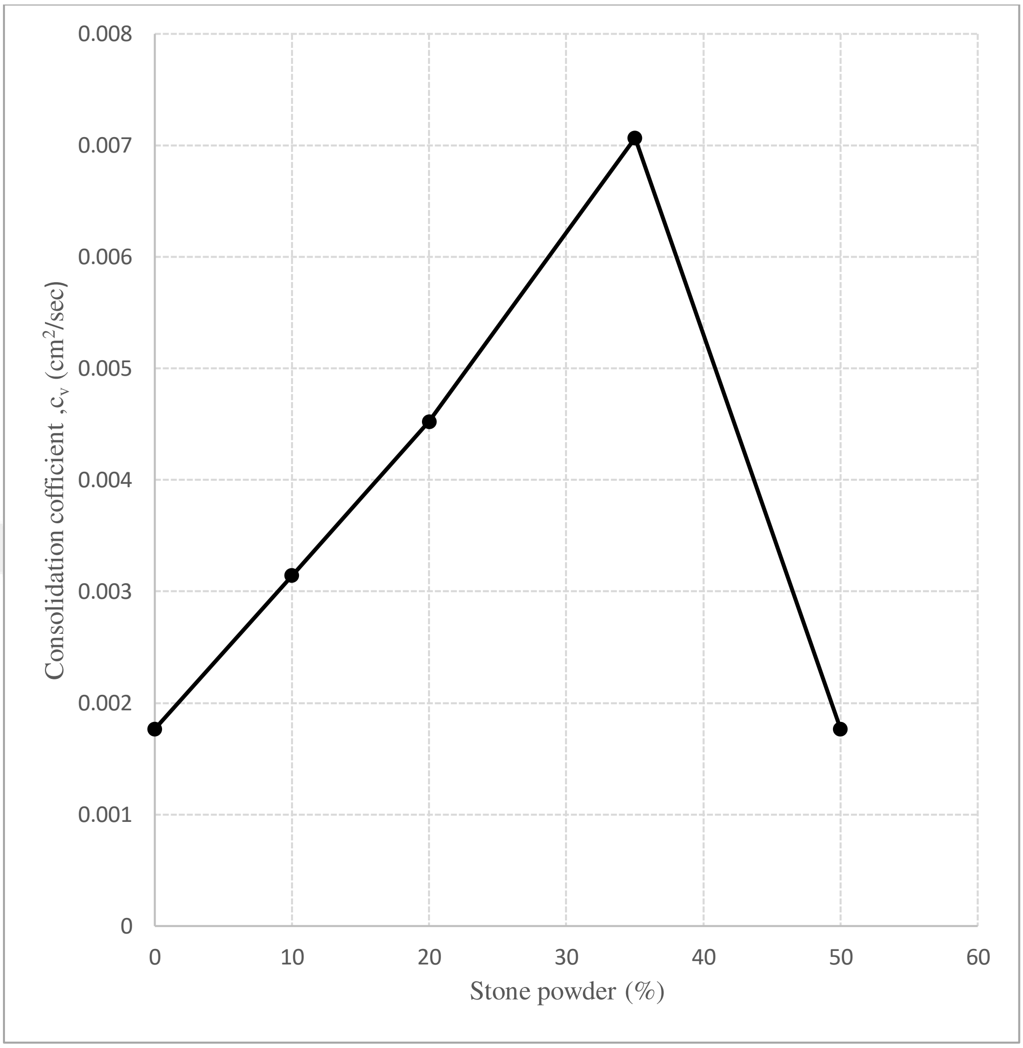

Figure 4.22 Variation of consolidation coefficient (c_v) with SP at pressure 400 kPa.

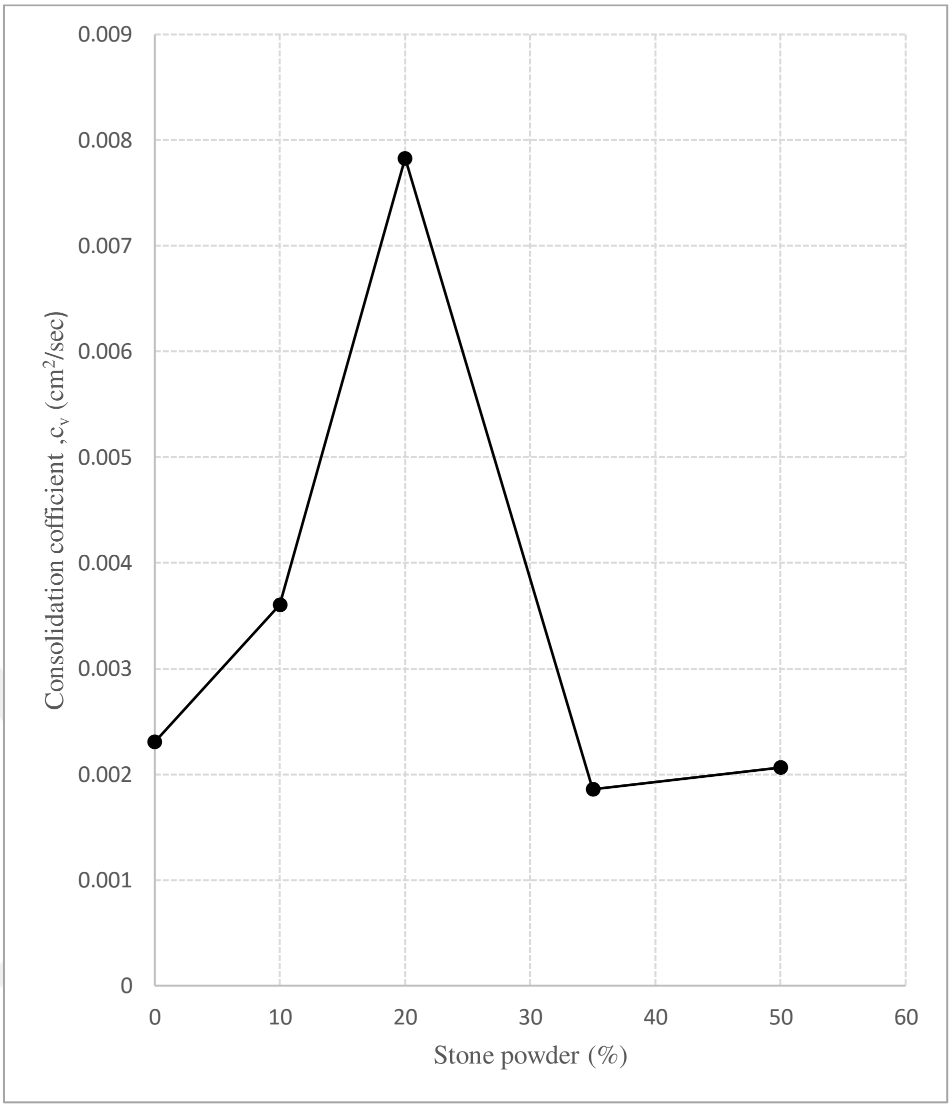

Figure 4.23 Variation of consolidation coefficient (c_v) with SP at pressure 800 kPa.

4.5.5 Volume Compressibility (m_v)

The coefficient of volume compressibility has decreased generally by adding stone powder. The results showed that this coefficient has decreased from 0.01121 cm^2/gm to 0.00349 cm^2/gm at 800kPa as shown in Figure 4.24.This reduction in m_v value is because of increasing the content of non-plastic materials at different loading. Furthermore, the value of m_v had decreased from 0.0 170 to 0.0112 by increasing the value of effective stress from 400 kN/m^2 to 800 kN/m^2. This test results are in agreement with Jain and Puri (2013) results which approved that the increasing in stone dust percentage decreased the value of m_v and it decreased also more with applying more effective stress.

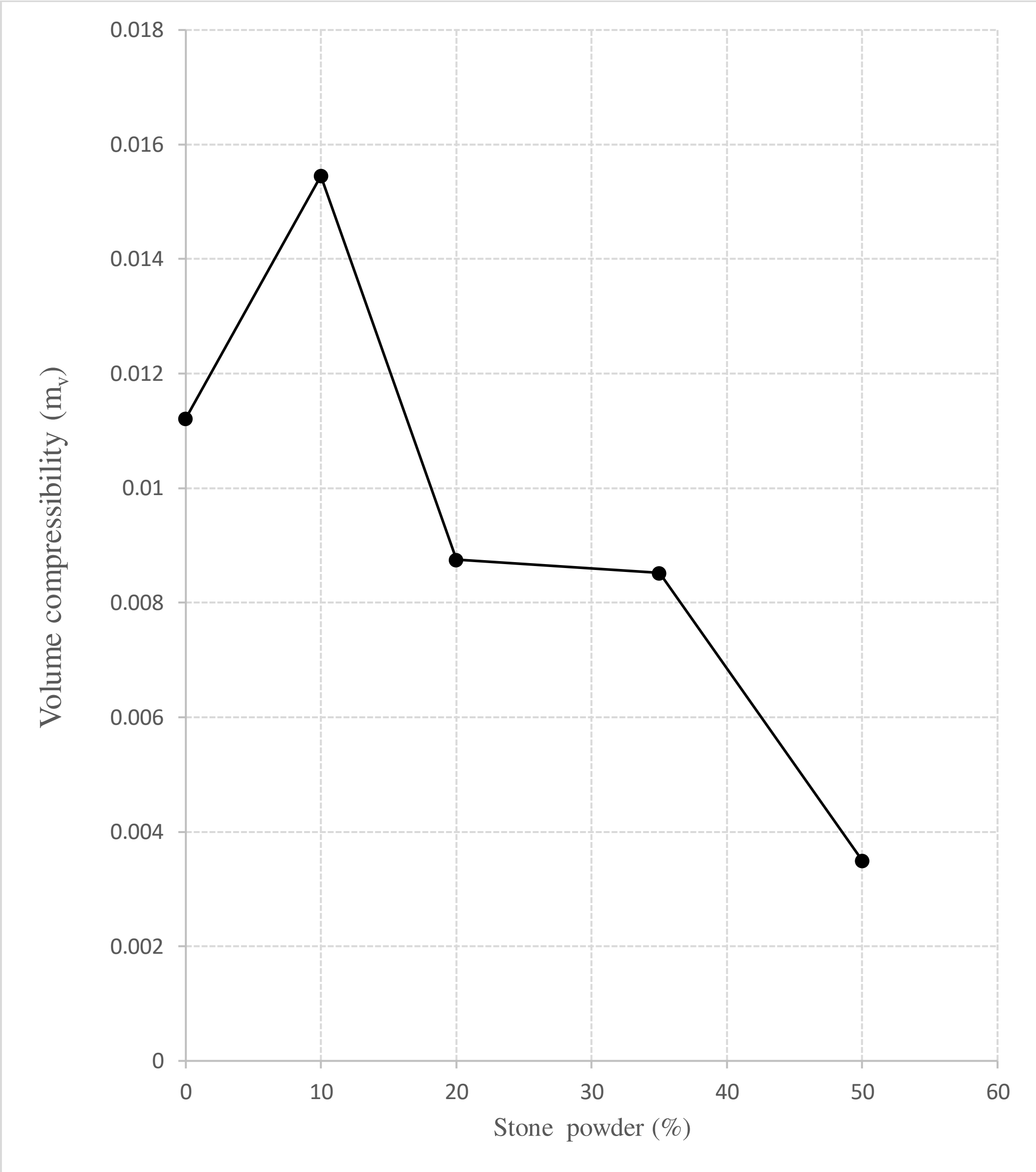

Figure 4.24 Variation of volume compressibility with SP at pressure 800 kN/m$^{2.}$

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 General

In this chapter, an attempt was made to sum up the experimental results. In addition, the advantages and disadvantages of using stone powder as an additive material to stabilize organic soil are concluded in this chapter. The recycling of stone powder helps in save environment from pollution and reduce the hazards on humanity. Some recommendations are given to help or guide the future researcher to find another beneficial topic about stabilizing organic soil or recycling stone powder with another soil.

5.2 Conclusions

1. The liquid limit, plastic limit and plasticity index of treated soil have decreased by 33.84%, 34.4% and 33.33% respectively compared with clean soil. The reduction of plasticity index is result of replacing the plastic material with non-plastic ones.
2. From fall cone test the undrained shear strength of soil mixtures has increased by increasing SP ratio. The maximum value was at 10% of SP.
3. The addition of SP to organic soil has changed the behavior of compaction results of untreated soil. The OMC has decreased by 50% from 30% to 15%, as the SP particles do not absorb a lot of water compared with organic soil particles absorption to water. Moreover, the MDD has increased by 19.85% from 12.23 kN/m^2 to 15.26 kN /m^2 due to increasing the content of stone powder which has higher unit weight than organic soil particles.
4. The UCS of treated samples has increased comparing with the untreated sample but the best obtained value for UCS was at 10% of SP which equals to 330.39 kPa. The ratio of UCS improvement was by 30.84% for untreated soil.
5. The obtained values from one-dimensional consolidation laboratory test, the initial void ratio (e_o) has decreased by 52.64 % with adding SP up to 50%. Likewise, the compression index (c_c) has decreased by 84.72% with adding SP at pressure between

200 kPa and 400 kPa. The swelling index (c_s) has decreased also by 65.37% at same pressure by adding more percentage of SP, it decreased from 0.037 to 0.013. In addition, the compression index and swelling index were also reduced with increasing the effective stress. The reduction in c_c value improves the soil in the side of consolidation characteristic which means less settlement so less problem in the soil and it is more preferred to construct on it.

6. For consolidation coefficient which obtained from consolidation test, the value of c_v had increased with the addition of SP. It increased by 24.78% from 0.00176 to 0.0071 at 400 kPa pressure.

7. The coefficient of volume compressibility has decreased by adding stone powder. The results showed that this coefficient has decreased from 0.0170 cm^2/gm to 0.0034 cm^2/gm and from 0.01121 cm^2/gm to 0.00349 cm^2/gm at 400 kPa and 800kPa respectively .This reduction in m_v value is because of increasing in the content of non-plastic materials at different loading. Furthermore, the value of m_v had decreased from 0.0 170 to 0.0112 by increasing the value of effective stress from 400 kN/m^2 to 800 kN/m^2.

The SP might be considered as a good stabilizer for organic soil. This technique will reduce the using of landfills for waste disposal and on the financial side, it is viewed as gratis and effectively accessible.

5.3 Recommendations

There are several recommendations and scopes for applying more future researches and studies. The following recommendations could be listed:

1. The performed tests were to stabilize organic soil with SP additive material, another problematic soil could be used to be stabilized with SP such as bentonite.

2. The SP could be used with another waste material such as sewage sludge ash, fly ash or lime to increase soil stabilization rate and enhance the properties of soil. In addition, for increasing the hardness of materials, SP could be used as filler material.

3. Using another percentage of SP or conducting another tests maybe lead to another results and improvement.

4. The effect of particle size and shape of SP could change the results of applied tests. Therefore, the used stone powder should have the mineralogical and chemical properties.

5. The studying of curing time and temperature should be taken into account because all tests were conducted immediately on test samples so maybe performing the test after 7, 14 and 28 days will lead to higher results in UCS value.

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