KARADENİZ TECHNICAL UNIVERSITY
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

DEPARTMENT OF CIVIL ENGINEERING

STABILIZATION OF EXPANSIVE SOILS USING PUMICE, LIME, AND MARBLE

MASTER'S THESIS

Dina AQRA

JULY 2021
TRABZON

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This thesis is accepted to give the degree of
"MASTER OF CIVIL ENGINEERING"
By
The Graduate School of Natural and Applied Sciences at
Karadeniz Technical University

The Date of Submission	: 23 /06 /2021
The Date of Examination	: 14 /07 /2021

Supervisor : **Assoc. Prof. Dr. Zekai ANGIN**

Trabzon 2021

ACKNOWLEDGEMENTS

First and foremost, I want to thank the respectful members of the Geotechnical Department of Karadeniz Technical University for giving me the opportunity to complete my M.Sc. thesis. I would like to thank my advisor, Assoc. Prof. Dr. Zekai ANGIN, who was a great mentor, and who shared his knowledge with me and guided me throughout my study.

I wish to express my respectful gratitude to Prof. Dr. Sabriye Banu İKİZLER for her encouragement, patience, valuable time, and supportive guidance during the experimental works and the preparation of this thesis.

My gratefulness goes to Soil Mechanics Laboratory staff Mr. Yüksel HARDAL for his support and in the implementation of the laboratory works.

I would like to express my gratitude to teaching assistant Mrs. Kübra UZUN from Physics Department at Karadeniz Technical University for her helping in performing the mineralogical analysis for the soil.

I would like to thank Ünye Cement Factory and its employees for offering all kinds of opportunities and assistance to provide the soil used in this research.

Finally, I want to express my deepest gratitude to my parents and my family for their encouragement, and endless moral support. I also would like to dedicate this thesis to my parents. They, more than anyone else, have been the best role models I could have hoped for. Any achievements of mine are due to them.

Dina AQRA

Trabzon 2021

THESIS ETHICS STATEMENT

I have completed this study titled "Stabilization of Expansive Soils Using Pumice, Lime, and Marble", which I submitted as a master's thesis, which was fully supported by my advisor Assoc. Prof. Dr. Zekai ANGIN, that I have collected the data/samples myself, that I have done/had the experiments/analysis done in the relevant laboratories, that I have fully indicated the information I received from other sources in the text and in the bibliography, that I have acted in accordance with scientific research and ethical rules during the working process, and that I have made all kinds of decisions in case the opposite arises I declare that I accept the legal result. 14/07/2021

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ABSTRACT

STABILIZATION OF EXPANSIVE SOILS USING PUMICE, LIME, AND MARBLE

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The Graduate School of Natural and Applied Sciences
Civil Engineering Graduate Program
Supervisor: Assoc. Prof. Dr. Zekai ANGIN
2021, 92 Pages

Expansive soils widely exist in barren and semi-barren areas and cause severe engineering problems because they tend to shrink when dry and expand when wet. For this purpose, the efficiency of using lime, pumice, and marble to stabilize an expansive soil, which is classified according to Unified Soil Classification System (USCS) as high plasticity soil (CH), was investigated. Several experiments were performed to determine the properties of swelling soil such as wet sieve analysis, hydrometer analysis, Atterberg limits, specific gravity, permeability, compaction, one-dimensional swelling test, and unconfined compressive strength. In addition, differential thermal analysis (DTA) and X-ray diffraction analysis were conducted to determine the mineralogical properties of the soil.

In this study, pumice was used at a range of 0-25% with increments of 5%, whereas lime was utilized at ratios of 0%, 3%, 5%, 6%, and 8%. In addition, marble was used at ratios of 0%, 5%, 7%, 12%, and 15%. The effects of using each of these additives alone on the consistency limits were examined. Afterwards, in the first stage, the Taguchi method for optimization was implemented to analyse the effects of the mixtures of all additives together with soil on the consistency limits. In the second stage by the Taguchi method, the best ratios for each additive were determined and their effects on swelling parameters, unconfined compressive strength, and compaction parameters were studied.

Based on the obtained results, the SP25L6M15 mixture decreased the plasticity index and swelling pressure by 86.34% and 96.91%, respectively, while the SP20L8M15 mixture decreased the liquid limit by 51.04%. Furthermore, the SP25L8M12 mixture enhanced the strength of soil by 93.45%, 428.20%, and 1252.9% for curing times of 1, 7, and 28 days, respectively. In addition, the SP25L8M12 mixture decreased the swell percentage by 99.18%.

Keywords: Expansive Soils, Swelling Soils, Soil Stabilization, Swelling Pressure, Swell Percent, Pumice, Lime, Marble

Yüksek Lisans Tezi

ÖZET

POMZA, KİREÇ VE MERMER KULLANILARAK ŞİŞEN ZEMİNLERİN STABİLİZASYONU

Dina AQRA

Karadeniz Teknik Üniversitesi
Fen Bilimleri Enstitüsü
İnşaat Mühendisliği Anabilim Dalı
Danışman: Doç. Dr. Zekai ANGIN
2021, 92 Sayfa

Şişen zeminler, kurak ve yarı kurak alanlarda yaygın olarak bulunur ve kuru haldeyken büzülme ve ıslak haldeyken genleşme eğilimi gösterdiğinden ciddi mühendislik sorunlarına neden olur. Bu amaçla, Birleştirilmesi Zemin Sınıflandırma Sistemine (USCS) göre yüksek plastisiteli zemin (CH) olarak sınıflandırılmış şişen zemine kireç, pomza ve mermer kullanımının stabilizasyondaki etkisi araştırılmıştır. Şişen zeminin özelliklerini belirlemek için yıkamalı elek analizi, hidrometre analizi, Atterberg limitleri, özgül ağırlık, geçirimlilik, sıkıştırma, tek boyutlu şişme ve serbest basınç dayanımı gibi çeşitli deneyler yapılmıştır. Ayrıca zeminin mineralojik özelliklerini belirlemek için diferansiyel termal analiz (DTA) ve X-ışını kırınım analizi yapılmıştır.

Bu çalışmada pomza %5'lik artışlarla %0-25 aralığında, kireç ise %0, %3, %5, %6 ve %8 oranlarında kullanılmıştır. Ayrıca mermer %0, %5, %7, %12 ve %15 oranlarında kullanılmıştır. Bu katkı malzemelerinden her birinin tek başına kullanımının kıvam limitleri üzerindeki etkileri incelenmiştir. İlk aşamada, tüm katkı malzemelerinin zeminle birlikte karışımlarının kıvam limitleri üzerindeki etkilerini analiz etmek için Taguchi optimizasyon yöntemi uygulanmıştır. İkinci aşamada Taguchi yöntemi ile her bir katkı malzemesi için en iyi oranlar belirlenmiş ve bu oranların şişme parametreleri, serbest basınç dayanımı ve sıkıştırma parametreleri üzerindeki etkileri incelenmiştir.

Deney sonuçlarından elde edilen verilere göre, SP25L6M15 karışımı plastisite indisini %86,34 ve şişme basıncını %96,91 azaltırken, SP20L8M15 karışımı, likit limiti %51,04 azaltmıştır. Ayrıca SP25L8M12 karışımı, 1, 7 ve 28 günlük kür süreleri için zeminin mukavemetini sırasıyla %93.45, %428.20 ve %1252.9 oranında arttırmıştır. Ayrıca SP25L8M12 karışımı şişme yüzdesini %99,18 oranında azaltmıştır.

Anahtar Kelimeler: Şişen Zeminler, Genleşen Zeminler, Zemin Stabilizasyonu, Şişme Basıncı, Şişme Yüzdesi, Pomza, Kireç, Mermer

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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO : American Association of States Highways and Transportation Officials

A_c : Activity of clays

ASTM : American Society for Testing and Materials

CBR : California bearing ratio

CH : Inorganic clays of high plasticity

CL : Inorganic clays, silty clays, sandy clays of low plasticity

DTA : Differential thermal analysis

e : Void ratio

G_s : Specific gravity

LL : Liquid limit

MDD : Maximum dry density

MH : Inorganic silts of high plasticity

ML : Inorganic silts, silty or clayey fine sands, with slight plasticity

ω_n : Natural water content

OMC : Optimum moisture content

PI : Plasticity index

PL : Plastic limit

q_u : Unconfined compressive strength

S/N : Signal to noise ration

SEM : Scanning electron microscopy

UCS : Unconfined compressive strength

USCS : Unified soil classification system

XRD : X-ray diffraction

γ_n: : Natural unit weight

1. GENERAL INFORMATION

1.1. Introduction

The stability and service life of structures directly correlate with the engineering characteristics of the soil on which they settle and/or the soil used in the fill. For this reason, soils, together with the foundation systems, must be able to safely resist the stresses created by the building weights and usage loads. In addition, due to environmental and climatic conditions, changes in soil properties (for example, swelling, heave, settling, changes in water content, etc.) should not occur at a level that will cause severe effects on the structure, such as reduction in bearing capacity or emergence of additional stresses. Therefore, in engineering applications, it is common to build structures on soil with adequate properties.

In the semi-arid and arid places, expansive soils cause severe geotechnical problems because they tend to shrink in the dry weather and heave in wet weather. They are considered a natural hazard and worldwide problem that causes several challenges for civil engineers and huge damage to structures. Therefore, it is very important to determine the swell that would occur due to swelling soil.

One of the reasons for the examination of the swelling behaviour of soils is the great damage caused by swelling. In fact, the problems created by the swelling soil are of a size that cannot be underestimated financially. According to recent studies, swelling soils cause an annual loss of 9 billion dollars in the USA alone (Jones and Jones, 1987). This damage is more than that caused by other natural disasters such as hurricanes, earthquakes, floods, etc. (Holtz, 1984).

Using chemical additives is aimed to enhance the physical, chemical, and mineralogical properties of soil. Over time, the places where safe buildings can be built have reduced and the necessity to build on any kind of soil has arisen. For these reasons, soil improvement (stabilization) is a broad subject in which geotechnical science is interested. Stabilizing soils by using chemical additives is commonly and economically regarded as an efficient method to prevent volume changes. Stabilization of swelling soils with different materials including lime, fly ash, and cement has proved beneficial (Desai and Oza, 1997).

In addition, lime is widely utilized as a chemical stabilizer for swelling soils with other additives like pumice, fibers, marble, rice husk ash, etc.

1.2. Objective of Study

In the previous studies, improving swelling soils using cement, lime, and fly ash has been investigated. Some studies have addressed stabilization using marble, lime, or pumice alone or mixtures of pumice-marble, pumice-lime, and marble-lime mixtures with clay. However, this study aims to present the effect of utilizing triple mixtures of pumice-marble-lime as a stabilization material for soil. The admixtures were added to the soil at different percentages and separately to compare the effect of each admixture.

The main objectives of this study include considering the changes in the geotechnical properties such as plasticity, swelling percent, unconfined compressive strength, and compaction parameters on the treated soil with pumice, lime, and marble. Besides, this study checks whether a stabilization can be made, and it tries to reveal which additive ratio or ratios show the most appropriate performance in stabilization. Additionally, the Taguchi method is used to reduce the number of test trials and to find the optimum mixtures to enhance the plasticity properties, swelling potential, and strength of the expansive soil collected from a site in Ordu city. Following these purposes, an experimental study was carried out at Karadeniz Technical University Geotechnical and Transportation Laboratory. This study confirms that pumice, lime, and marble can successfully be used for the stabilization of the high plasticity clayey soil with swelling potential.

Details about the aims and content of the thesis, comprehensive information about the soil stabilization and expansive soil closely related to the thesis topic, and summaries of the literature studies regarding the stabilization of expansive soil using chemical additives are introduced in Chapter 1; details about the methodology of the experimental study and information about the materials used in the study (natural soil, pumice, marble, and lime) are presented in Chapter 2; The results of the experiments, discussion of using each additive alone or together in stabilization, and the change curves of the test parameters are included in Chapter 3; the final conclusions of the study are mentioned in Chapter 4. As for Chapter 5, it introduces the general results obtained from this thesis and the suggestions presented to shed light on future academic studies. Finally, the references used within the thesis are listed in Chapter 6.

1.3. Clay Minerals

Clay materials are very important in the fields of construction, geology, agriculture, industry, etc., therefore, related research on their characteristics has been of great interest since ancient times. Many researchers have dedicated their efforts to studying clay materials where they found major differences in characteristics between soil and clay materials, even if they have the same texture, colour, or general appearance. It was noticed that clays of the same chemical composition considerably have very different physical properties and that clays with identical physical properties might have big differences in chemical composition (Grim, 1968).

The term clay implies particle size or represents a class of minerals. The Wentworth scale considers 4 microns as the upper limit of the particular size of clay (Wentworth, 1922), while most classification systems used to describe a particular size of less than 0.002 mm. In soil mineralogy, it indicates mineralogical properties such as net negative charge, high resistance to weathering, small particle size, and plasticity when mixed with water.

Basically, clay minerals are made up of silicates of aluminium, iron, and magnesium. Silicon-oxygen tetrahedron and magnesium or aluminium octahedron are the basic crystalline units of clay minerals as shown in figure 1.1.

Tetrahedron units, which consist of a silicon atom at the centre surrounded by four equally spaced oxygen atoms at corners (figure 1.1a), are bounded to make a silica sheet as illustrated in figure 1.1b.

The octahedral units, shown in (figure 1.1d), consist of a magnesium or aluminium atom surrounded by six hydroxyl atoms combined to form the octahedral sheet (figure 1.1e). If the combination consists of aluminium octahedral units, the sheet is called gibbsite (figure 1.2). On the other hand, the combination of magnesium octahedral units is called brucite sheet.

Clay mineral particles are made up of stacks of the sheets mentioned previously, with different ways of bonding among the layers. Kaolinite, montmorillonite, and illite are the most common clay mineral groups. The particle properties and the engineering characteristics of the clay minerals are represented in Table 1.1.

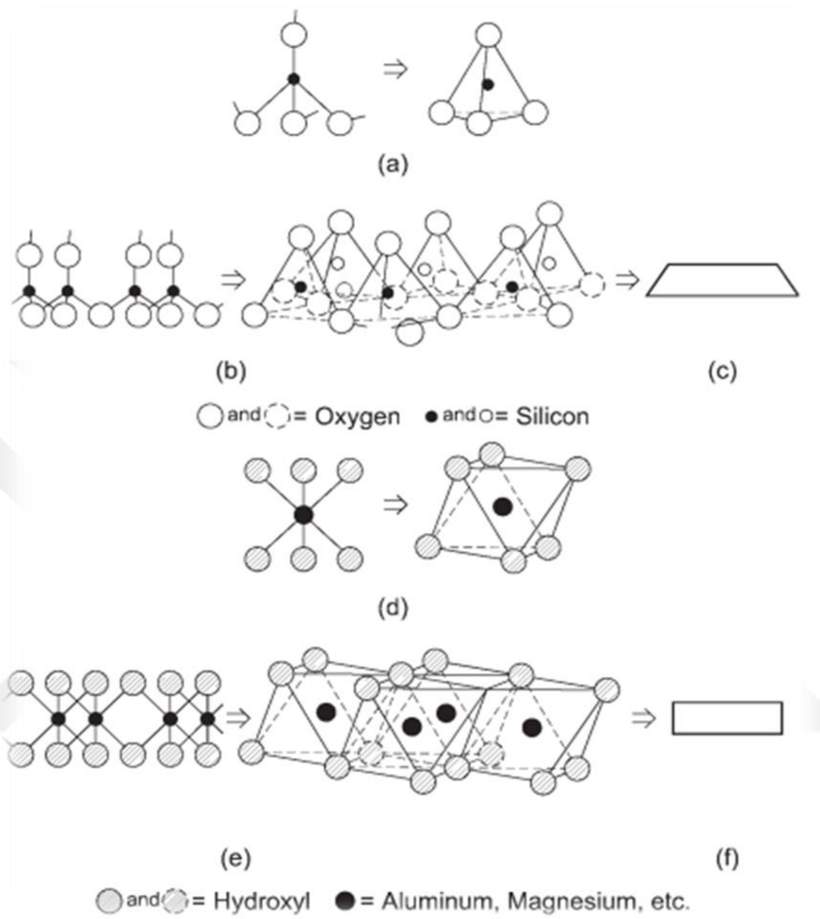

Figure 1.1. Atomic structure of silicon tetrahedra and alumino-magnesium octahedra: (a) silicon tetrahedron; (b) silica sheet; (c) symbolic structure for silica sheet; (d) alumino-magnesium octahedron; (e) octahedral sheet; (f) symbolic structure for octahedral sheet (after Lambe and Whitman, 1969).

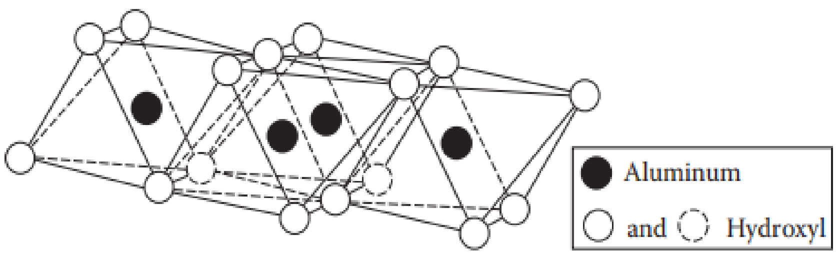

Figure 1.2. Gibbsite sheet (Grim, 1959).

Table 1.1. Features of typical clay minerals: (a) Mitchell, 1976; (b) Skempton, 1953.

Mineral Group	Basal Spacing (A°)	Particle Features	Interlayer Bonding	Specific Surface (m^2/g)	Atterberg Limits[a] LL (%)	PL (%)	SL (%)	Activity[b] (PI/ % Clay)
Kaolinites	14.4	Thick, stiff 6-sided flakes 0.1 to 4 × 0.05 to 2 µm	Strong hydrogen bonds	10 – 20	30 – 100	25 – 40	25 – 29	0.38
Illites	10	Thin, stacked plates 0.003 to 0.1 × 1.0 to 10 µm	Strong potassium bonds	65 – 100	60 – 120	35 – 60	15 – 17	0.9
Montmorillonites	9.6	Thin, filmy, flaks >10 A° × 1.0 to 10 µm	Very weak van der Waals band	700 – 840	100 – 900	50 – 100	8.5 – 15	7.2

1.3.1. Kaolinite

The combination of a sheet of gibbsite with a sheet of silica is the base structure of kaolinite, which is shown in figure 1.6a, with a comparatively strong hydrogen bonding. The kaolinite particle might be formed by more than 100 stacks.

Kaolinite, which has a structural formula of $Al_2Si_2O_5(OH)_4$, has electrically neutral layers (Gillott, 1987). Kaolinite is generally nonexpansive and has a cation exchange capacity of 3 mel/100g. It also has a specific surface area, which is the surface area of clay particles per unit mass, of about 10-20 m^2/g. The scanning electron microscopy (SEM) of kaolinite is shown in figure 1.3.

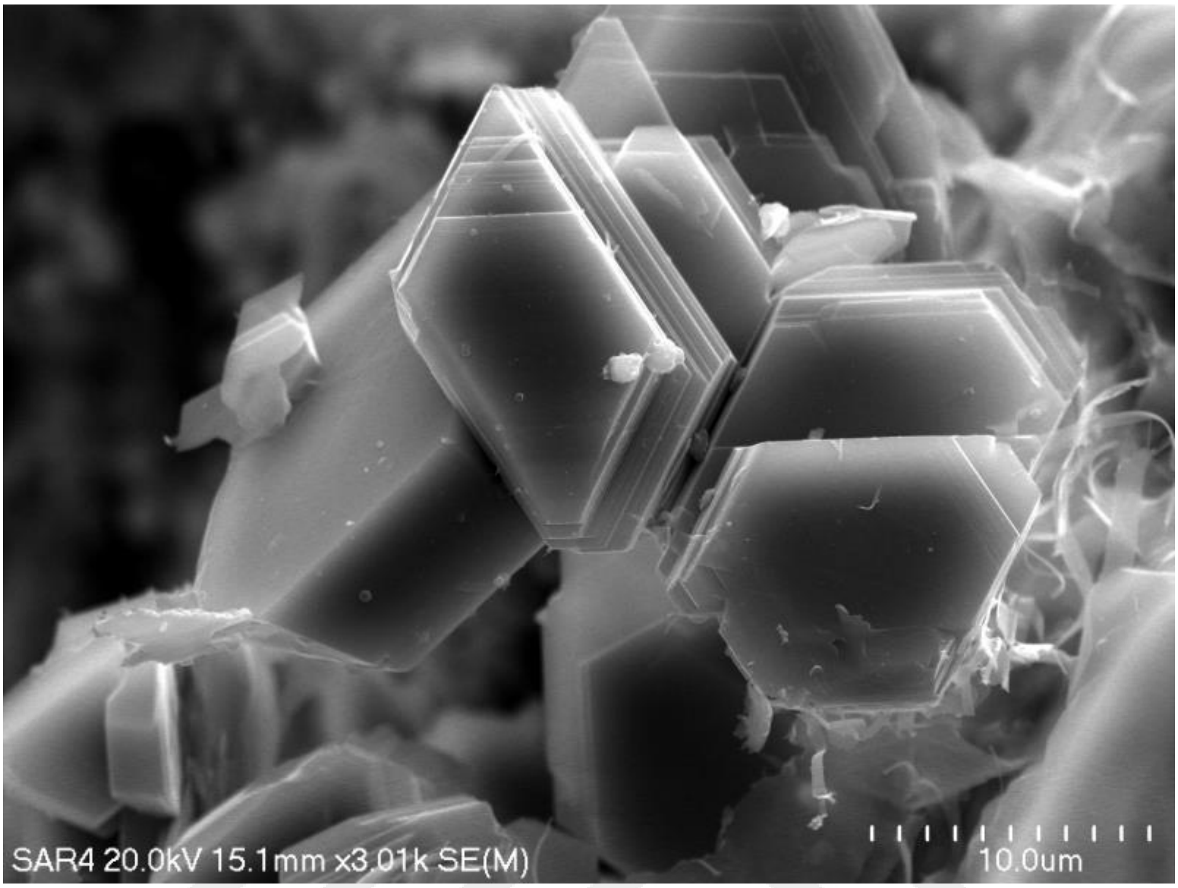

Figure 1.3. SEM of kaolinite (URL-1, 2021).

1.3.2. Illite

Illite is made up of a gibbsite sheet between two sheets of silica as illustrated in figure 1.6b. There is a partial substitution of silicon by aluminium in silica sheets to balance the potassium ions that bond the sheets. This leads to relatively weak bonding and less swelling potential than montmorillonite. The cation exchange capacity of illite is 25 mel/100g and it has a specific surface area of 80-100 m^2/g. Figure 1.4 displays a diagrammatic sketch of the structure of illite. The scanning electron microscopy of illite is shown in figure 1.5.

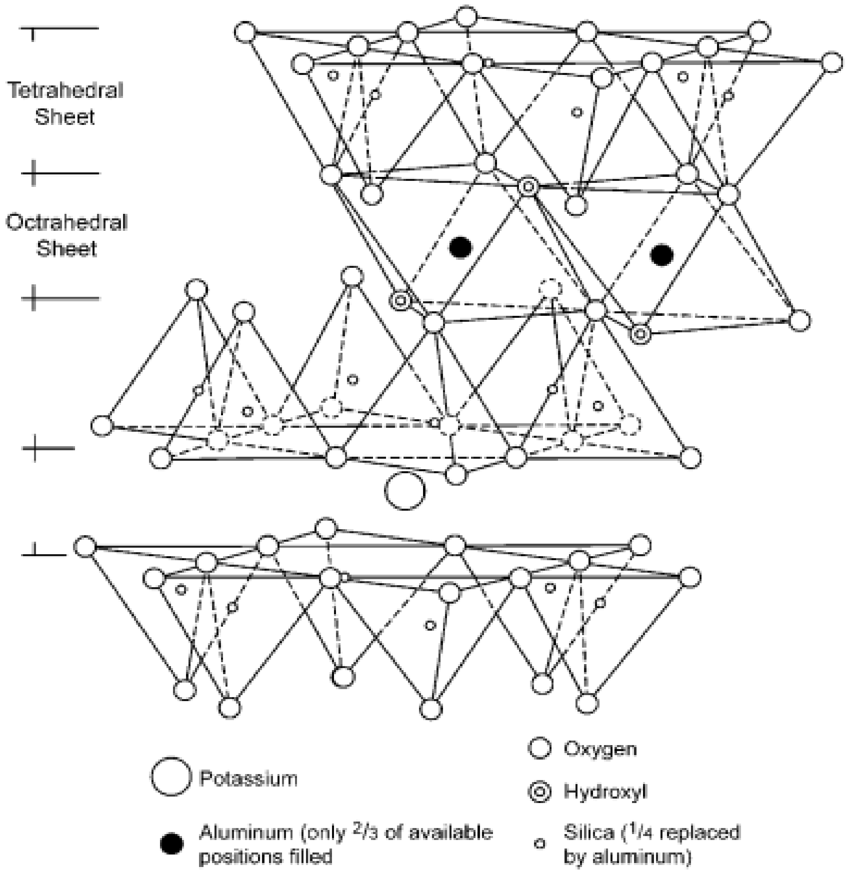

Figure 1.4. Schematic diagrams of the structure of illite (Grim, 1962).

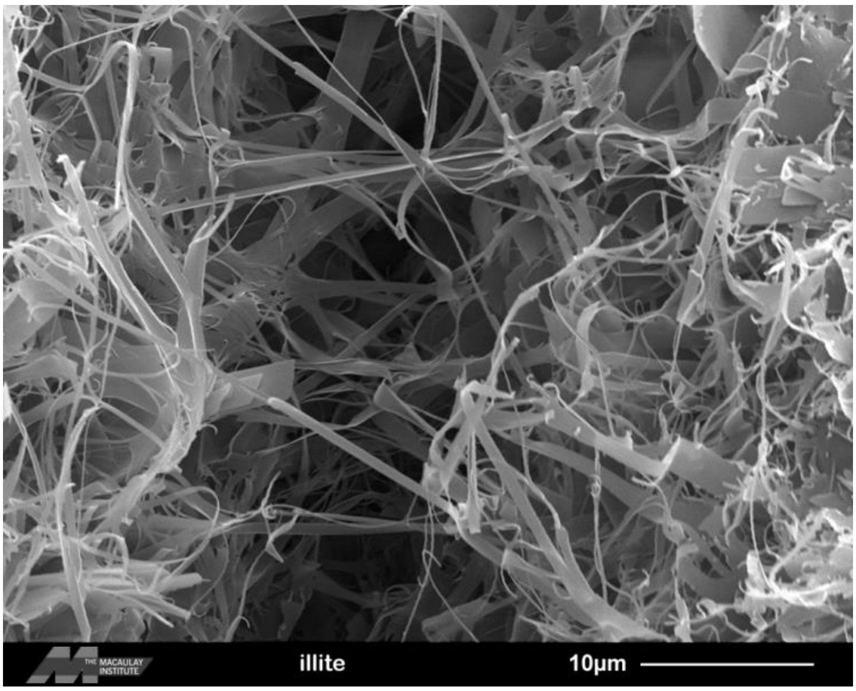

Figure 1.5. SEM of illite (URL-1, 2021).

1.3.1. Montmorillonite

Montmorillonite, shown in figure 1.6c, is similar to illite in structure and the partial substitution of silicon by aluminium in silica sheets. However, unlike illite, the octahedral sheet has a partial substitution of aluminium by iron and magnesium. Furthermore, the combined sheets have a weak bond as a result of water molecules and interchangeable cations besides potassium within sheets.

The cation exchange capacity of montmorillonite is 100 mel/100g and it has a specific surface area of 800m^2/g. Hence, little water absorption can cause significant swelling in soils that contain this type of clay minerals. The SEM of montmorillonite is shown in figure 1.7.

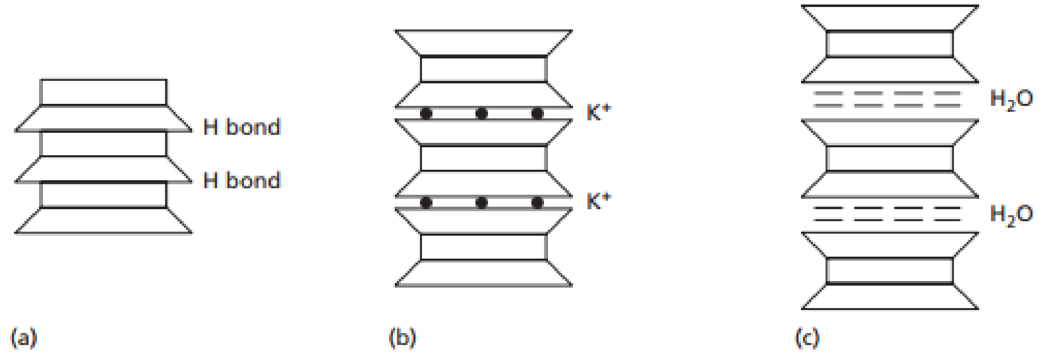

Figure 1.6. Diagrammatic sketch of the structure of; (a) Kaolinite (b) illite and (c) montmorillonite (after Craig, 1997).

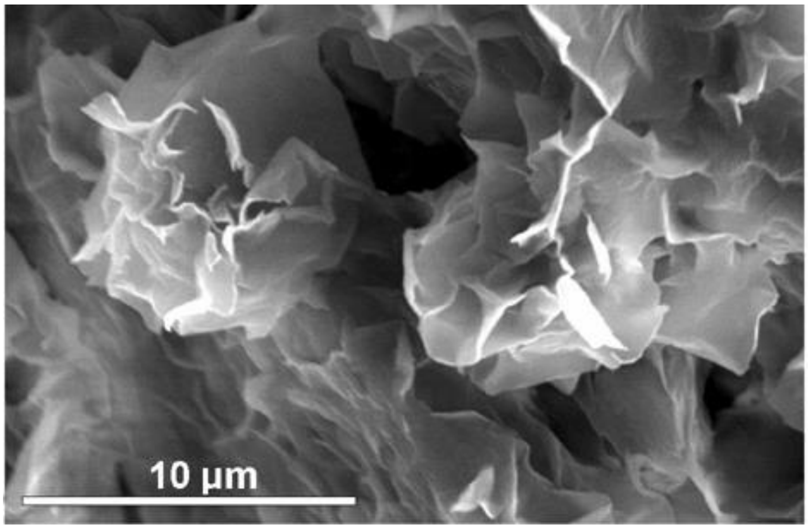

Figure 1.7. SEM of montmorillonite (URL-1, 2021).

1.4. Expansive Soil

Expansive soil, which is also called swelling soil, is a term used to describe any rock material or soil that has swelling or shrinking potential. Expansive soils are found in barren and semi-barren areas of the world and cause challengeable problems for engineers due to their susceptibility to shrinking when dried and expanding when wet.

Expansive soils are a universal challengeable issue that poses different difficulties for geotechnical engineers. The damages of pavements and ground, as shown in figure 1.8, that are caused by expansive soil are more severe than other natural hazards like floods and earthquakes (Jones and Holtz, 1973). The annual costs of damages due to swelling soil are listed in table 1.2.

Table 1.2. Annual cost of damages due to swelling soils (Jones and Holtz, 1973).

Construction Category	**Estimated average cost of damage (millions of dollars)**
Multi-story buildings	80
Highway and streets	1140
Single-family homes	300
Walks, drives, parking areas	110
Commercial buildings	360
Urban landslides	25
Underground utilities and service	100
Airports	40
Others	100
TOTAL	2255

The classification of soil is commonly performed according to two main systems; the American Association of States Highways and Transportation Officials Method (AASHTO) and the Unified Soil Classification System (USCS). Soils with a classification of A7 or A6 by AASHTO and CH or CL CH by USCS tend to swell (Nelson and Miller, 1992). Furthermore, a classification approach according to the particle size and index characteristics of soil is innovated by Holtz, (1969) and Gibbs, (1969) and shown in table 1.3.

Table 1.3 Classification of soil (Holtz, 1969; and Gibbs, 1969).

Clay content (%)	**Plasticity index (%)**	**Shrinkage limit (%)**	**Liquid limit (%)**	**Swelling potential**
> 28	> 35	> 11	> 63	Very high
20-31	25-41	7-12	50-63	High
13-23	15-28	10-16	39-50	Medium
< 15	< 18	< 15	< 39	Low

Figure 1.8. Cracks due to expansive soil (URL-1, 2021).

1.4.1. Factors Affecting Swelling Potential of Expansive soil

The mechanism of swelling is affected by several factors that influence the swell potential of soil. These factors can be sorted into three sets; soil properties, state conditions, and environmental factors. These factors are summarized in Tables 1.4, 1.5 and 1.6 (Nelson and Miller, 1992).

Table 1.4. Soil properties that influence swell potential (Nelson and Miller, 1992).

Factor	**Explanation**
Soil suction	Soil suction, which is also expressed by the negative pore pressure in unsaturated soils, is considered an effective stress variable that is correlated with saturation, shape and pore size, gravity and electrical and chemical features of soil particles and water. Expansion of soil increases by increasing the suction.
Clay mineralogy	Typically, some volume changes in soil are caused due to some clay minerals such as vermiculates, montmorillonites, and some mixed layers. However, other clay minerals, such as illites and kaolinites, are nonexpansive but when fines exist, they may cause volume changes.
Plasticity	The swell potential is affected by the plasticity of soil as it is considered an indicator of soil expansivity. Generally, soils that demonstrate high values of plasticity and liquid limit over wide ranges of moisture content have a great tendency to swell.
Dry density	The increase in the dry density of soils causes an increase in the swelling potential since higher densities usually imply closer particle spacing and higher repulsive forces between particles.
Chemistry of Soil Water	Swelling is inhibited by cation valence and cation concentration increasing. Lower cation valence of the adsorbed cations is associated with more swelling. For example, soil water that contains Na^+ cations would cause more swelling than water that contains Mg^{2+} cations.
Soil Structure and Texture	Dispersed clays have less tendency to be expansive than flocculated clays. Furthermore, cemented particles reduce the swelling. Remoulding or compaction at high moisture content can change the fabric and structure.

Table 1.5. Environmental factors that impact swell potential (Nelson and Miller, 1992).

Factor	**Explanation**
Vegetation	The vegetative cover is very important since it exhausts moisture through transpiration which leads the soil to be dissimilarly moist.
Initial Water Condition	The initial water content affects the swelling potential since soils having low water content would gain water more than wet soils. Conversely, expansive soil with high water content would have lower suction or little susceptibility for water than the same soil with lower water content and higher suction. For this reason, the initial suction of soil has to be taken into consideration with the predicted conditions of final suction.
Climate	The moisture availability is considerably influenced by the variety and amount of rainfall and evapotranspiration. Therefore, semiarid and arid regions have the greatest seasonal heave and have expansive soils at most due to the short wet seasons.
Moisture Variations	The moisture variation and volume change occur in the active zone, and they are very important factors that affect the swelling potential.
Temperature	As temperature increases, moisture increases. This causes swelling especially under buildings and pavements.
Drainage	Poor surface drainage from water sources and leakage in plumbing causes an excess in the moisture content of the soil.
Permeability	The accessibility of water to the soil is increases with higher permeability in soils which leads to higher swelling potential.

Table 1.6. Stress conditions that influence swell potential (Nelson and Miller, 1992).

Factor	**Explanation**
Loading	The percent of swelling decreases if the surcharge load increases on the swelling soil. The soil will not expand if the external load equals or exceeds the swelling pressure.
Stress History	At the same void ratio for the same soil, the overconsolidated soil has more swelling potential than the normally consolidated soil. Repeating wetting and drying of soil is efficient for decreasing the expansion potential of soil samples up to a particular number of wetting-drying cycles beyond which the soil expansion will no longer be affected.
Soil profile	Possible movements are extremely affected by the location and thickness of swelling layers in the soil profile. If the expansive clays at the surface goes under the active zone, substantial movements happen. Conversely, lesser movements would occur if bedrock or non-expansive material overlies the expansive soil.
In situ Conditions	Possible movements are extremely affected by the location and thickness of swelling layers in the soil profile. If the expansive clays at the surface goes under the active zone, substantial movements happen. Conversely, lesser movements would occur if bedrock or non-expansive material overlies the expansive soil.

1.4.2. Identification of Expansive Soil

Identification of expansive soil is essential during the preliminary stages and surveying works of site investigation to determine the appropriate test methods and suitable sampling to be carried out. The identification methods can be categorized into two groups to determine the expansion potential of soils. The first group includes the determination of the physical characteristics of soils, such as free swell, potential volume change and consistency limits. The second group involves the measurement of the chemical and mineralogical properties of soils, such as cation exchange capacity, clay content and specific surface area. Usually, geotechnical engineers only rely on the measurement of the physical properties. On the other hand, geological and agricultural practitioners routinely depend on the mineralogical and chemical properties.

1.4.2.1. Methods Based on Physical Properties

1.4.2.1.1. Methods Based on Plasticity

Until now, there has been no simple universal method for identifying the swelling soil. Thus, the consistency limits are among the primary properties and the most common methods to forecast swell performance of expansive soils. Various research works have been carried out to find the expansion potential using the index characteristics of soils. Some classifications of soil expansivity based on the plasticity index and liquid limit are listed in table 1.7 and table 1.8, respectively.

Table 1.7. Relationship between the expansion potential and the plasticity index of a soil

Expansion Potential	**Plasticity Index (%)**			
	Holtz and Gibbs, 1956	Chen, 1975	Peck et. al., 1974	Indian Standard (IS 1498), 1970
Very high	> 32	> 35	> 35	> 32
High	23-45	20-55	20-55	23-32
Medium	12-34	10-35	0-35	12-23
Low	< 20	0-15	0-15	< 12

Skempton, (1953) defined activity, as illustrated in equation (1.1), as a parameter combines the clay content with Atterberg limits. He classified the clays into three categories depending on their activity. The first category is inactive clays which have activity less than 0.75. The second category is normal clays whose activity ranges between 0.75 and 1.25. The third category is active clays with activity greater than 1.25 which have the most swelling potential. Activity values for some clay minerals are shown in table 1.9.

$$\text{Activity (Ac)} = \frac{\text{Plasticity index}}{\text{\%by weight finer than 2}\mu\text{m}} \qquad (1.1)$$

Table 1.8. Relationship between liquid limit and soil expansivity

Degree of expansion	**Liquid limit (%)**		
	Indian Standard (IS 1498), 1970	Chen, 1975	Dakshanamurthy and Raman, 1973
Very high	70-90	> 60	> 70
High	50-70	40-60	50-70
Medium	35-50	30-40	35-50
Low	20-35	< 30	20-35

Table 1.9. Activity values for some clay minerals (Skempton, 1953).

Mineral	**Activity**
Montmorillonite (Na)	7.2
Montmorillonite (Ca)	1.5
Kaolinite	0.33-0.46
Illite	0.9

1.4.2.1.2. Free Swell Tests

The free swell test is known as the ratio of the final volume to the initial volume of a specific dried volume of soils passing sieve No. 40 and completely settled in a graduated cylinder filled with water. Equation (1.2) is used in the free swell test. According to Holtz and Gibbs (1956), the free swell value of commercial bentonite free has been found to vary between 1200-2000%. Moreover, a considerable expansion might be exhibited in soils that have 100% free swell when moistened under light pressure in the field. For soils having less than 50% free swell, a considerable volume change may not be encountered. However, Dawson (1953) stated that considerable damages in Texas have been caused for clays that have around 50% free swell.

$FS = 100*(V-V_o)/V_o$ (1.2)

Where

FS = free swell, %,

V = the soil volume after swelling, cm^3, and

V_o = the volume of dry soil, 10 cm^3.

Due to the difficulty of determining an exact volume of 10 cm^3 of soil sample, the differential free swell test is recommended. In this test, two oven-dried soil samples of 10g and passing through sieve No. 40 are taken. The first sample poured into a 100-mL graduated cylinder containing distilled water and the other is poured into a 100-mL graduated cylinder containing carbon tetrachloride (CCl_4) or kerosene. The final volume of each sample is recorded and the differential free swell index is calculated by equation 1.3. In addition, based on free swell index, soil expansivity according the expansion potential of the soil is shown in table 1.10.

$FSI = 100*(V_w-V_k)/V_k$ (1.2)

Where

FSI = differential free swell index, %,

V_w = the final volume of the soil in distilled water,

V_k = the final volume of the soil sample in CCl_4 or kerosene.

Table 1.10. Expansion Potential Based on FSI

FSI, %	**Expansion Potential**
> 50	Very high
35-50	High
20-35	Medium
< 20	Low

1.4.2.2. Mineralogical Methods

Potentially expansive soils are typically identified through the clay mineralogy especially the presence of montmorillonite. Electron microscopy, differential thermal analysis (DTA) and X-ray diffraction (XRD) are the most common identification mineralogical methods. DTA depends on heating an inert substance and a clay sample simultaneously. XRD is based on the concept of measuring basal plane spacing by the amount of X-ray diffraction around the crystals. The obtained thermographs, which are graphs of applied heat versus temperature difference, are compared to those of pure minerals. Each mineral shows individual exothermic and endothermic reactions on the thermograms.

1.4.3. Measuring Swell Parameters Using Oedometer Methods

The one-dimensional oedometer or consolidometer is one of the most frequently used apparatus for the prediction of the swelling parameters of expansive soils. Based on ASTM D4546-21, the methods for swelling parameters forecast can be classified into three categories; Method A, Method B and Method C which are summarized in sections 1.4.3.1-1.4.3.3.

1.4.3.1. Method A

This method (shown in figure 1.10), which is called wetting-after-loading tests on multiple samples, is conducted by assembling four or more similar natural samples from the field or prepared by compaction in the laboratory. Afterwards, the samples are subjected to different stresses that cover the range of in-situ pressures in addition to any stresses that might occur due to additional constructions. After applying stresses and reading the settlement caused by stresses (Δh_1), samples are immediately inundated with water. While samples are collapsing or swelling, deformations are taken at 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes and 1, 2, 4, 8, 24, 48, and 72 hours until primary swell or collapse (Δh_2) is finished as shown in figure 1.9. After swelling or collapsing is completed, the final deformation and final moisture content must be measured.

The minimum vertical pressure needed to prevent swelling which is called swelling pressure, and the free swell magnitude which is the swell strain corresponding to near-zero pressure of 1 kPa can be found from the results or graph of this test.

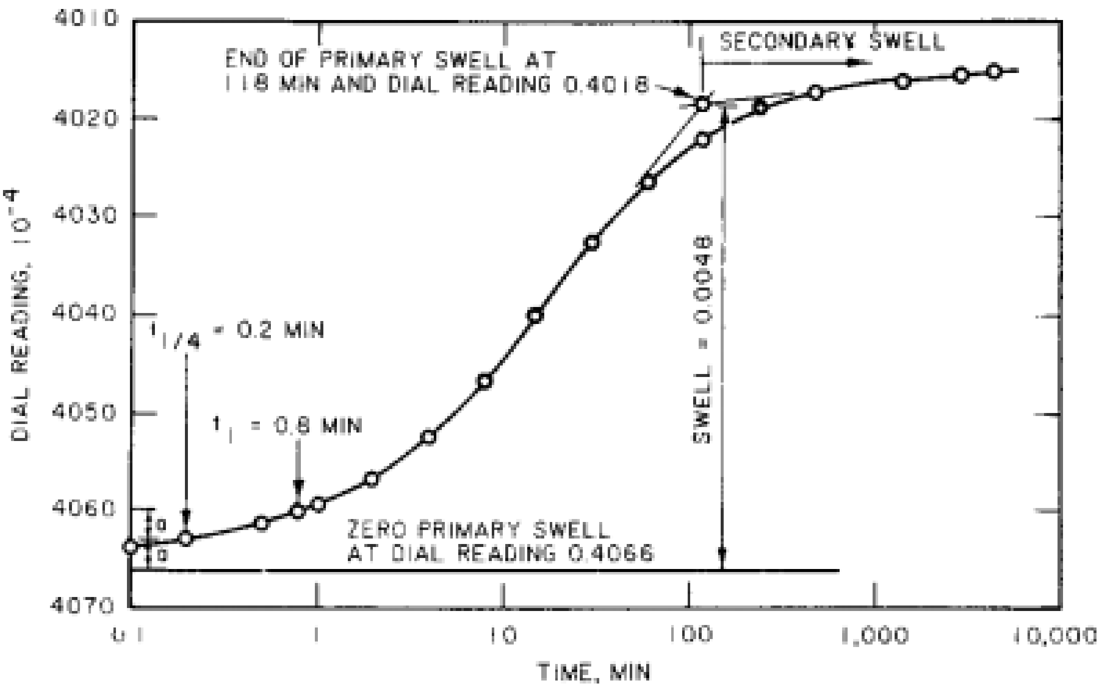

Figure 1.9. Swell-time curve (ASTM D4546, 2021).

1.4.3.1. Method B

This method (shown in figure 1.11), which is called the single-point wetting-after-loading test on a single sample, is carried out by assembling a natural sample from the field or prepared by compaction in the laboratory. Subsequently, the sample is subjected to a stress equal to 1 kPa, any particular design stress, or the in-situ pressure for estimating the swell strain. After loading, the sample is unloaded, loaded again, inundated with water and the deformations are recorded in a similar manner as of method A to find the swell strain.

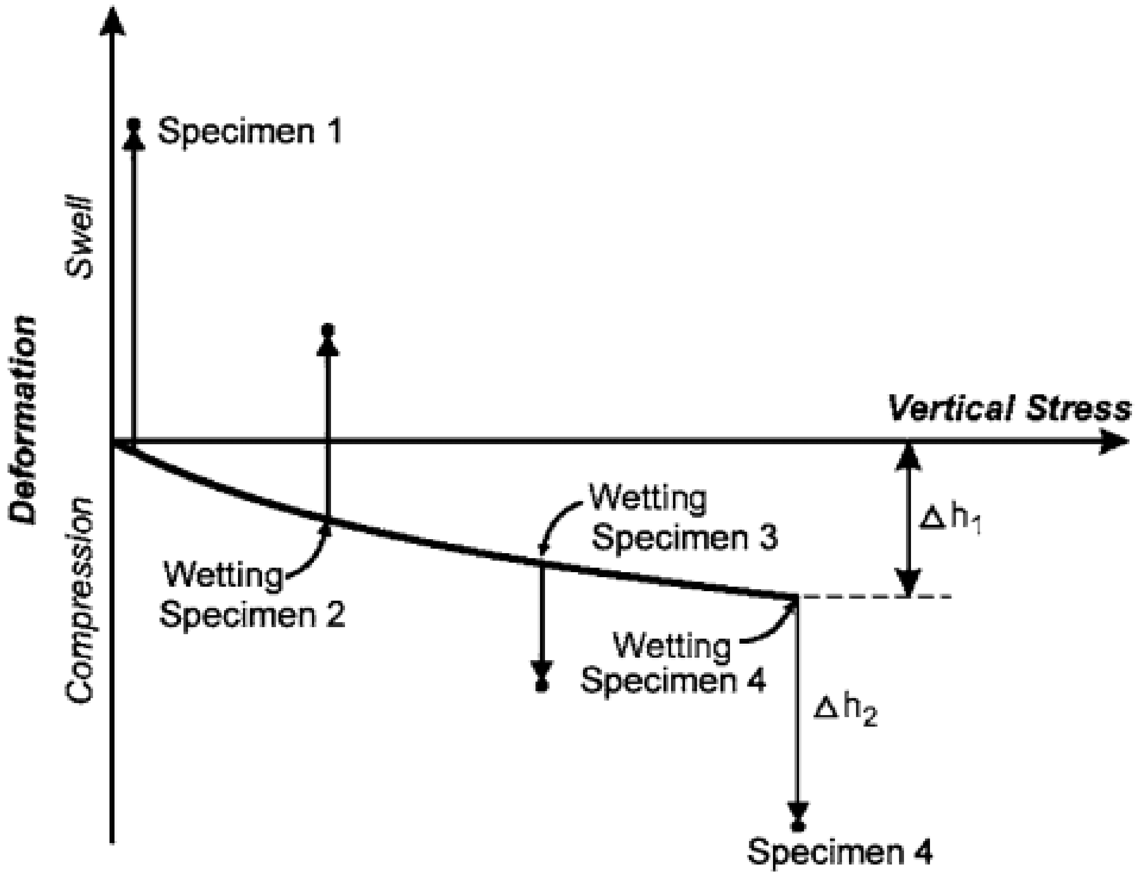

Figure 1.10. Vertical stress vs. deformation, method A (ASTM D4546, 2021).

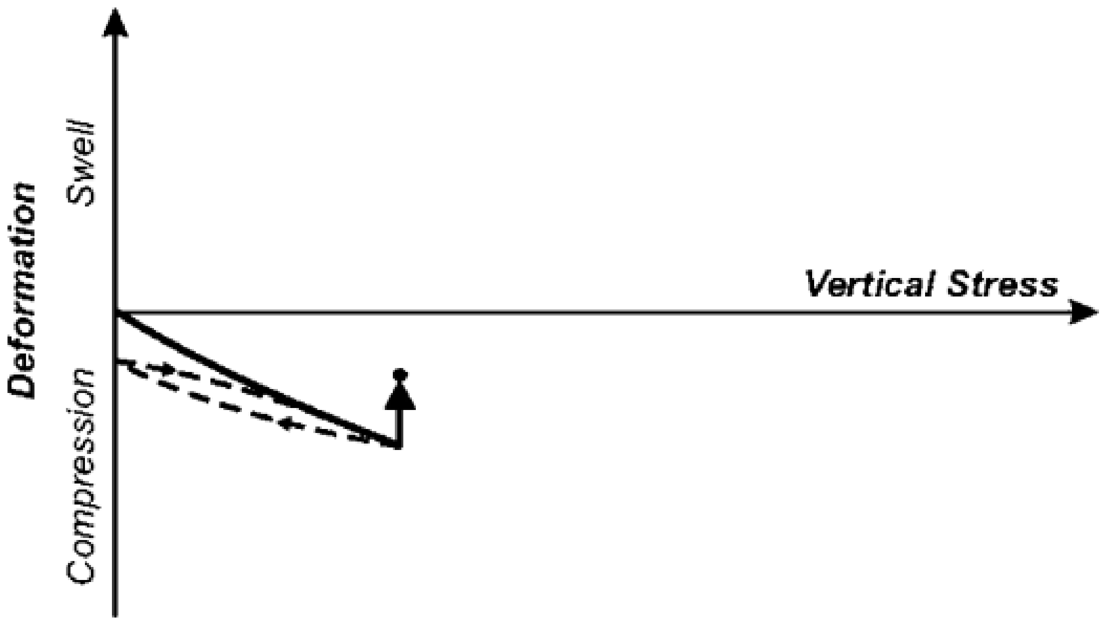

Figure 1.11. Vertical stress vs. deformation, single-point test method B (ASTM D4546, 2021).

1.4.3.2. Method C

Method C can be seen in figure 1.12. It is called loading-after-wetting tests, and it includes taking a natural sample from the field or preparing a sample by compaction in the laboratory. A stress equal to in-situ pressure is then applied to the sample and the rest of the test is performed similar to Method A. After the completion of the swell, the specimen is subjected to additional loads increased with time increments similar to the consolidation test (ASTM D2435, 2020) to find the swelling pressure which is the pressure that required the specimen to return to its initial height.

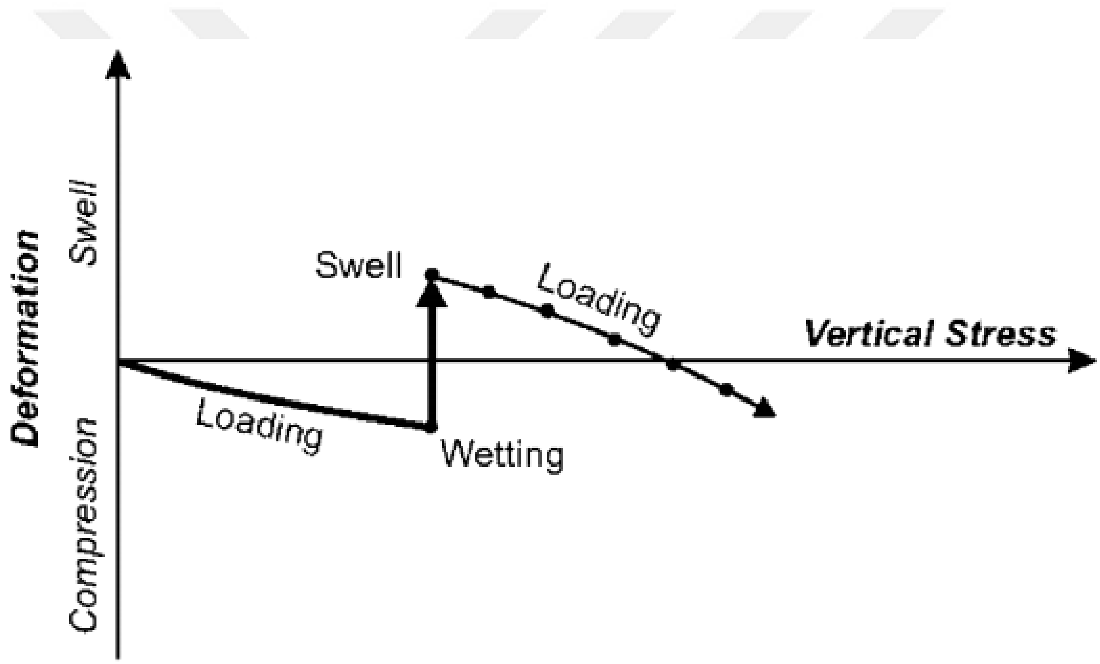

Figure 1.12. Vertical stress vs. deformation, loading-after-wetting test method C (ASTM D4546, 2021).

1.5. Treatment of Expansive Soils

Stabilization is a term used to express the required changes in permeability or strength properties (or both) for soils or rocks for construction (Karol, 2003). It is a process for improving the soil properties in order to enhance the soil for purposed engineering use (Winterkorn, 1955). As a branch of soil science, it has evolved for 40 years (Kezdi, 1979).

Improving soil strength and reducing permeability and water absorption are the major objectives of soil stabilization. Other objectives include improving bearing capacity and durability under conditions like repeated applications of stress at amplitudes less than the soil ultimate strength-fatigue life or changing moisture content (Gillott, 1987).

There are several treatment options to improve expansive soils either before or after the construction. These procedures are listed below (Nelson and Miller, 1992):

• Chemical additives.

• Surcharge loading.

• Soil removal with compaction.

• Moisture control.

• Thermal methods.

• Prewetting.

In general, choosing an appropriate treatment procedure is not a straightforward approach. A preliminary investigation of the soil characteristics must be performed to select the suitable method.

1.5.1. Chemical Additives

Chemical soil stabilization is aimed to adjust the soil-water interactions by surface reactions to make the behaviour of the soil with respect to water effects most convenient for a particular purpose. In such stabilization, the most important roles are played by the polarity of the surfaces and their water imbibition, the soil particles' surface activity, and the entire adsorption (Kezdi, 1979).

There are two types of chemical admixtures for improving soils to meet engineering requirements: the natural admixtures which are referred to as granular stabilizers and the manufactured admixtures which are very popular because of the wide availability, environmental acceptability, and their applicability to a wide range of soils (Hausmann, 1990).

The most common manufactured admixtures are fly ash, lime, bitumen, and Portland cement (Hausmann, 1990; Winterkorn and Pamukcu, 1991).

Besides the utilization of lime for stabilization of expansive soils, many chemicals, either organic or inorganic, are also used. Fly ash and cement have achieved successful results in laboratory tests. Definitely, the cost of cement is substantially high compared with that of lime. Furthermore, adding fly ash to the soil-lime mixture sometimes increase pozzolanic reaction (Chen, 1975).

Mixing additives with soils is done either in a stationary mixing plant or at site by using a traveling plant mixer to mix and compact over the soil intended for treatment. However, in a stationary mixing plant the soil is mixed with the additive, transported to the required construction site, spread, and finally compacted (Gillott, 1987).

1.5.1.1. Lime Stabilization

Lime is well recognized as one of the most antiquated chemical additives used in the stabilization of many aggregates and soils which have efficiently reduced swelling and enhanced soil workability and plasticity for many projects. Generally, lime is added at ranges of 3%-8% is to the top layers of the soil (McDowell, 1965; Teng et. al., 1973). Stabilization utilizing lime has been widely used to reduce swelling pressure, plastic properties and swelling potential, and to increase the strength of soils. The type and amount of lime, curing conditions and type of soil are the primary characteristics that affect the strength of lime-treated soil.

The efficiency of lime stabilization is affected by the factors summarized below (Thompson, 1966):

- The required amount of lime might increase for soils that contain gypsum or ammonium fertilizers.
- An effective reaction is allowed to occur for soils with a pH of more than 7.
- Reactions of lime-soil mixtures are inhibited by the presence of organic carbon, some iron or sulfates.
- A good reactivity occurs for calcareous soils.
- The reactivity of lime is tended to be less for soils with poor drainage conditions.

The chemical reactions for lime included complex reactions (Thompson, 1966; 1968). Flocculation and agglomeration, cation exchange and pozzolanic reaction are the main reactions of lime stabilization (Nelson and Miller, 1992).

Adding lime to clay increases the number of cations such as magnesium ions (Mg^{+2}) and calcium ions (Ca^{+2}) that replace the cation of clays like potassium (K^{+}) and sodium (Na^{+}) and this is a reaction known as cation exchange, in which the cations of lower valence are replaced by cations of higher valence. A typical series of cation replacement is shown below;

$$Na^{+} < Li^{+} < K^{+} < Rb^{+} < Cs^{+} < Mg^{+2} < Ca^{+2} < Ba^{+2} < Cu^{+2} < Al^{+3} < Fe^{+3} < Th^{+4}$$

The following is an example of a cation exchange reaction:

$$(Ca^{+2}) + (Na^{+}\text{-Clay}) \rightarrow (Ca^{+2}\text{-Clay}) + (Na^{+})$$

The texture of the clay is changed to granular material due to the flocculation and agglomeration. Furthermore, a decrease in the quantity of clay-sized particles and agglomeration and flocculation of clay particles, which reduce the plasticity of soil, has resulted from cation exchange reactions (Terzaghi and Peck, 1967).

In addition, the stabilization of soil is improved and calcium aluminate hydrate (CAH) and calcium silicate hydrate (CSH) are produced, as shown below (Show et al., 2003), due to pozzolanic relations which are time-dependent.

$$(Ca^{+2}) + 2(OH)^{-} + Al_2O_3 \text{ (Clay Alumina)} \rightarrow CAH$$

$$(Ca^{+2}) + 2(OH)^{-} + SiO_2 \text{ (Clay Silica)} \rightarrow CSH$$

Based on the impacts of lime-soil reaction, there are several advantages that can be summarized as follows:

- The plasticity of soil, swelling potential, and the liquid limit are reduced.
- The workability of soil is increased.
- The shrinkage limit is increased.
- The strength of the soil is improved.
- The exchange capacity and pH of soil are raised.

1.5.1.2. Cement Stabilization

Cement has commonly been adopted for efficiently stabilizing an extensive assortment of soils such as clays, silts and granular materials. Hydration of cement is considered a complicated pozzolanic reaction which releases a set of diverse gels and compounds. The results of blending cement with clays are are similar to the case of lime. Thus, it causes an increase in shear strength and a decrease in plasticity index, liquid limit and swell potential (Chen, 1988). However, stabilization using cement is considered less efficient than lime in clays with high plasticity because the pozzolanic reaction may not complete due to the less sufficiency of cement hydration in clays with a high affinity for water (Mitchell and Raad, 1973).

Stabilization using cement for swelling soils or soft clays includes blending soils with water and cement (figure 1.13). The mixture is then compacted to a high density in order to increase the resistance to alterations in stresses (Winterkorn and Pamukcu, 1991). In general, using cement at ranges 2%-6% is enough for soil to behave like a semi-rigid slab.

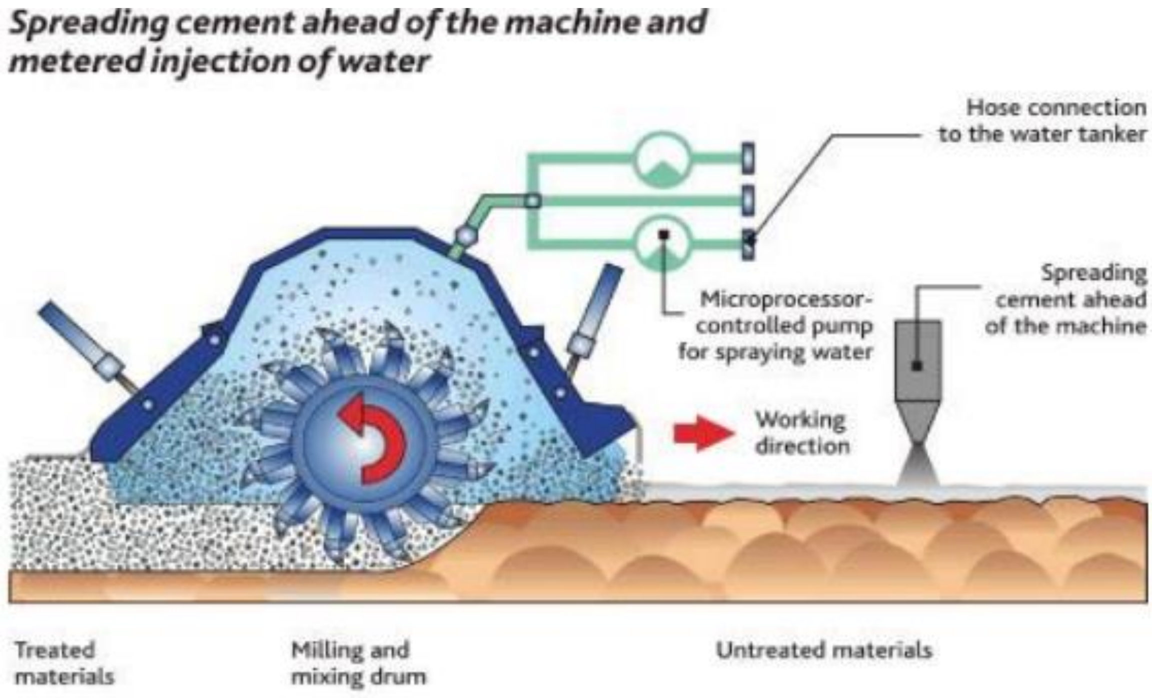

Figure 1.13. Cement stabilization

1.5.2. Surcharge Loading

Swelling can be effectively restricted only in soils that have low to medium swelling pressure through loading the soil with a load greater than the expected swelling pressure. This method would be less effective in soils with high swell pressure due to the nonlinear relationship between swelling and pressure and would not be restrained by overburdening pressure alone. For example, soils that have up to 25kPa swelling pressure might be restrained by 1.3m of filling material topped with foundation. However, several types of soils have high swelling pressure.

1.5.3. Soil Removal and Replacement

Providing stable material for pavements and foundations by replacing the expansive soils with nonexpansive soil such as crushed rock, sand, and gravel is an old procedure. In order to apply this method, the depth of expansive layers has to be determined in order to define the depth of fill and excavation to restrict swelling (Nelson and Miller, 1992).

The backfill depth has to provide enough pressure to resist the predicted uplift pressures. No specific research has been developed for defining this depth but a minimum depth of 1 to 1.3m is recommended by Chen, (1988).

There are many advantages for applying the removal and replacement method such as increasing the bearing capacity of soil due to the ability to compact the nonexpansive soils at densities higher than the other methods. Besides, it is more economical procedure compared to other stabilization methods. Furthermore, it avoids the delay of construction, contrast to other methods. However, the removal and replacement process might be inapplicable due to the requirement of the large thickness of the backfill (Nelson and Miller, 1992).

1.5.4. Prewetting

Prewetting is a method that aims to increase the moisture in swelling soils which leads the swell to occur before construction and this would inhibit swelling hazards afterward.

Nevertheless, this method may have severe obstacles that restrain its application since the required time of this process is long and might last for up to many years because the hydraulic conductivity is typically low for expansive soils. Moreover, the bearing capacity and slope stability of soils can be reduced after submerging the soil in water for long time (Nelson & Miller, 1992).

1.6. Literature Review

Stabilization of soil has become an inevitable application for improving expansive soils. Therefore, stabilization of soils techniques utilizing chemicals such as natural pozzolana and pumice (Çelik 2020; Harichane et. al. 2019; Harichane et. al. 2018), using cement kiln dust (Eberemu et. al. 2019; Yılmaz 2014), rice husk ash or fly ash (Phanikumar & Sharma 2004; Zha et. al. 2008; Liu et al., 2019) and utilizing glass powder (Ibrahim et. al 2019; Blayi et. al. 2020) have been widely investigated in the last decades.

Atom and Al-Sharif (1998) used the waste ash that they obtained by burning the olive oil industry waste product at 550ºC as an additive in soil stabilization studies. The mixtures prepared with four different ratios of ash waste material: 0%, 2.5%, 5%, and 7.5% added to 4 different soils, were cured at room temperature and humidity conditions for 72 hours. Afterwards, Atterberg limits, compressive strength and free swelling percentages were examined. For all soils, the plasticity of the mixtures decreased with the increased use of waste ash. The highest compressive strength was obtained with the use of 2.5% waste ash. Higher ratios of waste ash caused a decrease in the compressive strength. While ratios of ash waste material were increased, the free swelling percentages decreased, reaching the lowest value at 7.5% addition of waste material. The reason for these improvements was the non-plastic nature of the waste material ash and the CaO in its content.

Başer, (2009) used waste dolomitic marble dust (DMD) and limestone dust (LD), which passed sieve #40, for stabilization of prepared expansive soil in the laboratory. The expansive soil was prepared by mixing 15% bentonite with 85% kaolinite. Seven different ratios (0%, 5%, 10%, 15%, 20%, 25%, and 30%) of DMD and LD were added to the prepared soil. Grain size distribution, consistency limits, specific gravity and free swell ratio were determined. In addition, swelling percentages for the mixtures were determined without curing and with curing for 7 and 28 days. While plasticity index and liquid limit of mixtures decreased by raising the percentage of additive, an increase in the plastic limit was observed. The maximum reduction of liquid limit and plasticity occurred when 30% of limestone dust was used. Free swell slightly decreased upon increasing the percentage of additive up to 20% then a considerable decrease started to occur at increasing the percentages. Moreover, swell percentage decreased considerably with the addition of additives after curing especially in samples containing limestone dust. Even with 5% addition of LD, 37% reduction in swelling percentage was achievable with 28 days of curing.

Okafor and Okonkwo (2009) investigated the treatment of a soil classified as A-2-6 according to AASHTO classification with 5%, 7.5%, 10%, and 12.5% of rice husk ash content, and studied its effects on compaction parameters, consistency limits and strength properties. When rice husk ash ratio was increased, the maximum dry densities of the mixtures decreased, whereas the optimum water content values increased. Moreover, as the amount of rice husk ash was increased, the plasticity of the mixtures decreased, whereas their strength and volumetric stability increased. Considering all the test results, it was determined that the optimum rice husk ash ratio is 10%.

Harichane et. al. (2010) investigated the treatment of two types of clayey soil, CL and CH soil according to the USCS classification system, with pozzolana and lime, and they studied the effect of periodic wetting and drying for 12 times on the behaviour of compressive strength. They added pozzolana or lime and combinations of them to clayey soils at ranges of 0-20% and 0-8%, respectively. They noticed an increase in the optimum moisture content and a decrease in the maximum dry density with the increasing of lime-natural pozzolana combination percentage in CL-class soil. On the other hand, an increase in the maximum dry density and a decrease in the optimum water content were observed when the percentage of lime-natural pozzolana combination increased in CH-class soil. An increase in the unconfined compressive strength was also observed after increasing the percentage of lime-pozzolana mix and curing time in both soils. According to durability results, the stabilized soils by the natural pozzolana-lime combination showed a great performance and endured a full 12 cycles of wet-dry testing. In addition, it was concluded that the durability of soils can be influentially improved from poor to excellent by lime-natural pozzolana mix.

Harichane et. al. (2011) investigated the utilization of natural pozzolana combined with lime to improve the properties of clayey soils. Two types of soils, CL-class and CH-class soils, collected from a site in West of Algeria at a depth of about 4-5m underground. A series of laboratory tests were conducted to study the effect of adding natural pozzolana with lime on the geotechnical properties of cohesive soils. Lime or natural pozzolana were added to soils at two stages; in the first stage, natural pozzolana was added alone to soils at percentages of 0, 10 and 20%. Lime was also added alone to soils at percentages of 0, 4 and 8%. In the second stage, natural pozzolana was mixed with lime at the same ranges then added to soils. Compaction, consistency and undrained triaxial shear tests were performed. Moreover, unconfined compressive strength tests on treated samples with curing for 1, 7, 28 and 90days were conducted. Corresponding to the obtained results, the plasticity index was enhanced with increasing lime percentage and the plasticity behaviour of soils markedly changed when both natural pozzolana and lime were mixed with soils. Either using lime alone or natural pozzolana-lime mix transformed both soils into MH-class soil. It was also concluded that increasing lime content led to increasing the maximum dry density and decreasing the optimum water content in contrast with the addition of natural pozzolana. Furthermore, higher shear strengths and unconfined compressive strengths were obtained for samples treated with a combination of natural pozzolana-lime compared to those treated with either natural pozzolana or lime alone. In addition, the unconfined compressive strength considerably improved when curing time was extended.

Harichane et. al. (2012) investigated the effect of utilizing natural pozzolana, lime or a combination of them on two types of soft soils, CL and CH soil according to the USCS classification system. Natural pozzolana was added to soft soils at ranges of 0-20% while lime was added at ranges of 0-10% in addition to combinations of them at the same ranges. An experimental program of shear tests, compaction tests and unconfined compression tests was conducted with curing lasted for 1, 7, 28 and 90 days for unconfined compression tests. Based on the obtained results, the maximum dry density decreased and the optimum water content increased with the increase in the lime percentage of lime-stabilized soils, in contrast with natural pozzolana-stabilized soils. The increase in natural pozzolana-lime mix percentage increased the optimum moisture content and decreased the maximum dry density in CH-class soil in contrast with CL-class soil. Moreover, the addition of lime content considerably enhanced the shear parameters in both soils while the addition of natural pozzolana vaguely increased shear parameters in CL-class soil and reduced them in CH-class soil. Besides, increasing curing time and stabilizer content increased the unconfined compressive strength. It was clear that natural pozzolana-lime mixed effectively enhanced the UCS more than using natural pozzolana or lime alone.

Sabat, (2012) studied the effect of employing waste ceramic dust on the plastic limit, liquid limit, plasticity index, California bearing ratio, compaction characteristics, shear strength parameters, swelling pressure and unconfined compressive strength of expansive soil. Ceramic dust was mixed with locally collected expansive soil at a range of 0-30% by an increment of 5%. Based on the collected results, plasticity index, plastic limit and liquid limit decreased from 32% to 15%, 30% to 20%, and 62% to 35%, respectively, with the increase of ceramic dust content from 0% to 30%. Besides, increasing ceramic dust percentage decreased the optimum water content, cohesion and swelling pressure in contrast with the angle of internal friction, soaked California bearing ratio, maximum dry density and unconfined compressive strength which increased after increasing ceramic dust content.

Calik and Sadoglu, (2014) experimentally investigated the effect of adopting perlite and lime on the stabilization of high plasticity clayey soil. Perlite was used at ranges of 0-50% by an increment of 10% with 8% and 0% lime. A series of laboratory tests were conducted such as unconfined compressive strength, Atterberg limits, compaction, free swelling potential, and swelling pressure. All test samples were cured for 1, 7, 14, 28, and 84 days. The collected results showed a linearly decreasing plasticity index and liquid limit when the percentage of perlite increased, whereas no vital change was noted in these values with time. However, adding lime with perlite increased the liquid limit but decreased the plasticity index until the 14^{th} day of curing period then it slightly jumped. In addition, increasing perlite ratio in either perlite-lime or perlite stabilizer increased the maximum dry density but it was larger for mixtures which contain perlite alone. Results also showed a decrease in swell pressure and swell percent through increasing perlite ratio in both perlite and perlite-lime mixtures but it was smaller for mixtures which contain lime. Moreover, the mixture containing 8% lime and 30% perlite achieved the maximum value of unconfined compressive strength.

Al-Swaidani et al. (2016) investigated the influence of the addition of natural pozzolana on several geotechnical characteristics of lime-treated clayey soils. They added lime and natural pozzolana to soil within the range of 0-8% and 0-20%, respectively. They conducted several laboratory tests including Atterberg limits, CBR, linear shrinkage, and compaction. They reported that considerable enhance can occur when adding natural pozzolana as a stabilizing agent to lime-treated clayey soils. The highest reduction of PI took place when a mix of 8% lime and 20% natural pozzolana was used. However, using more than 4% lime had a marginal influence on reducing the PI. Similarly, utilizing 20% natural pozzolana with 8% lime exhibited the best enhancement in CBR value of 90%. Despite that, adding lime alone is more effectual than natural pozzolana. Moreover, it was clear that treatments using lime were more efficient in decreasing shrinkage strains than using only natural pozzolana. However, using lime and natural pozzolana together at percentages of 8% and 20%, respectively, achieved the lowest shrinkage strain.

In the study presented by Indiramma et al. (2020), the effect of using lime and fly ash on the geotechnical parameters of expansive soil was investigated. Individual soil admixtures of 4% and 8% of lime, and combined soil admixtures of 10% fly ash with 4% and 8% lime were prepared to study the effects on differential free swell index, plastic limit, liquid limit, plasticity index, compaction characteristics and unconfined compressive strength. They observed that the strength, plastic limit and maximum dry density of soil admixtures are increased, whereas the differential free swell index, plasticity index, liquid limit, and optimum moisture content decreased.

Referring to the research undertaken by Parihar & Gupta, (2021), liming leather waste ash (LLWA) was utilized as a stabilizer for expansive soil to examine its influence on several engineering features of swelling soil such as unconfined compressive strength (UCS), consistency limits, California bearing ratio (CBR), compaction parameters, and swell parameters. LLWA was mixed with black cotton soil (BCS), collected from a depth of 0.5, at ranges 0-10% by an increment of 2%. Using 6% LLWA increased shrinkage limit and decreased plastic limit by 67% and 70%, respectively. In addition, swell pressure and swell potential of the soil decreased by 85% and 70%, respectively, at a ratio of 8% LLWA. Moreover, increasing stabilizer content and curing time improved the CBR and UCS values by 387% and 278%, respectively. Therefore, 6-8% of LLWA content could be considered as optimum ratios of liming leather waste ash.

2. EXPERIMENTAL WORK AND MATERIALS

2.1. Materials

The properties and information about natural soil and additives such as lime, marble and pumice used material in this study are summarized in this section.

2.1.1. Soil

The soil was collected from a location near the Denizbükü neighbourhood at Üney town at Ordu city in Turkey from a depth of one meter. Views and location where soil collected are shown in figures 2.1 and 2.2, respectively. The mineralogical properties of soil, the physical properties, consistency limits, permeability, compressive strength, compaction and swelling properties and classification of soil were specified based on tests carried out in-site and in the laboratory.

Figure 2.1. Views of the place where soil was collected

Figure 2.2. Location of the place where soil was taken

2.1.1.1. Mineralogical Properties

In order to determine the mineralogical properties of clay in the soil, differential thermal analysis (DTA) and X-ray analysis were performed. The differential thermal analysis is used in the control of substances that show characteristic changes under the influence of temperature. In the DTA analysis, the temperature differences between the material to be examined and an inert material that does not contain water are measured under the same thermal conditions. With the formation of a temperature difference such as in a chemical reaction and phase change in the material being studied, peaks are formed in the curve drawn and the type of mineral is determined. For the current study, the sample was placed in the device and heated up to 1000°C. According to the results of the analysis made in the Laboratory of the Physics Department at Karadeniz Technical University (figure 2.3), it was determined that the dominant clay mineral in the natural soil sample was montmorillonite.

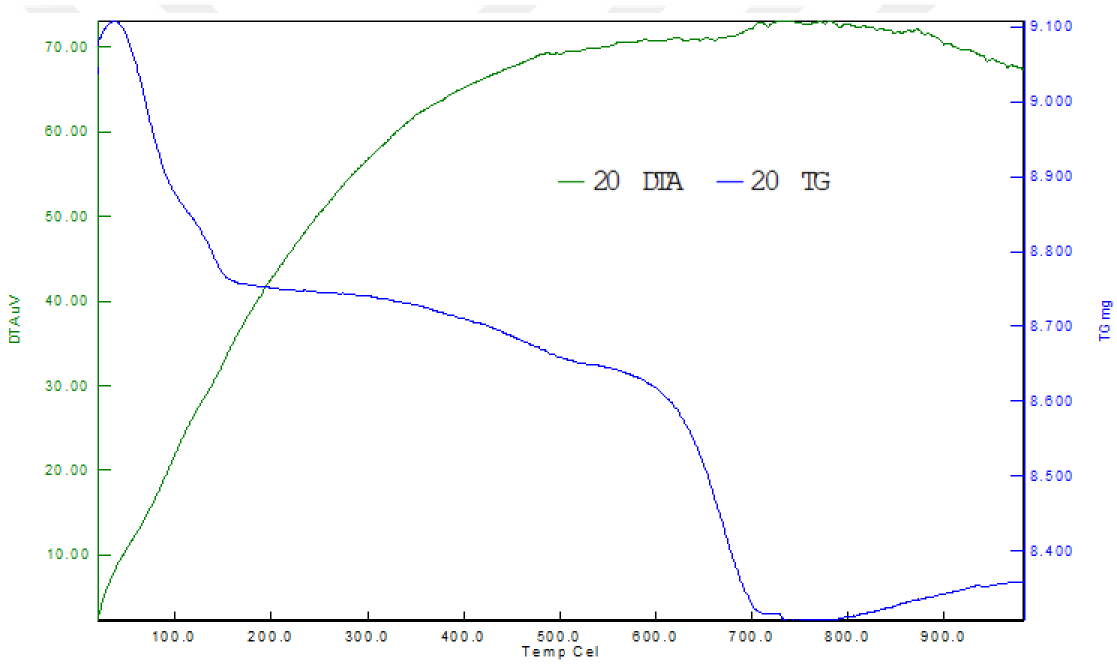

Figure 2.3. DTA analysis of soil

Another type of analysis to determine the mineralogical structure of a natural soil sample is the X-ray analysis. In this test the soil sample, which was ground and sieved through sieve no.200, was placed between the glass plates in a fixed source that sends X-rays. The angle at which it strikes any atom is reported while the sample is rotated at a constant speed. The test was carried out according to ASTM D934 (2013) in the Physics Department Laboratory at Karadeniz Technical University. Based on the results of the analysis performed in continuous scanning in the range of $2\theta = 5\text{-}40°$ (figure 2.4), it was determined that smectite was the dominant clay mineral in the soil sample. Referring to the same figure, illite and mixed-bedded clay minerals can also be seen.

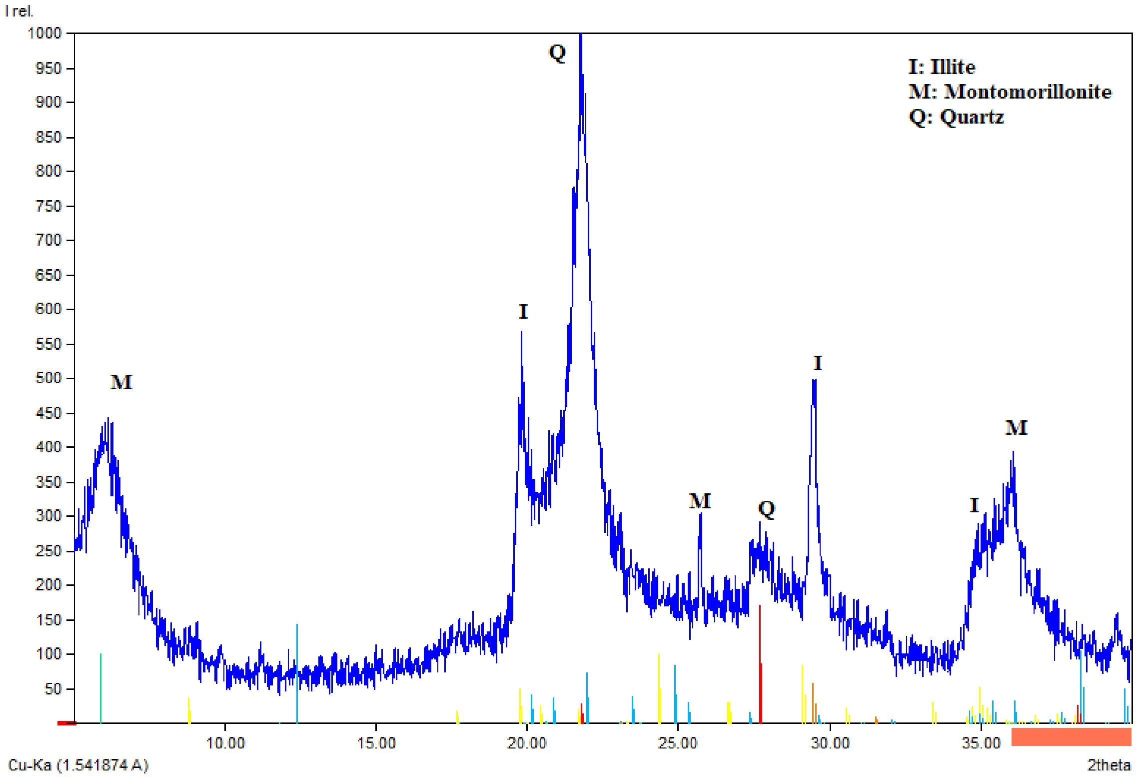

Figure 2.4. X-ray analysis of soil

2.1.1.2. Physical Properties

The physical properties of soil are shown in table 2.1. The unit weight of soil was determined according to the direct method provided in ASTM D7263 (2021) by using a 38mm diameter cylinder. The specific gravity of soil particles was determined by water pycnometer.

2.1.1.3. Consistency Limits

Cohesive soils show different behaviour depending on the change in water content. Atterberg, (1911) defined the water contents that distinguish these states and called them consistency limits. Based on the results obtained according to ASTM D4318 (2017) and shown in table 2.2, the liquid limit of soil was determined as 143.6% and the plastic limit as 38.8%. The soil had high plasticity since its plasticity index was 104.8.

Table 2.1. Physical characteristics of soil

Property	**Value**	**Standard reference**
Natural water content, ω_n (%)	34	(ASTM D2216, 2019)
Natural unit weight, γ_n (Kn/m^3)	16.67	(ASTM D7263, 2021)
Specific gravity, G_s (g/cm^3)	2.625	(ASTM D854, 2014)

.

Table 2.2. Consistency limits of soil

Liquid limit, LL (%)	**Plastic limit, PL (%)**	**Plasticity index, PI (%)**
143.6	38.8	104.8

2.1.1.4. Soil Classification

In order to classify soil, the wet sieve analysis method and hydrometer test were performed according to ASTM D422 (2007) and based on the grain size distribution curve in Figure 2.5. The percentages of gravel, sand, silt and clay in soil according to MIT is given in table 2.3. The gravel-sand boundary is defined by ASTM as No. 10 sieve (2 mm), the coarse-fines as 0.06 mm, and the boundary between clay and silts as 0.002 mm.

In the USCS system, fine-grained soils are named according to the plasticity graph given by Casagrande as shown in figure 2.6 using the liquid limit and plasticity index (Casagrande, 1938). Soil is classified as CH according to USCS system and A-7-5 according to AASHTO system.

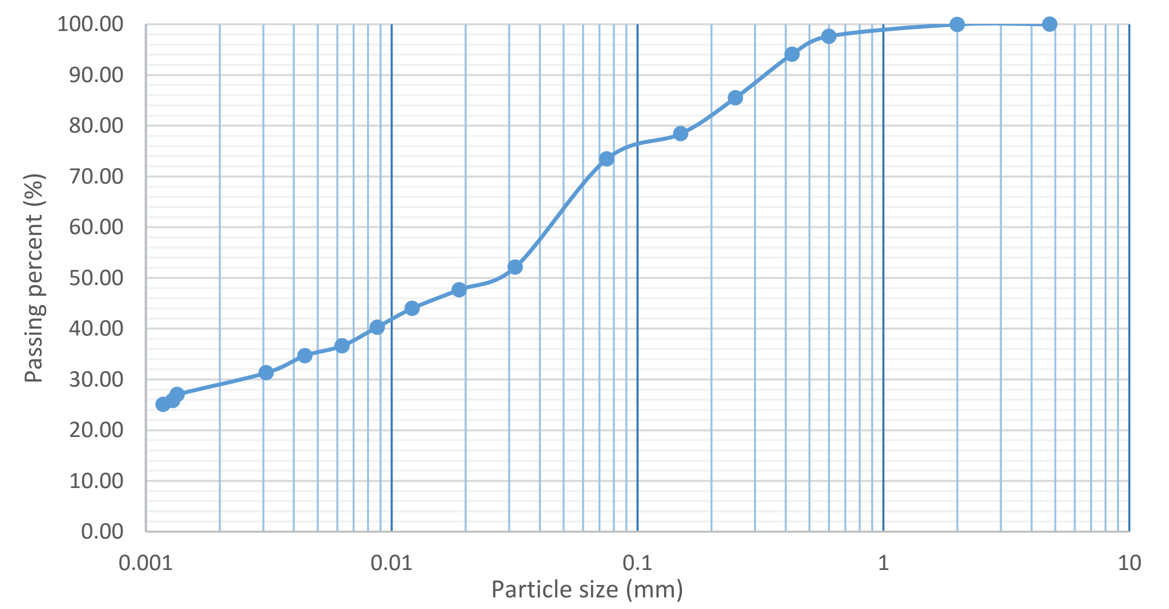

Figure 2.5. Particle size distribution curve of soil

Table 2.3. Soil texture classification according to MIT

Classification system	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
MIT	0.02	31.98	39	29

To know if the soil includes organic or not, the ratio in equation (2.1) is used;

$$\frac{Liquid\ limit(\ oven\ dried)}{Liquid\ limit\ (\ air\ dried)} \tag{2.1}$$

If the results of equation 2.1 equals to or less than 0.75, the soil is organic.

$\frac{Liquid\ limit(\ oven\ dried)}{Liquid\ limit\ (\ air\ dried)} = \frac{143.6}{152.4} = 0.94 > 0.75$, thus the soil is not organic.

2.1.1.5. Permeability

In order to determine the permeability coefficient of the soil, the sample, which was compacted at the optimum water content and maximum dry density, was placed in the falling head permeability instrument. At the end of the experiment, the permeability coefficient was found to be 7.511x10^{-9} cm/sec.

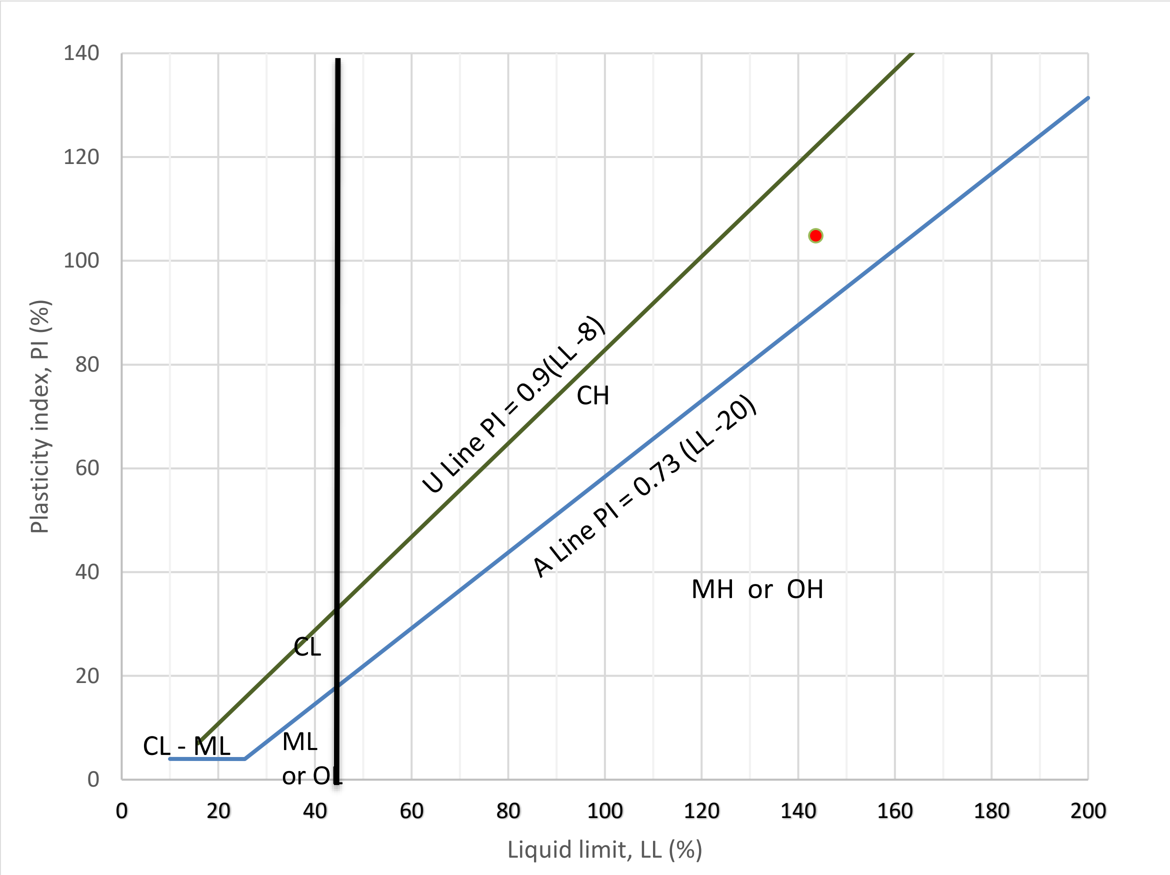

Figure 2.6. Classification of soil according to USCS Plasticity graph

2.1.1.6. Compaction Parameters

The standard proctor test was performed on the disturbed samples collected from the field according to (ASTM D698, 2012) in order to determine the optimum water content and maximum dry density values, which represent the compaction parameters. Based on the data obtained from this experiment, the compaction curve in Figure 2.7 was drawn and the maximum dry density was found to be 1.23 g/cm^3, and the optimum water content was 37.4%.

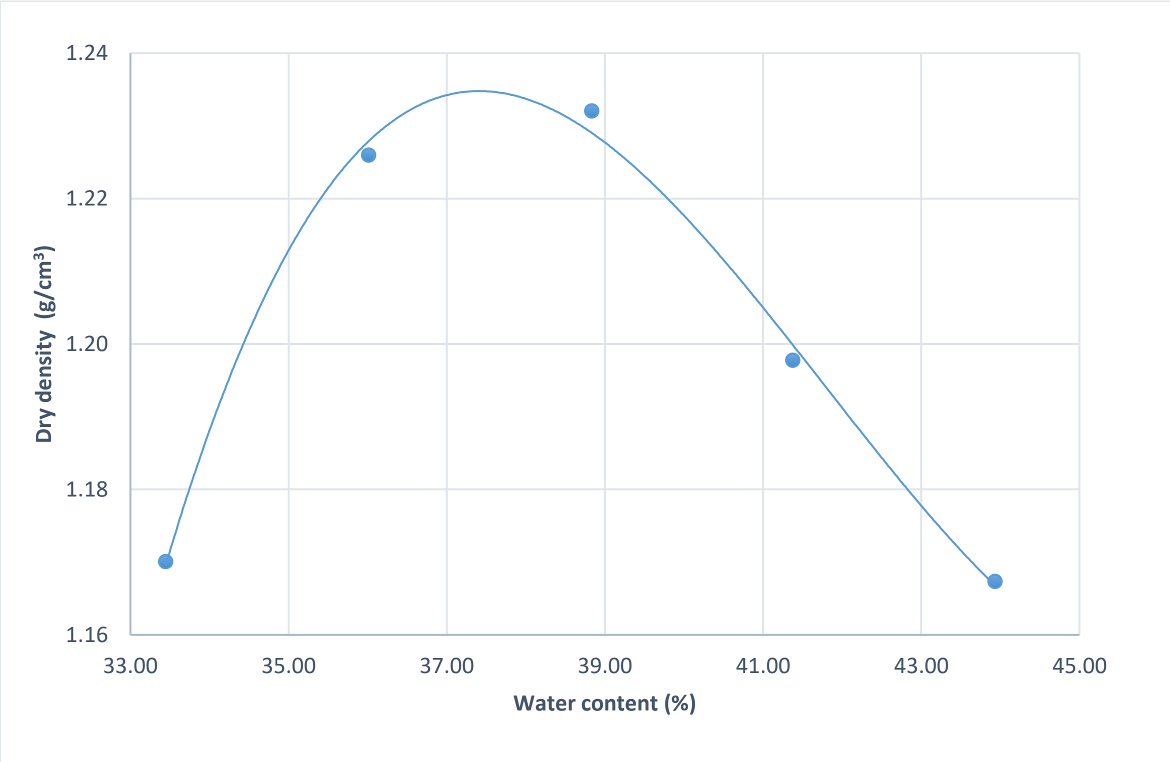

Figure 2.7. Compaction curve of soil

2.1.1.1. Strength of Soil

The strength of soil was found using unconfined compressive strength (ASTM D2166, 2016). The soil was compacted at its maximum dry density and optimum water content. Following this step, 3 samples of 38mm diameter and 76mm height were taken. The soil was tested after curing time of 1, 7, and 28 days by using 3 samples at each and taking the average of the values. Based on the tested values, the soil had a compressive strength of 327.17, 367.07, and 348.10 kPa after curing at 1, 7, and 28 days, respectively. The stress-strain curve for one of the samples after curing for 1 day is shown in figure 2.8.

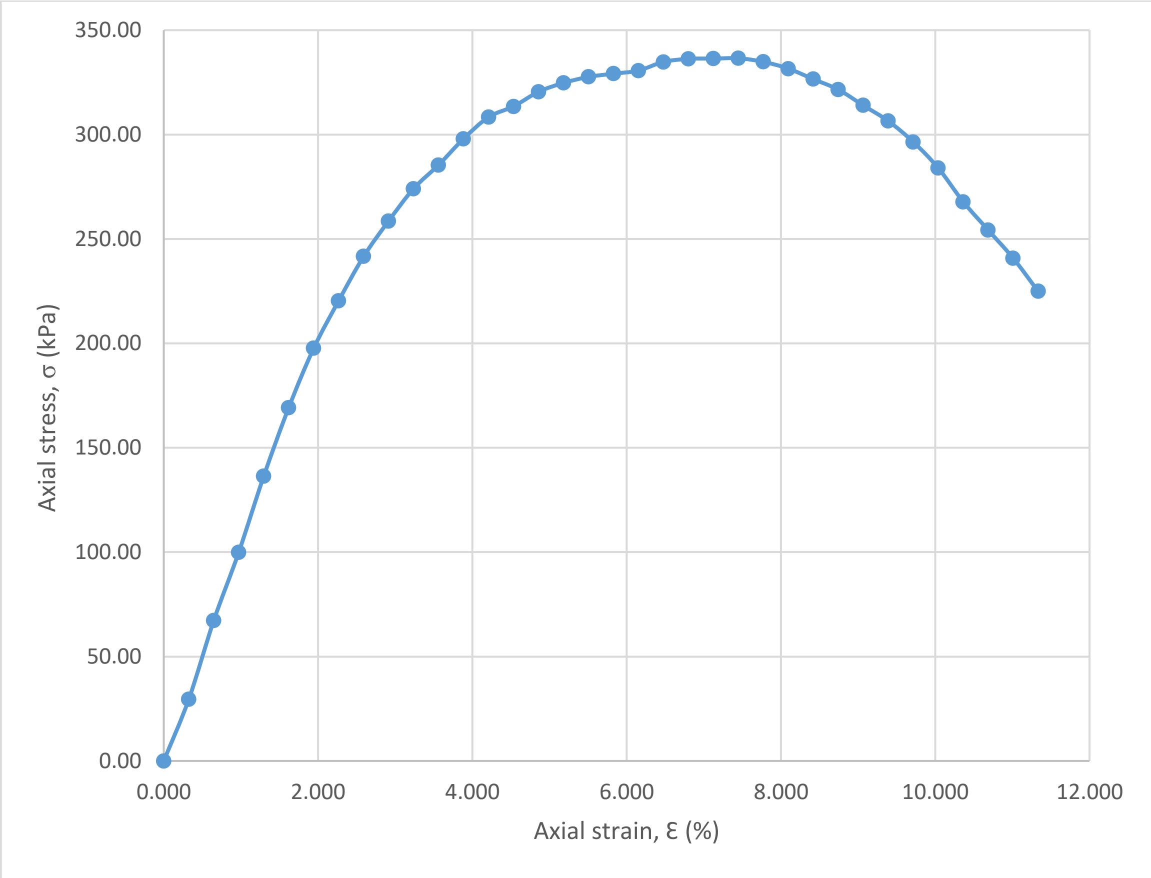

Figure 2.8. Stress-strain curve of soil after 1 day curing

2.1.1.2. Swelling Parameters

In order to find the swell percentage and swell pressure, the soil specimen was compacted under optimum water content and maximum dry density. Thereafter, the specimen was assembled in a consolidometer unit to perform the test accordance to method C of ASTM D 4546 (2021). The soil specimen was subjected to a pressure equals to in-situ pressure, which was 17 kPa. After the completion of swelling as shown in figure 2.9, the specimen was subjected to incremental loads according to ASTM D2435 (2020) to find the swelling pressure as shown in figure 2.10. The swell percentage was found to be 34.15% and the swell pressure was 480 kPa.

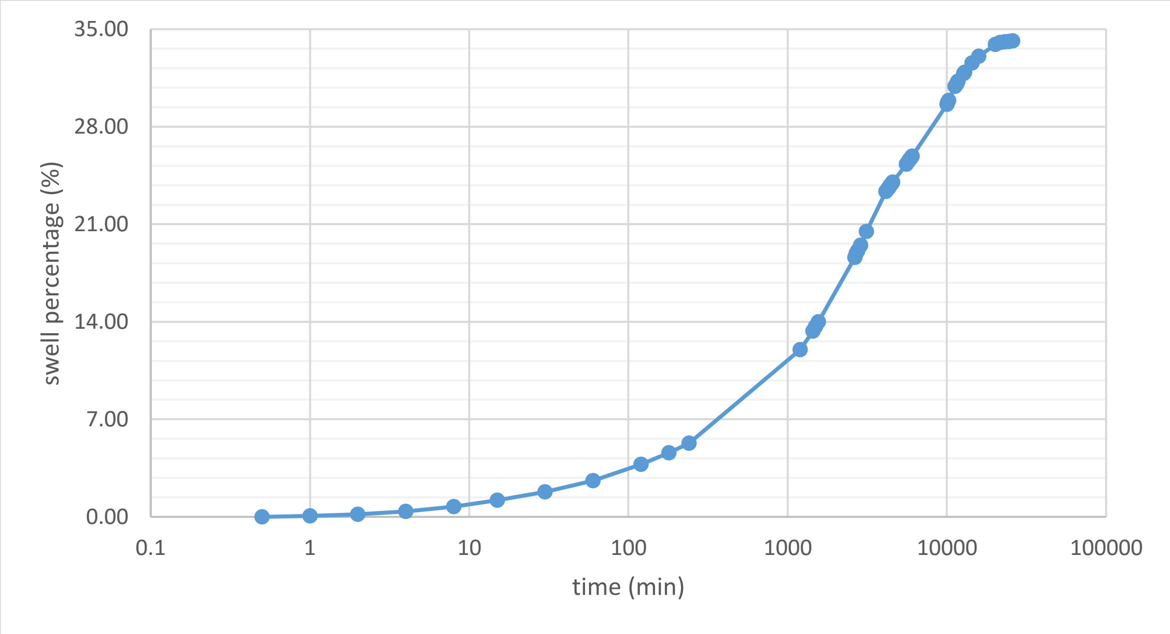

Figure 2.9. Time- swell curve for soil sample under 17 kPa

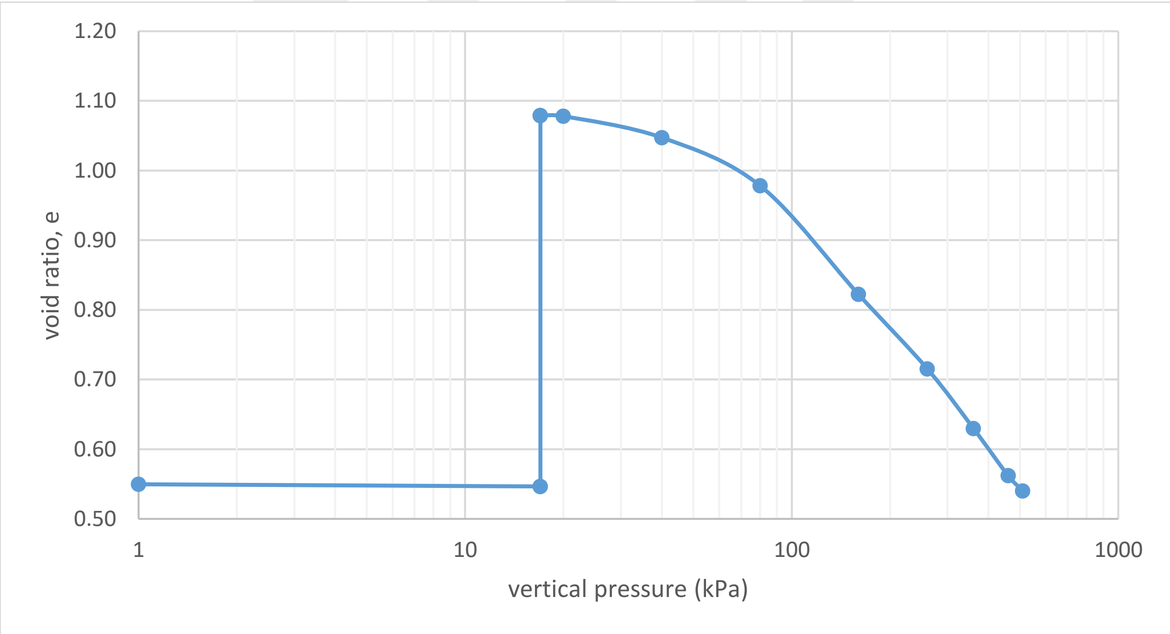

Figure 2.10. Vertical pressure vs. void ratio for soil sample under 17 kPa

However, the treated samples did not swell under the in-situ pressure, therefore the test was applied again under 1 kPa pressure for the soil and the treated sample in order to make comparison of the results. The swell percentage was found to be 62.17% and the swell pressure was 580 kPa as shown in figures 2.11 and 2.12.

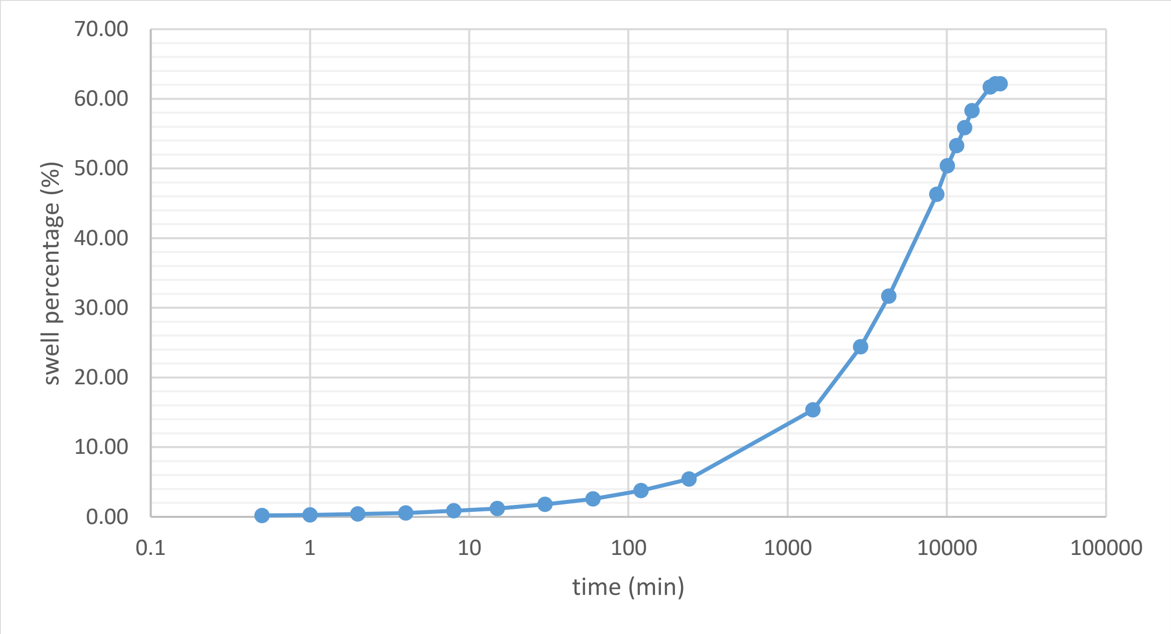

Figure 2.11. Time- swell curve for soil sample under 1 kPa

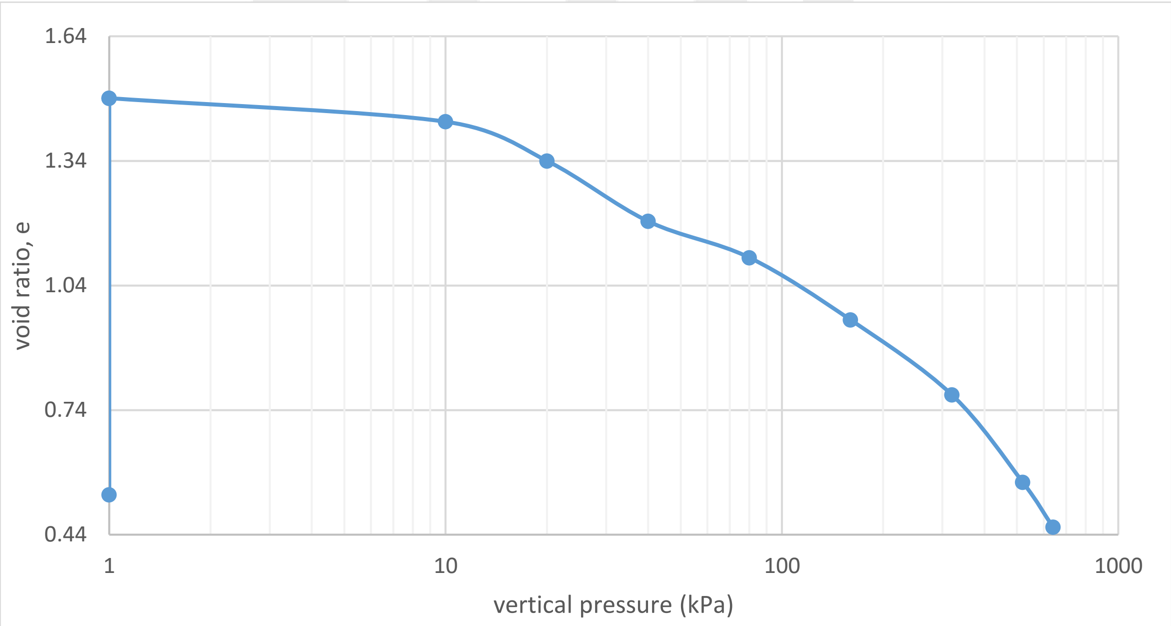

Figure 2.12. Vertical pressure vs. void ratio for soil sample under 1 kPa

2.1.2. Lime

Lime, which was taken from Barkisan Lime Factory (figure 2.13), is one of the additives that was utilized at ranges of 0-8% in the tests. It consisted of CL 80 S type slaked lime and its properties according to TS EN 459 (2015) are shown in table 2.4.

Table 2.4. Properties of CL 80 S lime according to TS EN 459 (2015)

CaO + MgO, %	Min. 80
MgO, %	Max. 5
SO_3, %	Max. 2
Unit weight, g/cm^3	Max. 0.6
retained percent above 200 µ sieve, %	Max. 2
retained percent above 90 µ sieve, %	Max. 7

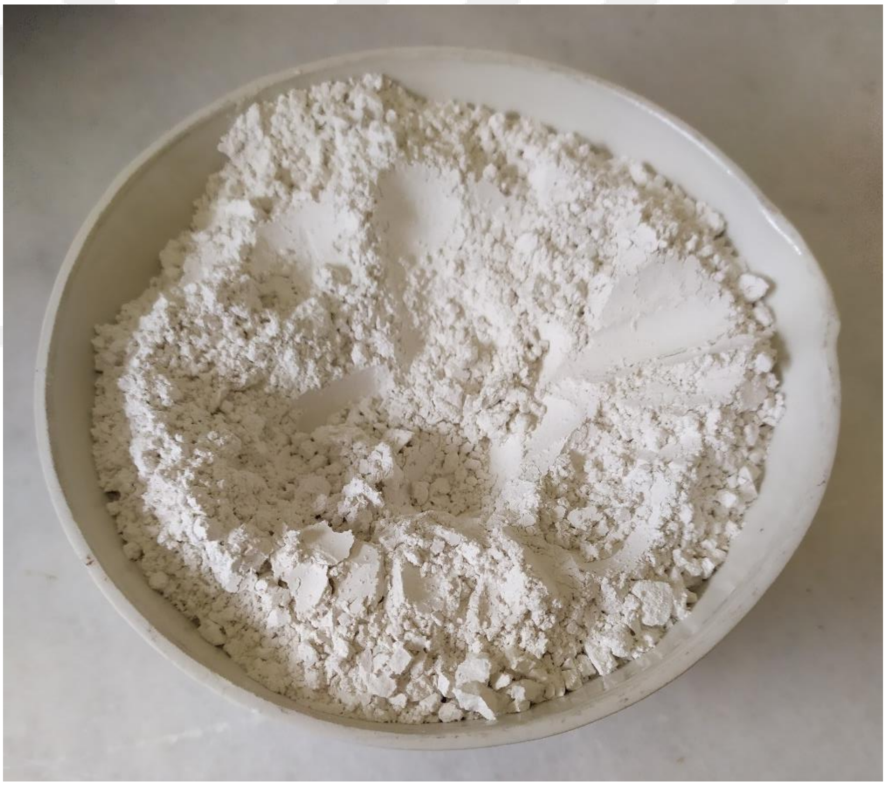

Figure 2.13. Lime

2.1.3. Pumice

Pozzolans are fine-grained materials that have little or no binding on their own, but react chemically with binders containing calcium hydroxide ($Ca(OH)_2$) at appropriate water content and at normal ambient temperature, and release cementation products. Pumice is considered as one of the natural pozzolans and is distinguished from other types of rocks that have a practically similar structure by its colour, porosity, and absence of crystalline water.

In parallel with the increasing attention given to lightweight building materials in the construction sector in recent years, the use of pumice as a building material in raw material consumption is becoming frequently widespread. In addition, it is a well-known fact that in the construction sector, the necessity of using lightweight concrete mixtures in buildings dates back to ancient times. It is probable to see several examples of its application in the production of lightweight concrete due to its high insulating properties.

In this study, pumice was collected from Erçiş region of Van province and used at ranges of 0-25% with an increment of 5%. Table 2.5 shows the chemical composition of the used pumice. It also shows that the total content of SiO_2 + Al_2O_3 + Fe_2O_3 exceeds 70%, which is one of the important conditions for materials to be used as pozzolans according to ASTM C618 (2019).

Table 2.5. Chemical composition of pumice

Chemical Composition	**Percentage (%)**	**Chemical Composition**	**Percentage (%)**
SiO_2	71.21	Ba	0.01
Al_2O_3	12.37	Cr_2O_3	0.01
Fe_2O_3	1.44	MnO	0.07
MgO	0.11	SO_3	0.067
CaO	0.77	P_2O_5	<0.01
Na_2O_3	3.62	TiO_2	0.09
K_2O	4.86	LOI	4.4
SiO_2 + Al_2O3 + Fe_2O_3 (%)		85.02	

2.1.4. Marble

Marble is a composition formed by the recrystallization of limestone and dolomitic limestones as a result of metamorphism. The main mineral in marbles consisting of $CaCO_3$ crystals is “Calcite", which consists of 90-98% of their composition. It contains a low amount of $MgCO_3$. Small amounts of silica, feldspar, iron oxide, mica, fluorine, and organic substances may also be present.

The total visible, probable, and potential resources of natural stones in Turkey, which has wide and quite diverse mineral deposits, are 5.2 million m^3. Marble is mined from 700 quarries in Turkey, and there are 120 factories and 1600 workshops that process marble. 90% of the marble quarries are in the western part of Turkey, and mainly located in the Aegean region, Marmara Island, and Afyon province.

In this study, marble was provided as waste pieces at a local workshop that collects the natural marble from Amasya Beige marble. The marble was crushed and used at ratios of 0, 5, 7, 12, and 15%.

2.2. Experimental Work

Lime, marble, and pumice were used in this study at ranges of 0-8%, 0-15%, and 0-25%, respectively, in order to investigate their effects on stabilization of the swelling soil. Experiments of consistency limits, compaction test, unconfined compressive strength, and one-dimensional swelling tests were performed at two stages as explained in sections 2.2.1 and 2.2.2.

2.2.1. First Stage

The first stage included measuring the consistency limits for mixtures of each additive alone with soil and then the additives mixed together with soil with each other using the Taguchi method for optimization. Firstly, all materials were oven-dried at 105°C and sieved through sieve #40. Secondly, each additive was mixed with soil at ratios as listed in table 2.6 where each mixture is given a specific name. Afterwards, water was added to mixtures and left for 1 day. The tests of liquid limit and plastic limit were performed as shown in figure 2.14.

Table 2.6. Mixtures of prepared samples and their ratios

Mixture name	**Soil %**	**Additive type**	**Additive ratio %**
S	100	-	-
SP5	95	Pumice	5
SP10	90	Pumice	10
SP15	85	Pumice	15
SP20	80	Pumice	20
SP25	75	Pumice	25
SL3	97	Lime	3
SL5	95	Lime	5
SL6	94	Lime	6
SL8	92	Lime	8
SM5	95	Marble	5
SM7	93	Marble	7
SM12	88	Marble	12
SM15	85	Marble	15

Where; S: soil, P: pumice, L: lime, and M: marble

Figure 2.14. Plastic limit and liquid limit for additives

In the same manner, all additives were mixed together with soil and the consistency limits were tested for mixtures prepared based on the Taguchi method. The Taguchi procedure utilizes standard tables called Orthogonal Arrays (OA) for building the design of experiments. An Orthogonal Array, L25, was adopted to define the experimental design because it is the most appropriate for the situations being examined, using three factors with five levels each. The Taguchi procedure employs the S/N ratio (signal to noise), which displays the scatter about a target value, rather than the average value.

Pumice, lime, and marble were added at different levels by mass of the total materials. Levels of the experimental factors to be studied are listed in Table 2.8. An L25 OA was selected to analyse the experimental results. Details of trial no. , levels, and the experimental design are listed in Table 2.7. Each row represents a different trial and shows the values related to each factor used in that trial.

Table 2.7. Chosen L25 (OA)

Trial no.	**Name**	**Pumice (%)**	**Lime (%)**	**Marble (%)**
1	SP5	5	0	0
2	SP5L3M5	5	3	5
3	SP5L5M7	5	5	7
4	SP5L6M12	5	6	12
5	SP5L8M15	5	8	15
6	SP10M5	10	0	5
7	SP10L3M7	10	3	7
8	SP10L5M12	10	5	12
9	SP10L6M15	10	6	15
10	SP10L8	10	8	0
11	SP15M7	15	0	7
12	SP15L3M12	15	3	12
13	SP15L5M15	15	5	15
14	SP15L6	15	6	0
15	SP15L8M5	15	8	5
16	SP20M12	20	0	12
17	SP20L3M15	20	3	15
18	SP20L5	20	5	0
19	SP20L6M5	20	6	5
20	SP20L8M7	20	8	7
21	SP25M15	25	0	15
22	SP25L3	25	3	0
23	SP25L5M5	25	5	5
24	SP25L6M7	25	6	7
25	SP25L8M12	25	8	12

Table 2.8. Test factors used and their levels for the first stage

Levels	**Factors**		
	Pumice (%)	Lime (%)	Marble (%)
1	5	0	0
2	10	3	5
3	15	5	7
4	20	6	12
5	25	8	15

The soil and additives were dried in an oven at nearly 105°C. The required quantities of soil, pumice, lime, and marble were mixed. 5-25% of pumice, 0-8% of lime, and 0-15% of lime by total dry mass of mixture were utilized in the preparation of samples. After determining the liquid limits and plastic limits for all trials (Table 2.7), the plasticity index was calculated and the analysis of these results was performed using Taguchi method.

Based on the results, the optimum two levels for each factor were selected. The Taguchi method was implemented again using the same factors and the best two levels for each factor forming an orthogonal array, L4. The trials of L4 were tested for the consistency limit to compare it with the expected results from the Taguchi method to ascertain the validity of the Taguchi procedure. In addition to consistency limits, the second experimental designed trials of Taguchi method were used in the second stage and tested for compaction, unconfined compressive, and one-dimensional testes as explained in section 2.2.2.

2.2.2. Second Stage

In the second stage of this study, the optimum 2 ratios for each additive were selected based on the consistency limits and used to build up an orthogonal array, L4, of 2 levels and 4 experimental factors to perform unconfined compressive strength, compaction, and one-dimensional tests as shown in table 2.10. Experimental factors and their levels are shown in table 2.9. An additional trial was also chosen to be performed in order to compare the expected results of Taguchi method with the experimental results.

Table 2.9. Test factors used and their levels for the second stage

Levels	**Factors**		
	Pumice (%)	Lime (%)	Marble (%)
1	20	6	12
2	25	8	15

Table 2.10. Chosen L4 (OA)

Trial no.	**Name**	**Pumice (%)**	**Lime (%)**	**Marble (%)**
1	SP20L6M12	20	6	12
2	SP20L8M15	20	8	15
3	SP25L6M15	25	6	15
4	SP25L8M12	25	8	12
5 Additional trial for comparison	SP20L8M12	20	8	12

2.2.2.1. Compaction Test

Compaction test is used to find two essential factors of soil including maximum dry density and optimum moisture content. The applied energy on soil and soil type are the major factors that influence soil compaction. Standard proctor compaction test was performed based on ASTM D698 (2012) standard to determine the compaction parameters for each mixture as shown in figure 2.15.

Figure 2.15. Compaction test

2.2.2.2. Unconfined Compressive Strength

The unconfined compressive strength (UCS) test is one of the most important tests used to find the strength of soil or treated soils. This test was performed according to ASTM D2166 (2016). Firstly, the samples were compacted at optimum moisture content and maximum dry density in the mold of the compaction test. Subsequently, three tubes were used to extract 3 samples for UCS as shown in figure 2.16. The samples were cured for 1, 7, and 28 days as shown in figure 2.17. After curing, for each mixture, and at each curing time, 3 samples with a height of 76 mm and diameter of 38 mm were tested as shown in figure 2.18.

Figure 2.16. Samples preparation for UCS test

Figure 2.17. Curing of the samples for 1, 7, and 28 days

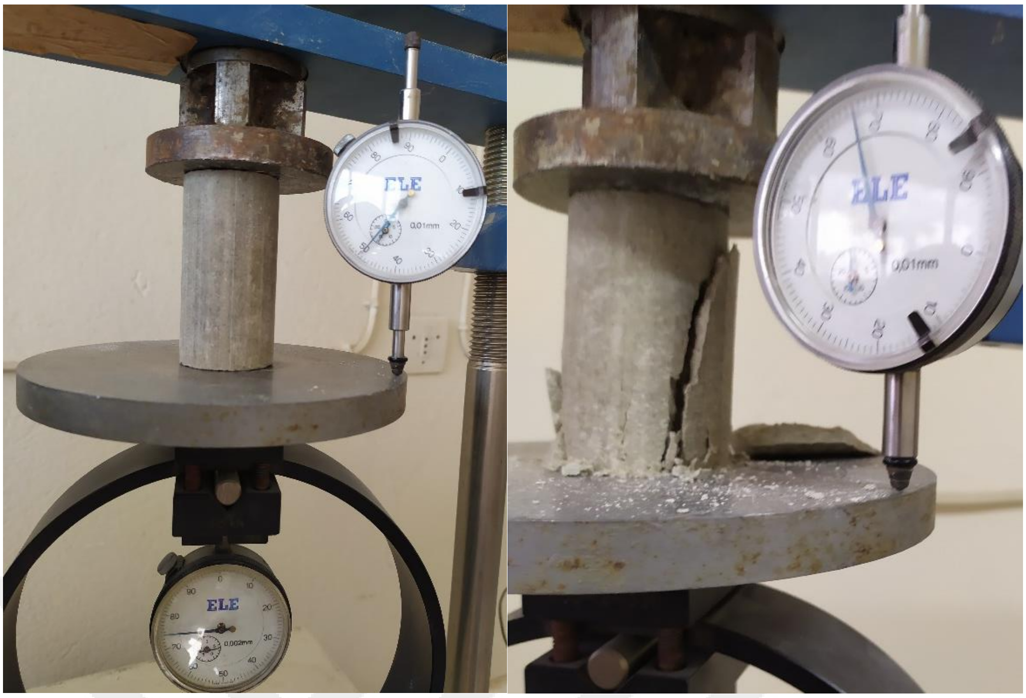

Figure 2.18. UCS test

2.2.2.3. One Dimensional Swelling Test

One-dimensional swell test is one of the most important tests used to find the swell percentage and swell pressure. This test was performed according to method C ASTM D4546 (2021). Firstly, the samples were compacted at optimum moisture content and maximum dry density in the mold of the compaction test. Following that, a ring of 5cm diameter was used to extract the sample. The sample was cured for 1 day. After curing, the sample was assembled in the consolidometer and the balance of the arm was controlled as shown in figures 2.19 and 2.20.

The one-dimensional swell test was performed by applying an in-situ pressure of 17kPa as explained in section 1.4.3.3. However, the specimens started to collapse, therefore the pressure was reduced to 1 kPa. The samples started to swell and after the completion of swelling, an incremental loading was applied to find the swell pressure according to ASTM D2435 (2020) as shown in figure 2.21.

Figure 2.19. Preparation of the samples for one-dimensional swell test

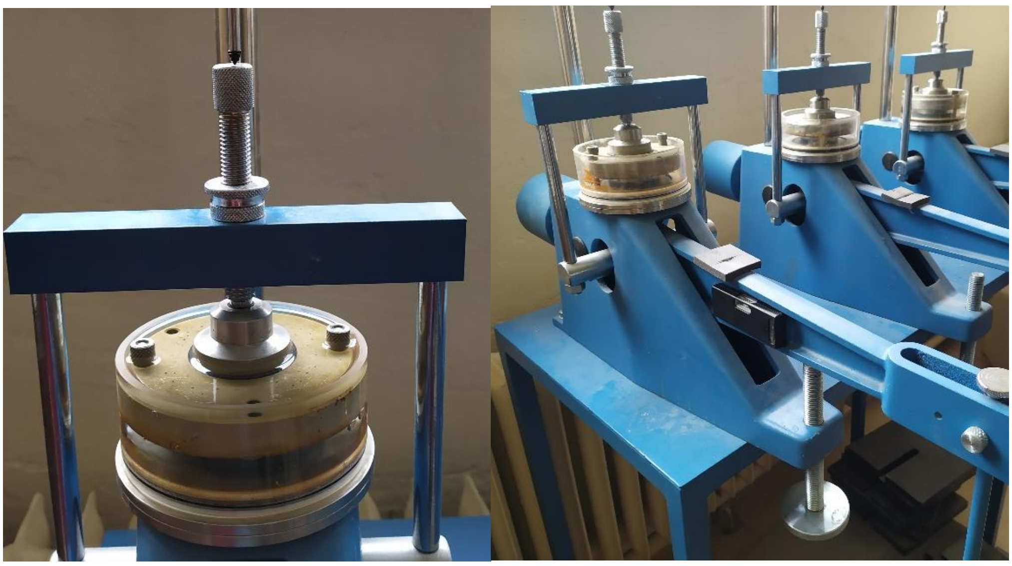

Figure 2.20. Assembling samples in loading device and balancing the arm of the deceive

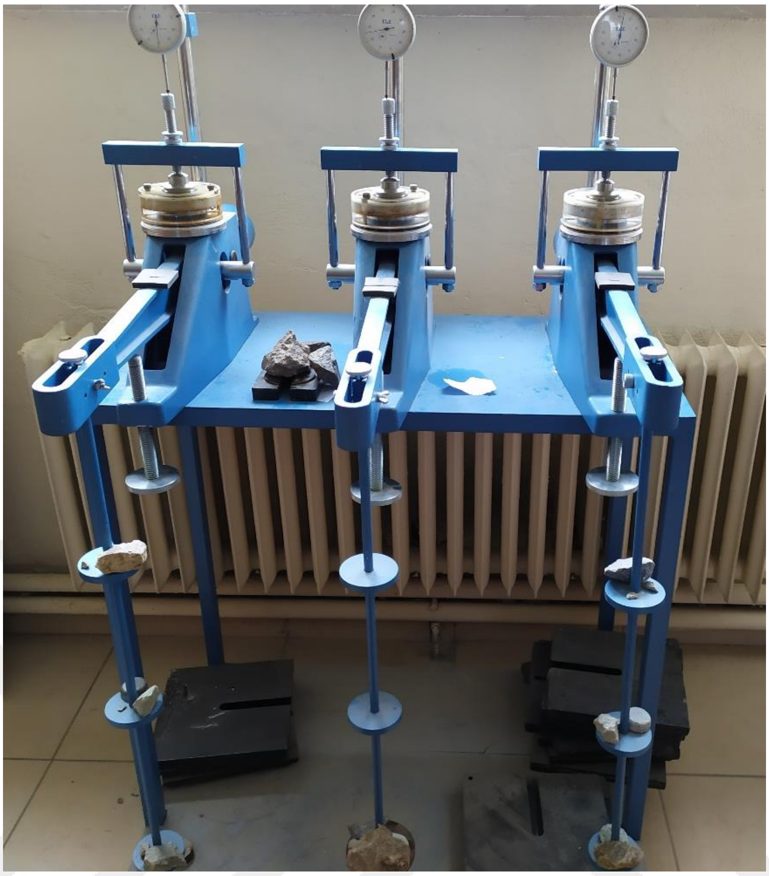

Figure 2.21. Applying loads to find the swelling pressure

3. RESULTS AND DISCUSSION

3.1. Effect of Stabilizers on Consistency Limit

For using single mixing of one stabilizer with high plasticity soil, the consistency limits were performed using pumice at the rates of 5%, 10%, 15%, 20%, and 25% and lime at ratios of 3%, 5%, 6%, and 8%. In addition, marble was used at percentages of 5%, 7%, 12%, and 15%. The results of single mixing with soil are listed in table 3.1 and also shown as a bar graph in figures 3.1, 3.2, and 3.3.

Table 3.1. Consistency limits for mixtures of soil with each additive alone

Mixture name	**Liquid limit, LL (%)**	**Plastic limit, PL (%)**	**Plasticity index, PI (%)**
S	143.6	38.8	104.8
SL3	131.7	57.1	74.6
SL5	99.3	52.7	46.7
SL6	103.1	61.1	42.0
SL8	97.4	55.0	42.4
SP5	141.0	36.1	104.9
SP10	133.1	36.7	96.4
SP15	128.0	40.6	87.4
SP20	116.7	38.5	78.1
SP25	112.5	33.6	78.9
SM5	142.0	37.9	104.0
SM7	137.1	37.0	100.1
SM12	115.2	35.9	79.2
SM15	129.0	32.6	96.4

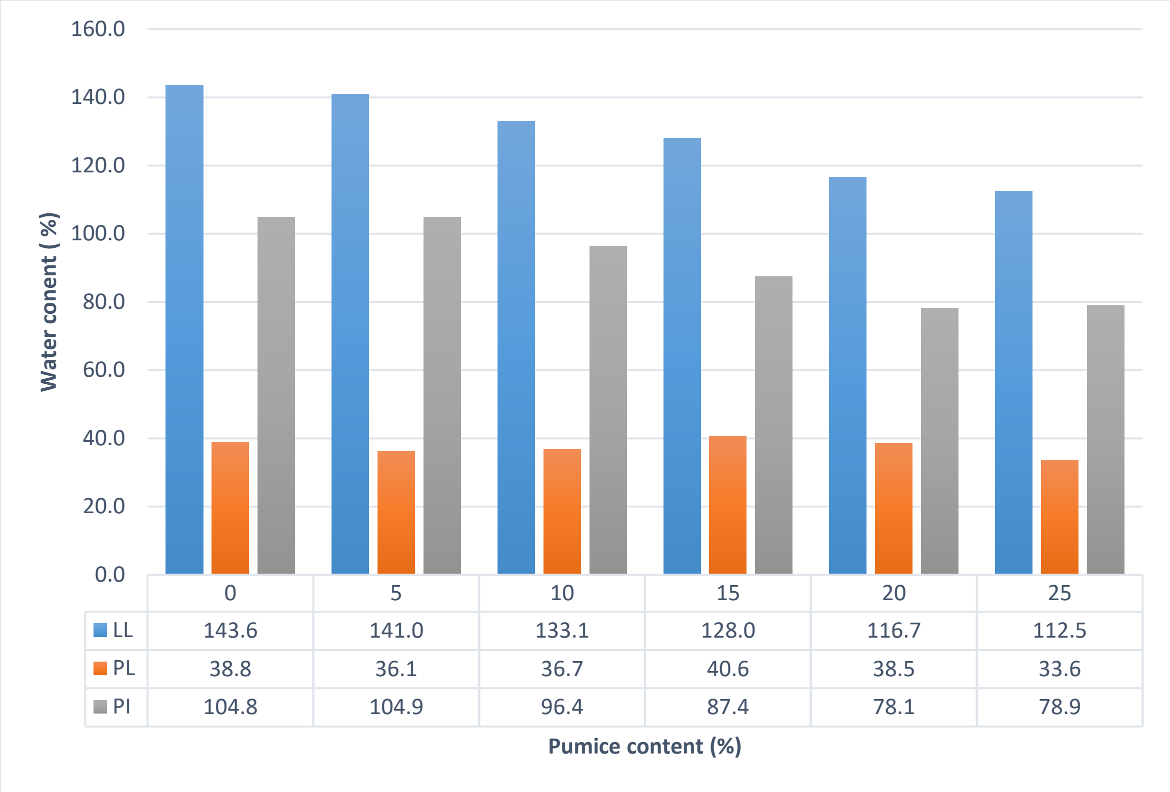

Figure 3.1. Consistency limits for soil with pumice

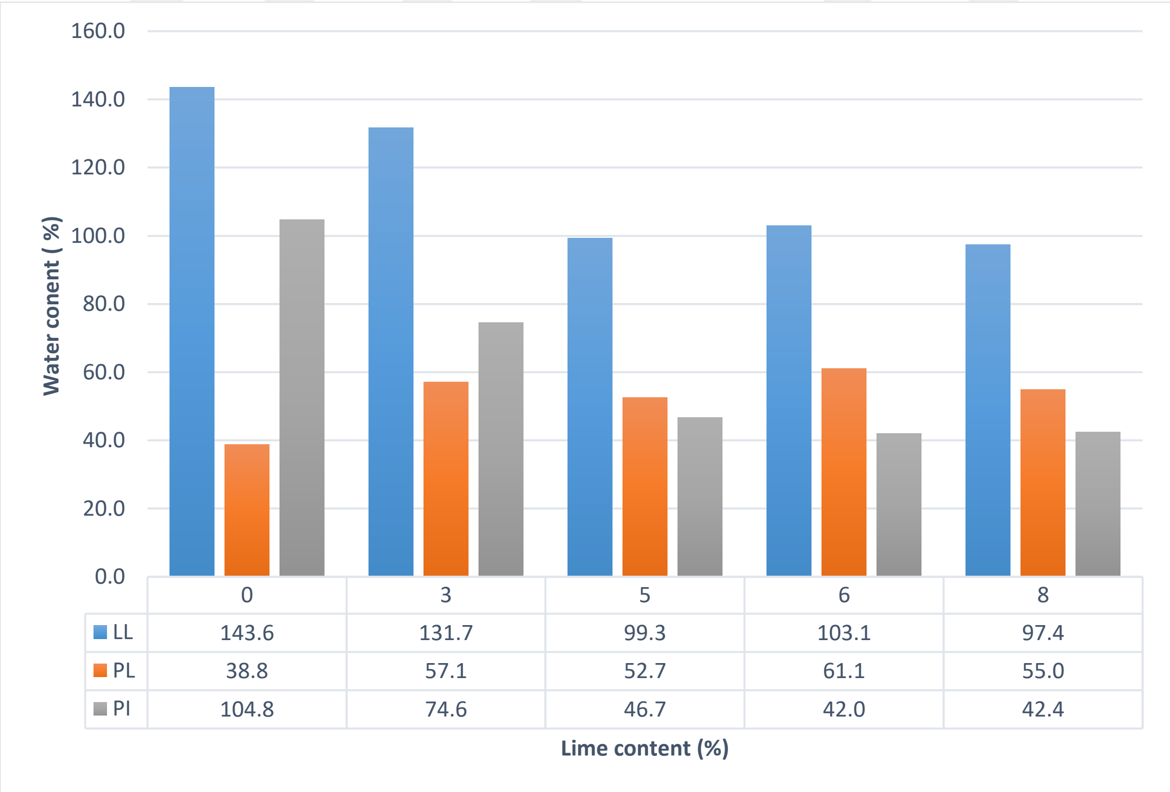

Figure 3.2. Consistency limits for soil with lime

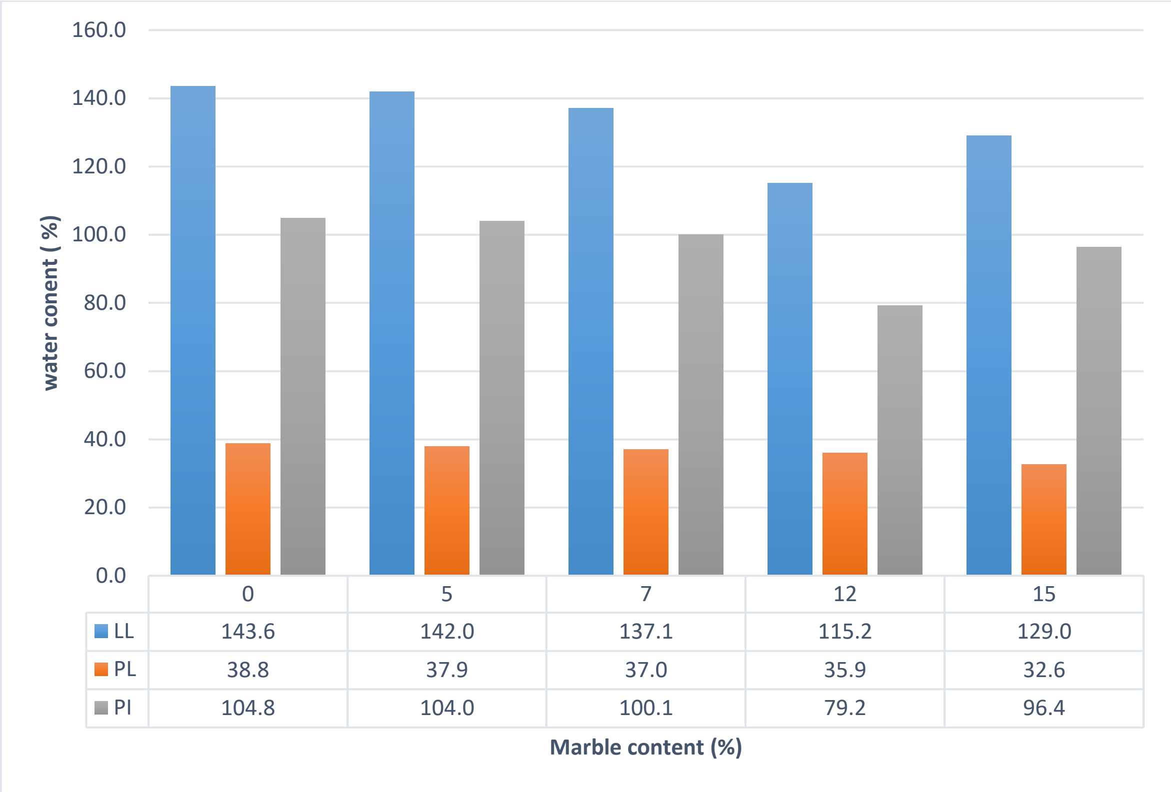

Figure 3.3. Consistency limits for soil with marble

Based on the liquid limit results for using each additive alone which are shown in figure 3.4, a decrease in the liquid limit value is observed as the pumice content is increased. In addition, as lime and marble content are increased up to 12% in marble and 5% in lime, the liquid limits generally decrease but slightly increase at 15% marble and 6% lime and then slightly continue to decrease with increasing the lime content. The maximum decrease percentage of liquid limit, which was 32.2%, occurred when 8% lime content was used, whereas the maximum decrease percentage in liquid limits for pumice and marble was 21.7% when using 25% of pumice and 19.8% when using 12% of marble.

According to the results shown in figure 3.5 of the plastic limit test for the case of using each additive alone with soil, the plastic limits increase when marble content is increased. However, the plastic limits strongly increase and are approximately between 52.7% and 61.1% when lime is used. For pumice, the results slightly decrease when the percentage of pumice is increased up to 10% and then increase at 15% pumice and then start to decrease again up to a value of 33.6% when 25% pumice is used.

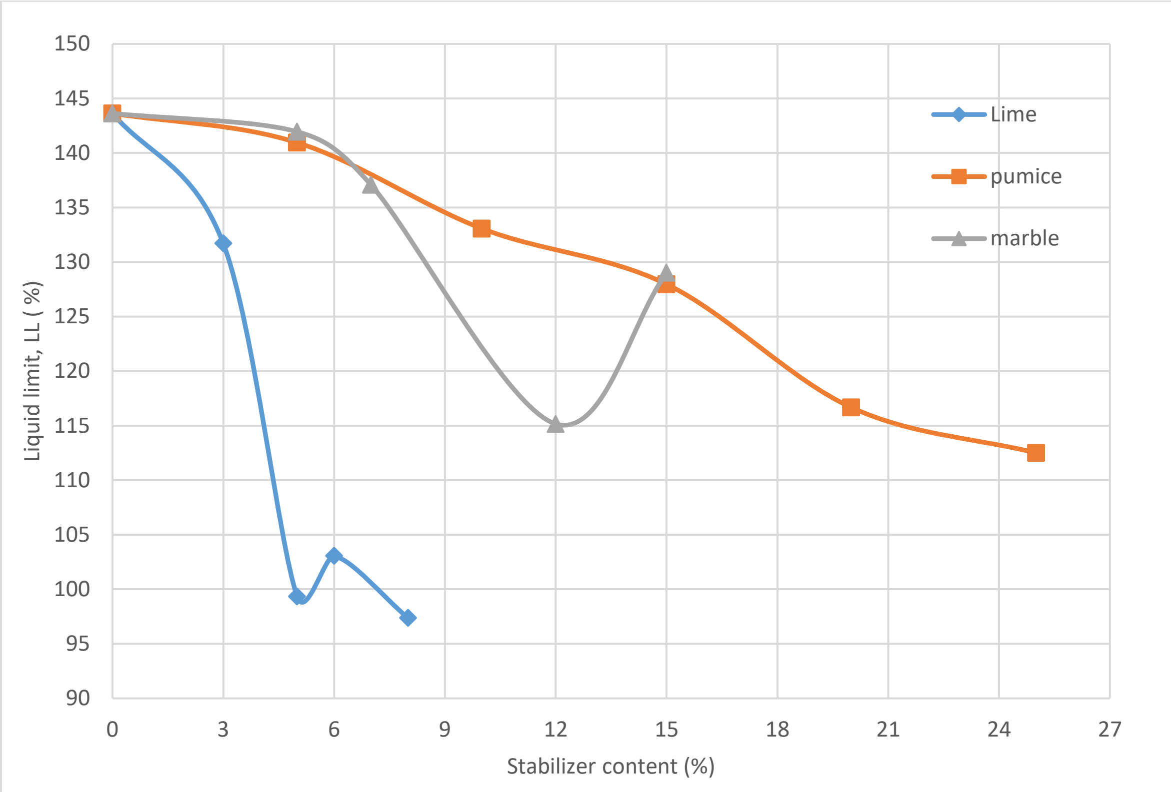

Figure 3.4. Results of liquid limits for soil with pumice, lime, and marble

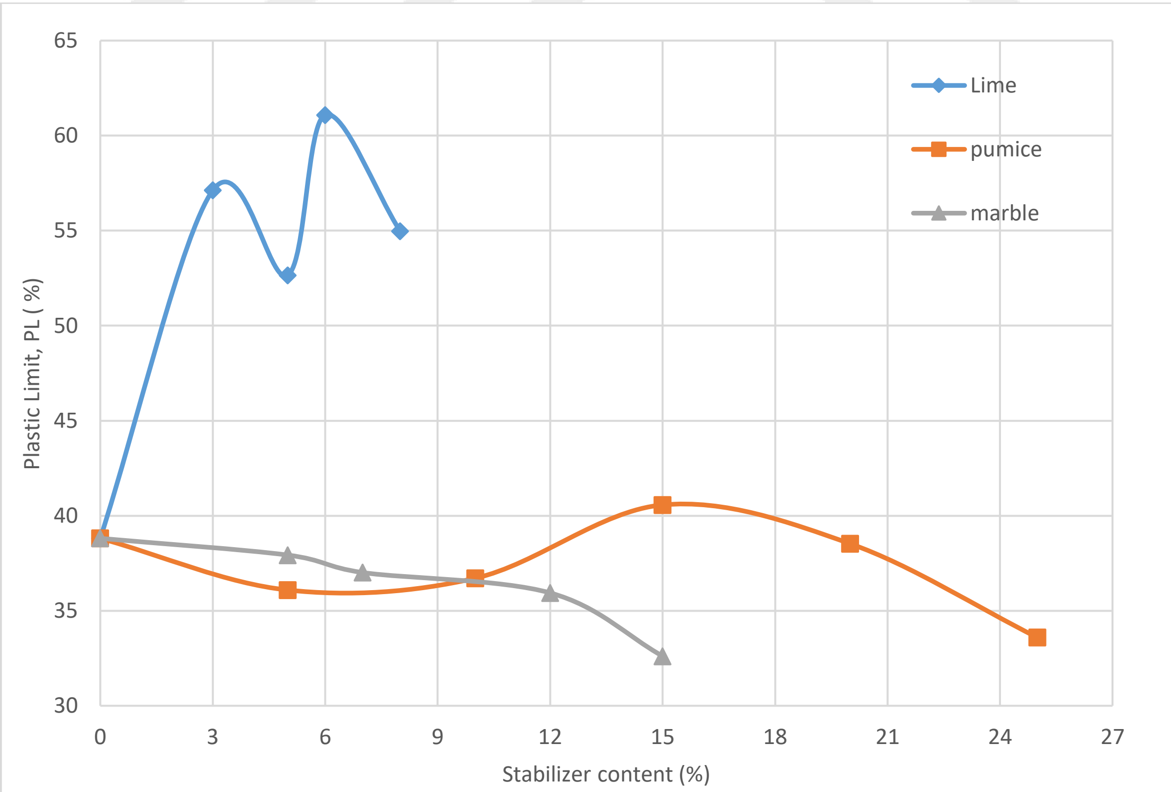

Figure 3.5. Results of plastic limits for soil with pumice, lime, and marble

The plasticity index results for the mixtures of an additive with the soil are shown in figure 3.6. It is observed that the plasticity index values decrease with the increase of pumice content up to 20% and then it becomes approximately equal to the result of using 25% pumice. The maximum reduction in plasticity index is 25.5% when 20% pumice is used. In addition, increasing the content of marble and lime up to 12% marble and 6% lime reduces the plasticity index and then imperceptibly increases when the lime content is increased and roughly increases when 15% marble is used. Using 6% lime reduces the plasticity index to a value of 42% while using 12% marble reduces the plasticity index by a percentage of 24.4%.

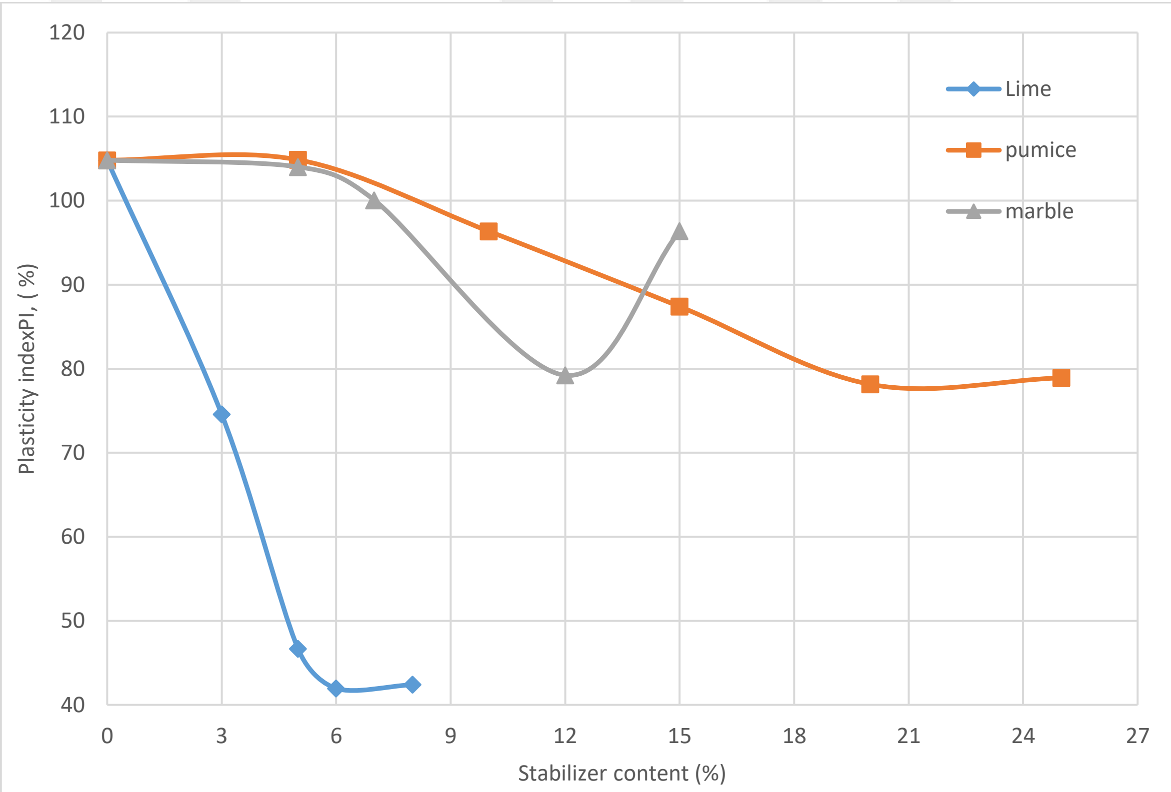

Figure 3.6. Results of plasticity index for soil with pumice, lime, and marble

The stabilizers were mixed with the soil by using the additive at the same ratios excluding 7% of lime and then 25 trials were tested at the first stage. The results of this stage are listed in table 3.2. The 25 trials were initially 125 but reduced to 25 by the Taguchi method. Using these 25 trials with 5 levels for each additive, the results for the rest of the mixtures (100 mixtures) were predicted. The test results obtained for 25 trials were analysed for the liquid limit and plasticity index using “the smaller is the better” function and the “larger is the better” function for the plastic limit. By interpreting the graphs determined according to the results of the analysis (figures 3.7-3.12), the best 2 ratios of each additive were determined, which were 20% and 25% for pumice. In addition, 12% and 15% for marble and 6% and 8% for lime were determined and used in the second stage of Taguchi design experiments, for which results are listed in table 3.3. While determining these values, it was taken into account at which ratios the best decrease occurred.

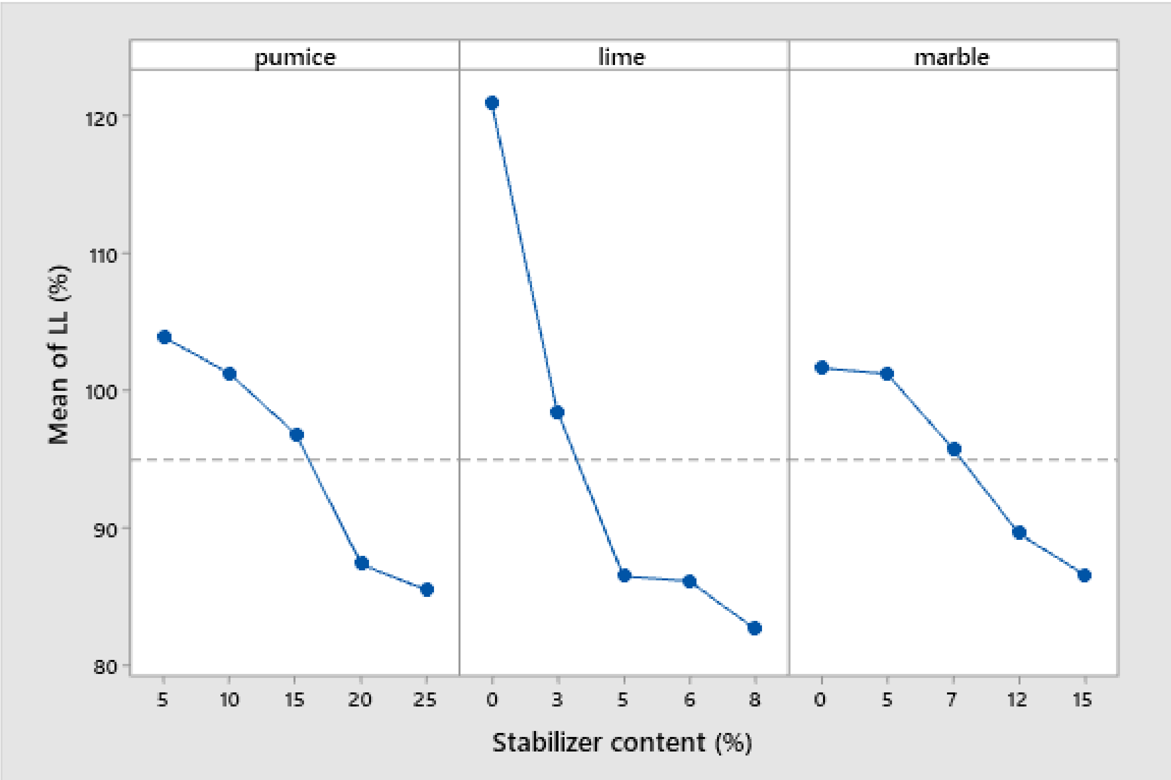

Figure 3.7. Parameter effects on mean of liquid limit

Table 3.2. Consistency limits for first-stage design experiments

Trial no.	Name	LL (%)	PL (%)	PI (%)
1	SP5	141.0	36.1	104.9
2	SP5L3M5	112.8	47.1	65.6
3	SP5L5M7	90.8	61.0	29.8
4	SP5L6M12	90.4	61.3	29.1
5	SP5L8M15	84.6	56.0	28.6
6	SP10M5	143.9	35.2	108.8
7	SP10L3M7	101.7	54.3	47.5
8	SP10L5M12	86.6	54.1	32.5
9	SP10L6M15	84.4	56.2	28.2
10	SP10L8	89.7	59.2	30.5
11	SP15M7	123.4	35.2	88.3
12	SP15L3M12	96.0	53.4	42.6
13	SP15L5M15	84.9	54.3	30.6
14	SP15L6	93.9	60.3	33.6
15	SP15L8M5	86.1	59.2	26.9
16	SP20M12	103.2	35.2	68.0
17	SP20L3M15	85.3	57.1	28.2
18	SP20L5	87.1	57.2	29.8
19	SP20L6M5	80.2	61.4	18.8
20	SP20L8M7	81.0	56.1	25.0
21	SP25M15	93.8	34.2	59.6
22	SP25L3	96.7	61.3	35.4
23	SP25L5M5	83.2	59.1	24.1
24	SP25L6M7	81.8	62.1	19.8
25	SP25L8M12	71.8	57.2	14.6

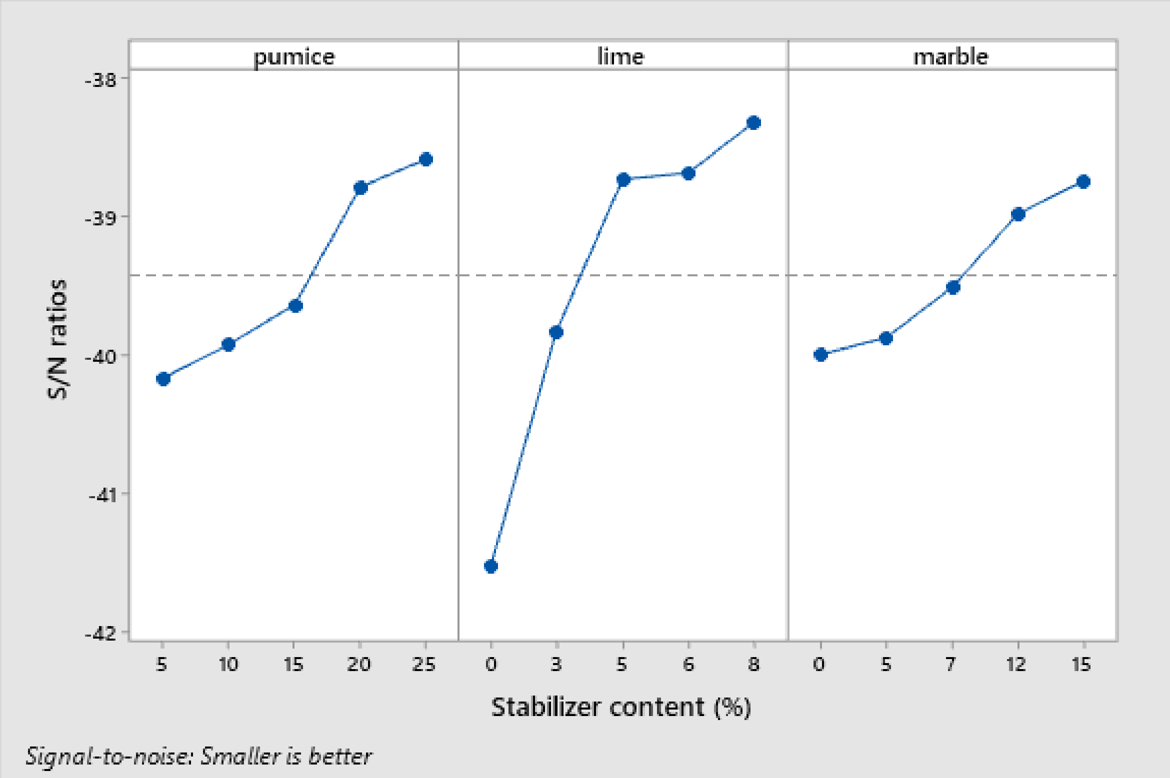

Figure 3.8. Parameter effects on mean S/N ratio for the LL

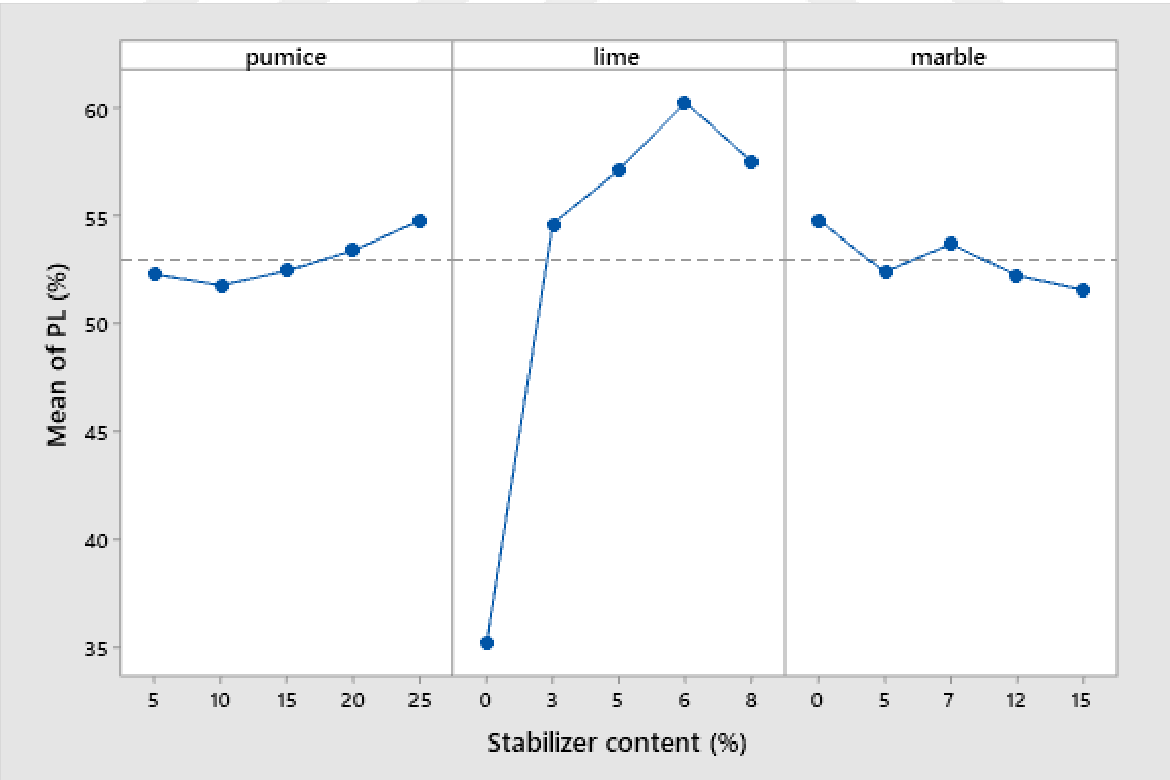

Figure 3.9. Parameter effects on mean of plastic limit

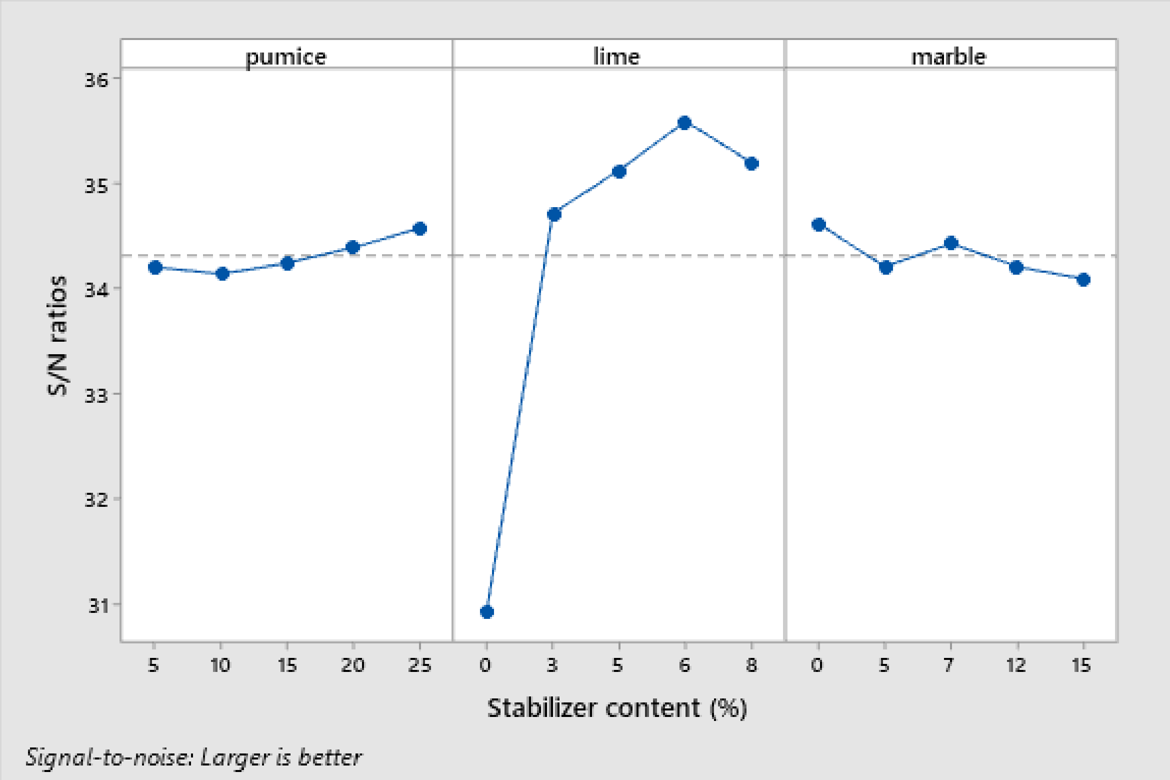

Figure 3.10. Parameter effects on mean S/N ratio for the PL

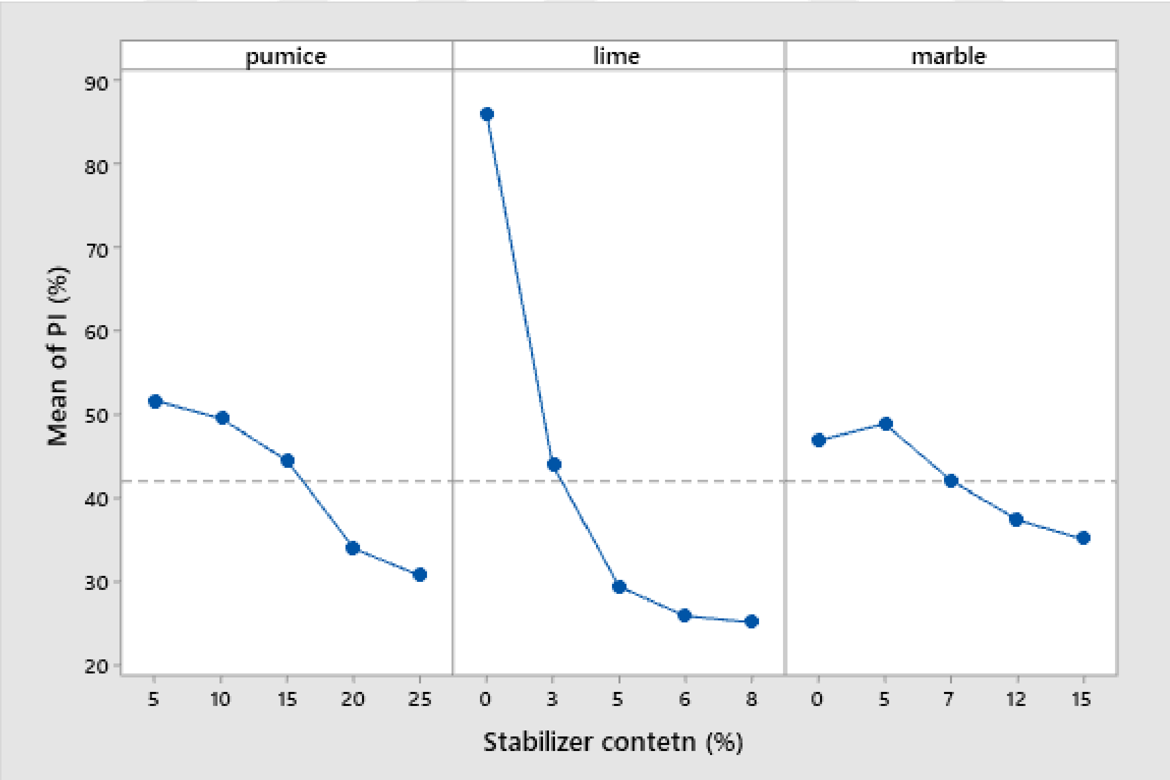

Figure 3.11. Parameter effects on mean of plasticity index

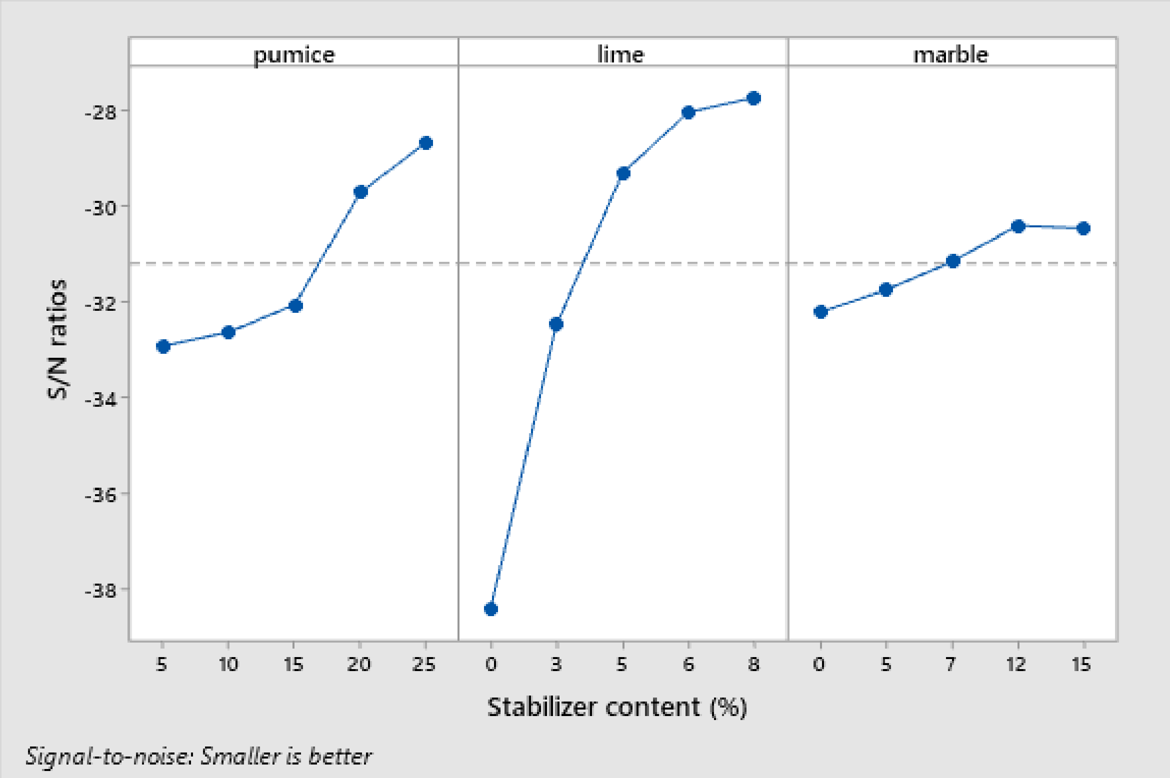

Figure 3.12. Parameter effects on mean S/N ratio for the PI

Table 3.3. Consistency limits for second-stage design experiments

Trial no.	Name	LL (%)	PL (%)	PI (%)
-	S	143.6	38.8	104.8
1	SP20L6M12	74.0	56.6	17.5
2	SP20L8M15	70.3	54.6	15.7
3	SP25L6M15	72.3	57.9	14.3
4	SP25L8M12	71.8	57.2	14.6
5	SP20L8M12	74.3	55.9	18.4

Based on the results shown in tables 3.2 and 3.3, the plasticity indexes decreased for all mixtures except for SP10M5 and SP5 whose plasticity indexes slightly increased due to the decrease in their plastic limits. In addition, the liquid limits for all mixtures reduced except for SP10M5, which insignificantly increased by 0.3%. However, the plastic limits generally increased in most of the mixtures due to the usage of lime. It was observed that the maximum reduction of liquid limit values occurred when the SP20L8M15 mixture was used, which was a mixture of 20% pumice in addition to 8% lime and 15% marble. This mixture reduced the liquid limit by 51%, whereas its plasticity index was reduced by 85%. However, the maximum decrease in plasticity index was in the case of SP25L6M15, which had a plasticity index of 14.3 and a liquid limit of 72.3%.

3.2. Effect of Stabilizers on Compaction Parameters

Based on the compaction parameters listed in table 3.4 and shown as a bar graph in figures 3.13 and 3.14, it is observed that the maximum dry density for mixtures is greater while the optimum water content values are less compared with the soil sample. In addition, the compaction curves for each additive are shown in figures 3.15-3.19. The maximum dry density is 1.29 g/cm^3 for the SP20L8M12 mixture, which also has the minimum optimum water content. The optimum water content values are between 29.6% and 32.8% for mixtures while it reaches 37.4% for the high-plasticity clay soil. Moreover, the maximum dry densities are between 1.26g/cm^3 and 1.29g/cm^3 for mixtures while it equals 1.23% for soil.

Table 3.4. Compaction parameters for soil and mixtures

Trial no.	**Name**	**Maximum dry density (g/cm^3)**	**Optimum water content (%)**
-	S	1.23	37.4
1	SP20L6M12	1.26	32.8
2	SP20L8M15	1.28	30.8
3	SP25L6M15	1.272	30
4	SP25L8M12	1.272	30.8
5	SP20L8M12	1.29	29.6

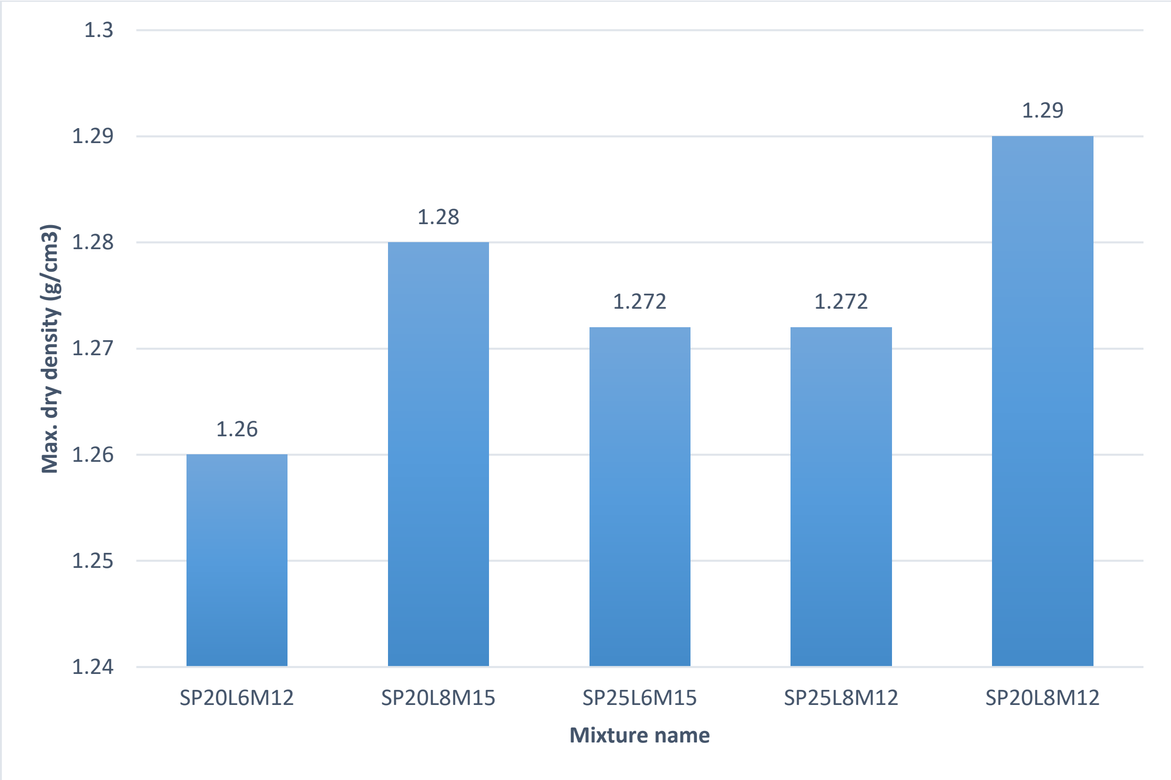

Figure 3.13. Maximum dry density for mixtures

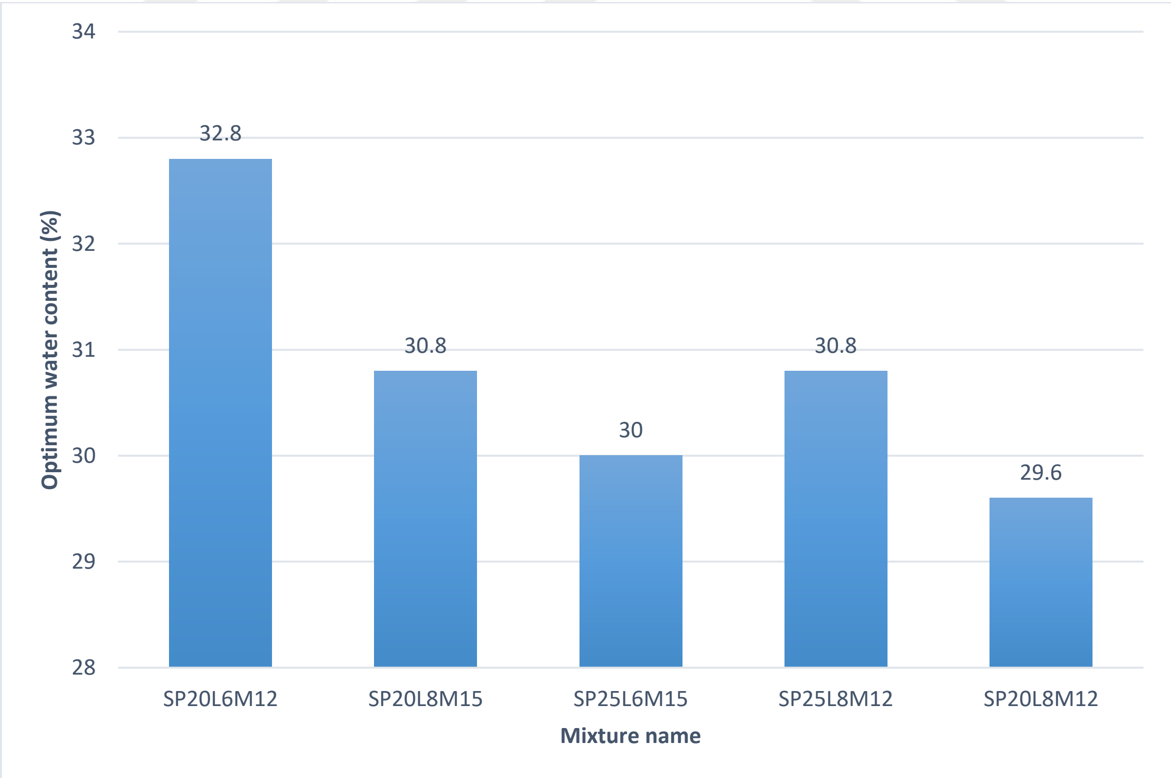

Figure 3.14. Optimum water content for mixtures

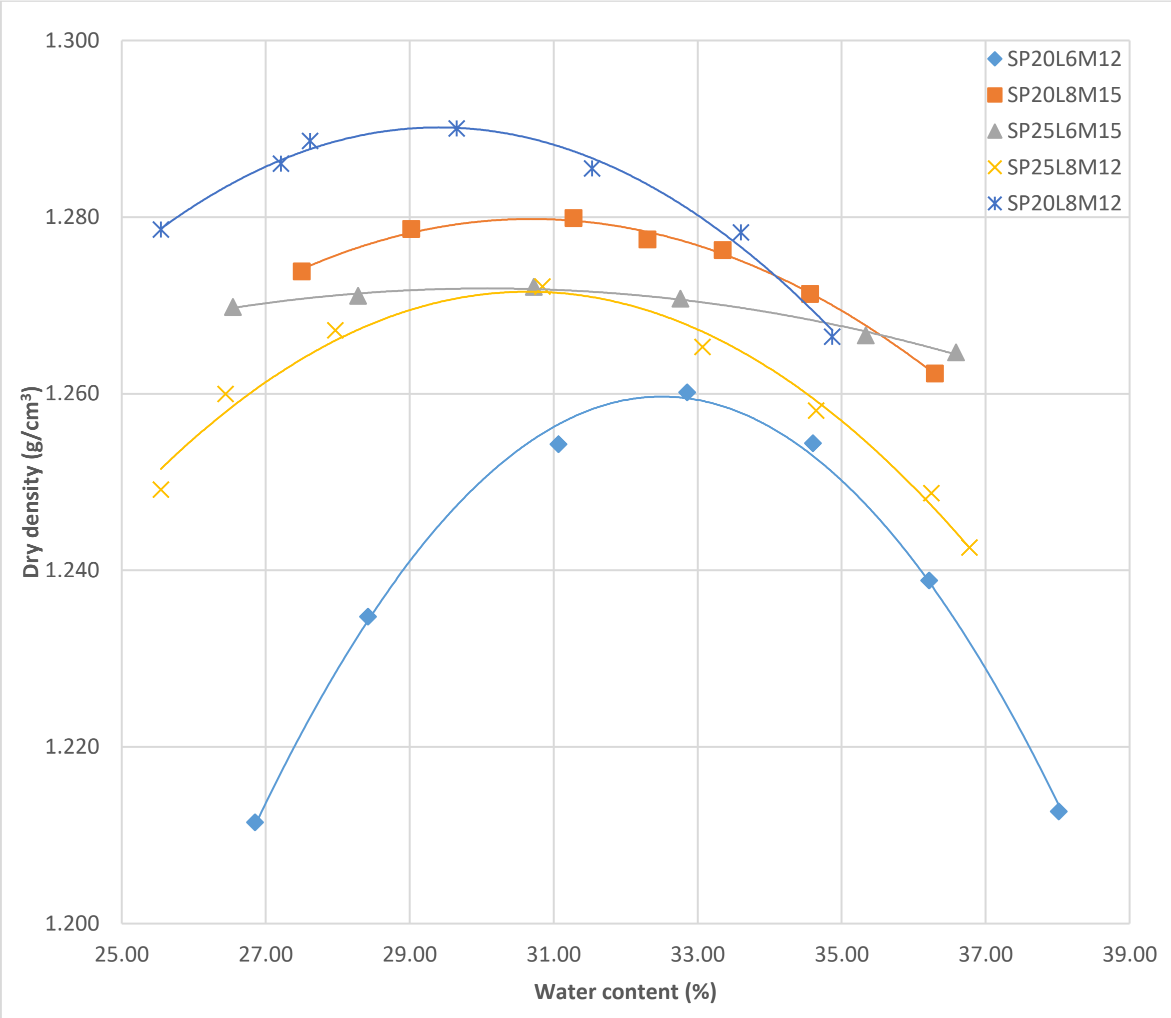

Figure 3.15. Compaction curves for mixtures

3.3. Effect of Stabilizers on Unconfined Compressive Strength

Based on the results of the UCS test for soil and mixtures listed in table 3.5 and shown in figure 3.16, it is clear that all mixtures enhanced the UCS value. In addition, the UCS results were affected by the curing time due to the pozzolanic reactions. The maximum increase of UCS belongs to the SP25L8M12 mixture for curing times of 1, 7, and 28 days. It is observed that mixtures with higher pumice content have higher unconfined compressive strength. SP25L8M12 mixture increased the UCS up to 632.9, 1938.86, and 4709.46 kPa for curing times of 1day, 7days, and 28 days, respectively. However, the minimum increase of UCS belongs to SP20L6M12, which increased the UCS up to 409.32, 1702.5, and 2671.63 kPa for 1day, 7days, and 28 days curing time, respectively. The stress-strain curves for each additive at different curing times are illustrated in figures 3.17-3.21. The UCS of the 1-day cured SP20L6M12 mixture was 409.34 kPa. This value increased by 79.75 kPa for the SP20L8M12 mixture when the lime content was increased by 2%. In addition, it was increased by 223.56 kPa for the SP25L8M12 mixture when the lime and pumice contents were increased by 2% and 5%, respectively. However, the UCS of the 1-day cured SP20L8M15 mixture was 458.35 kPa which increased by 30.65 kPa when the marble content was decreased to 12% for the SP20L8M12 mixture.

The UCS of the 7-day cured SP20L8M12 mixture was 1798.05 kPa. This value decreased to 1772.4 kPa for the SP20L8M15 mixture when the marble content was increased by 3%. Furthermore, it increased to 1938.86 kPa for the SP25L8M12 mixture when the pumice content was increased by 5%. Nevertheless, it decreased to 1702.05 kPa for the SP20L6M12 mixture when the lime content was decreased to 6%.

The UCS of the 28-day cured SP20L6M12 mixture was 2671.63 kPa which increased the UCS for soil by 2323.53 kPa. This value increased to 4709.46 kPa for the SP25L8M12 mixture when the lime and pumice contents were increased by 2% and 5%, respectively. Besides, it increased to 3476.5 kPa for the SP20L8M12 mixture when the lime content was increased to 6%. Despite this, the UCS of the 28-day cured SP20L8M15 mixture was 2880.61 kPa which increased to 3476.5 kPa when the marble content was decreased by 3% for the SP20L8M12 mixture.

Table 3.5. UCS results for soil with additives

Trial no.	**Name**	**q_u (kPa)**		
		1 day cured	7 days cured	28 days cured
-	S	327.17	367.07	348.10
1	SP20L6M12	409.34	1702.05	2671.63
2	SP20L8M15	458.35	1772.4	2880.61
3	SP25L6M15	608.18	1853.74	4165.18
4	SP25L8M12	632.9	1938.86	4709.46
5	SP20L8M12	489.09	1798.05	3476.5

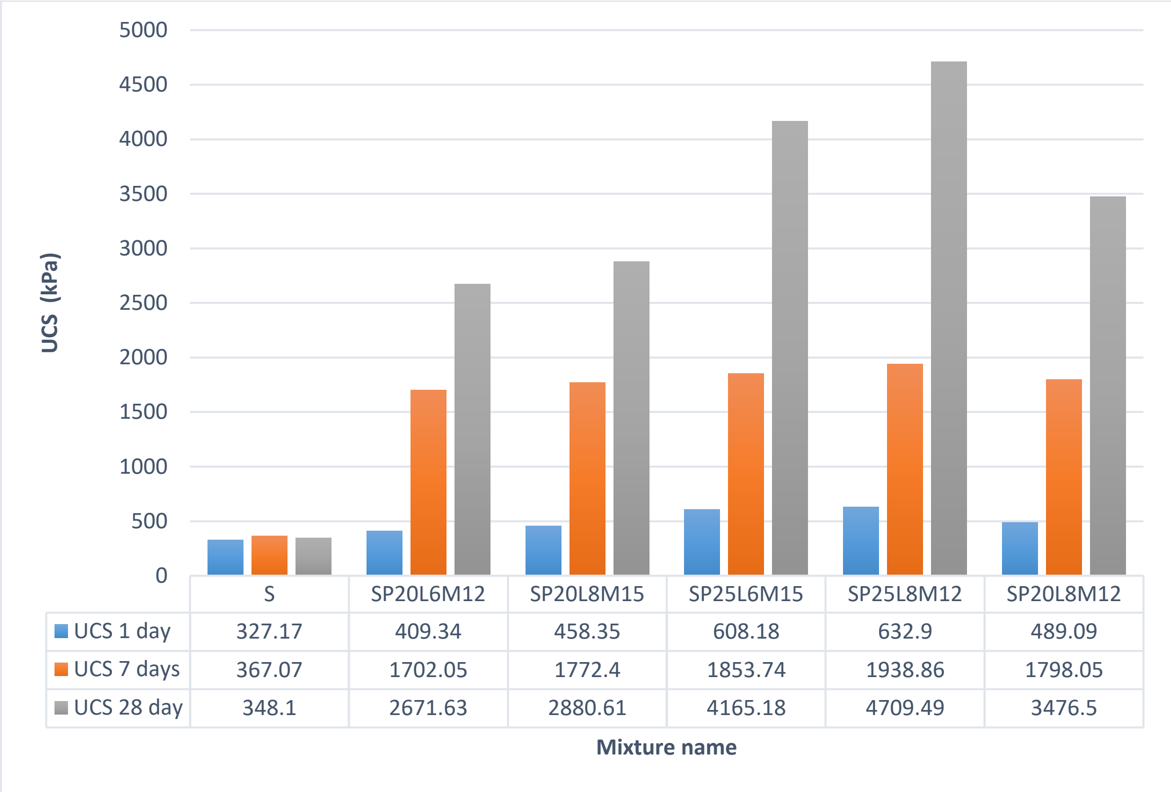

Figure 3.16. Results of UCS for soil with additives

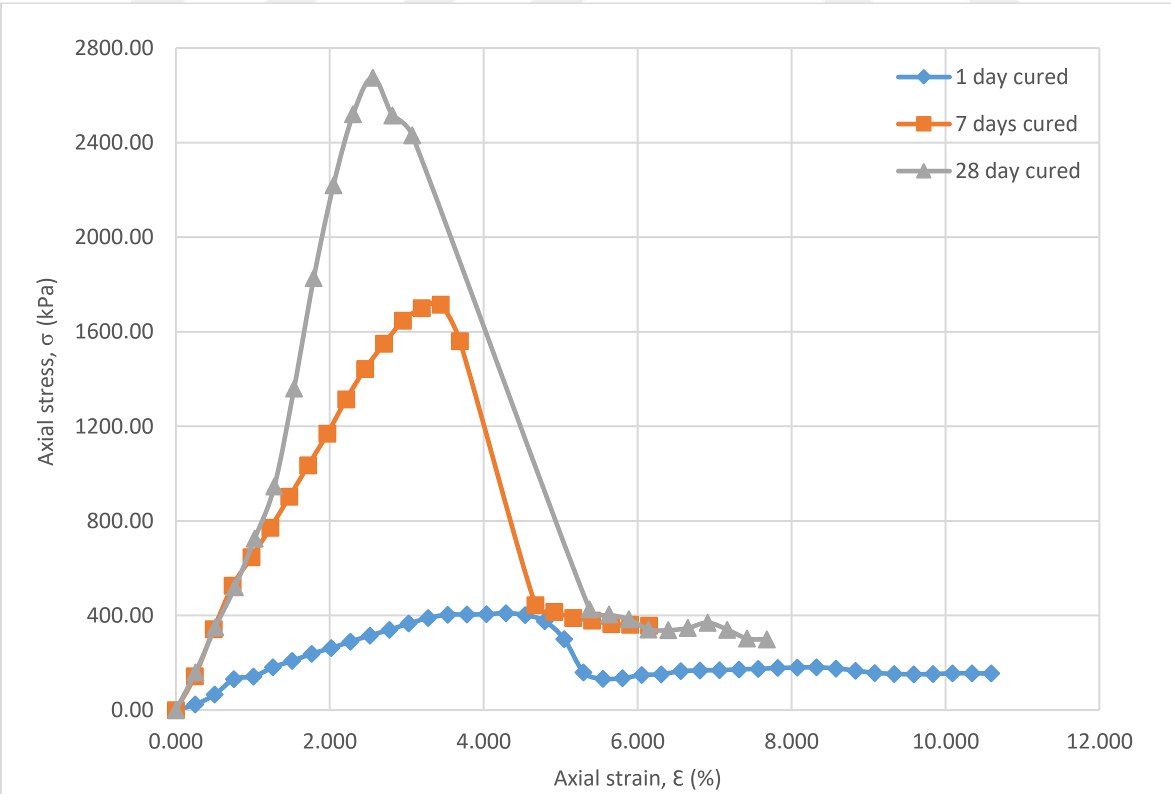

Figure 3.17. Stress-strain curves for SP20L6M12

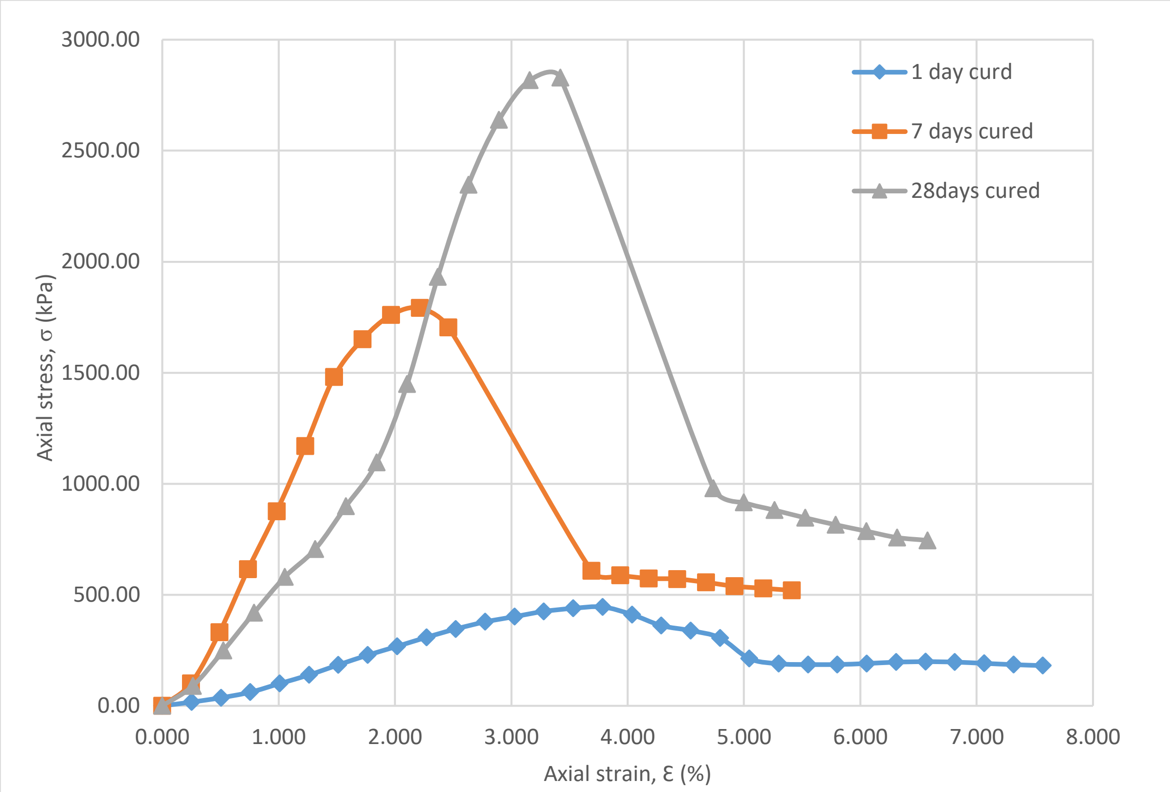

Figure 3.18. Stress-strain curves for SP20L8M15

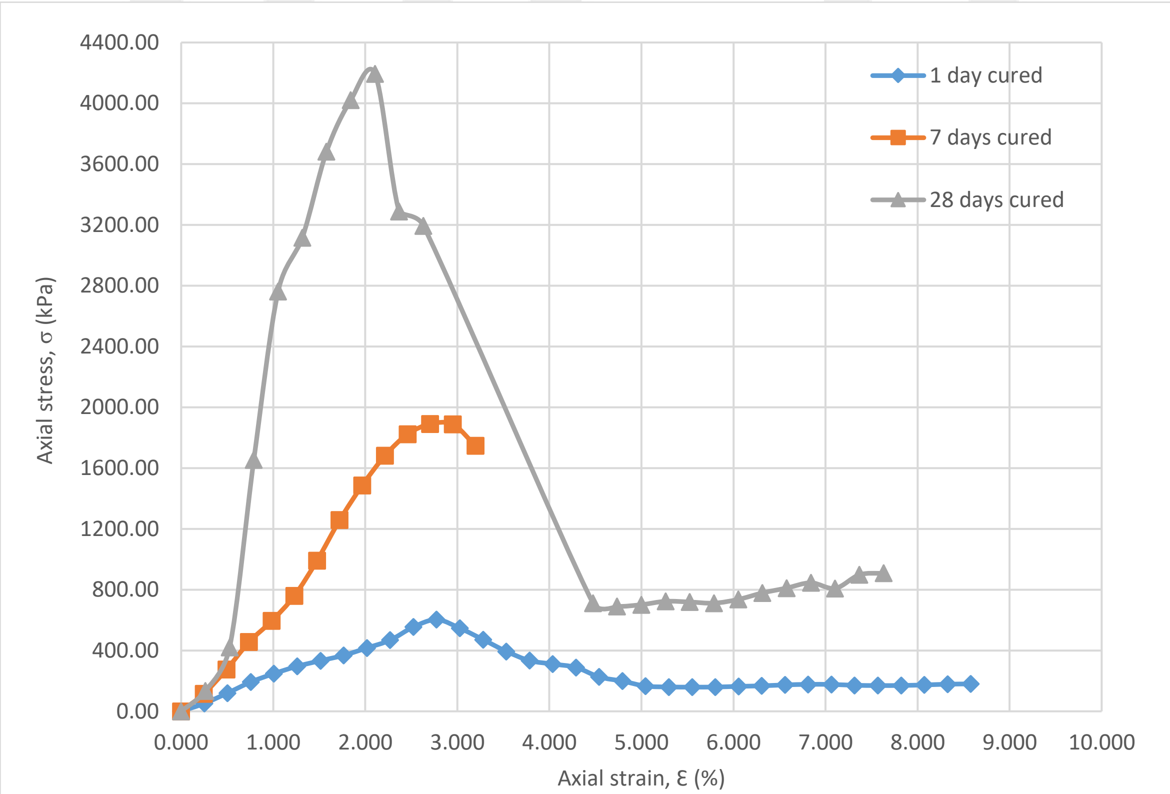

Figure 3.19. Stress-strain curves for SP25L6M15

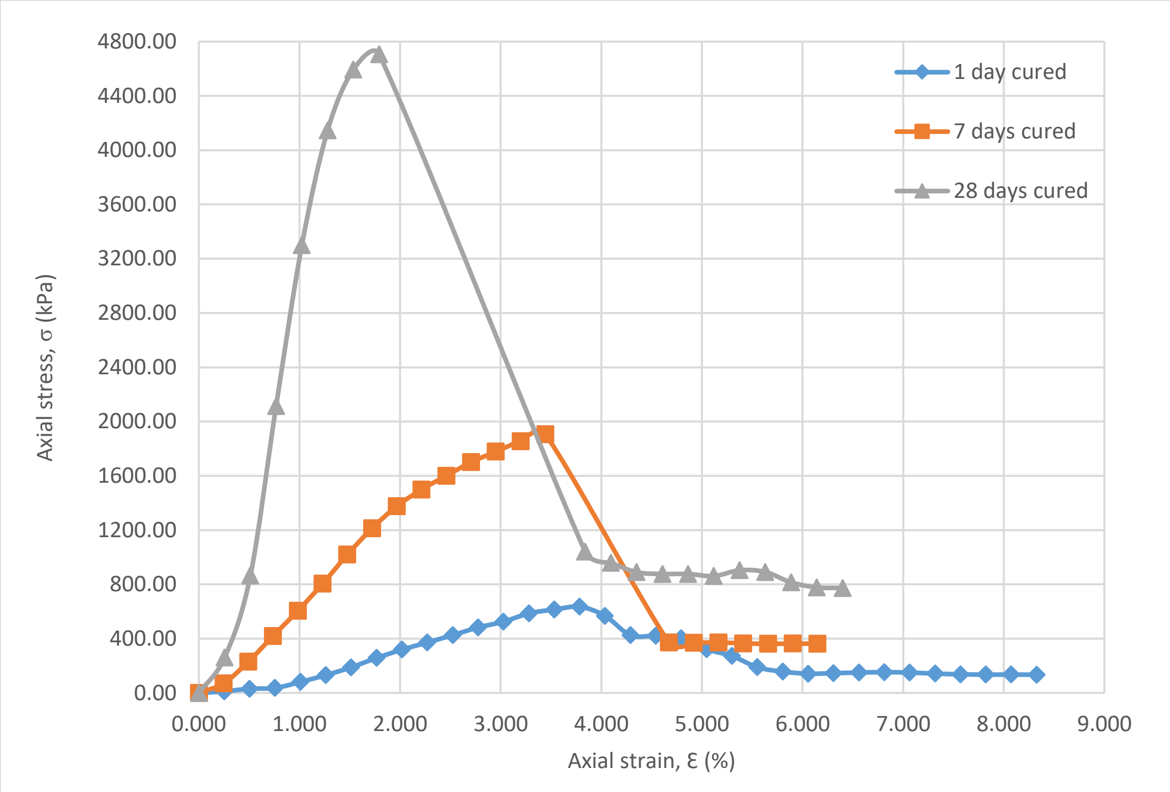

Figure 3.20. Stress-strain curves for SP25L8M12

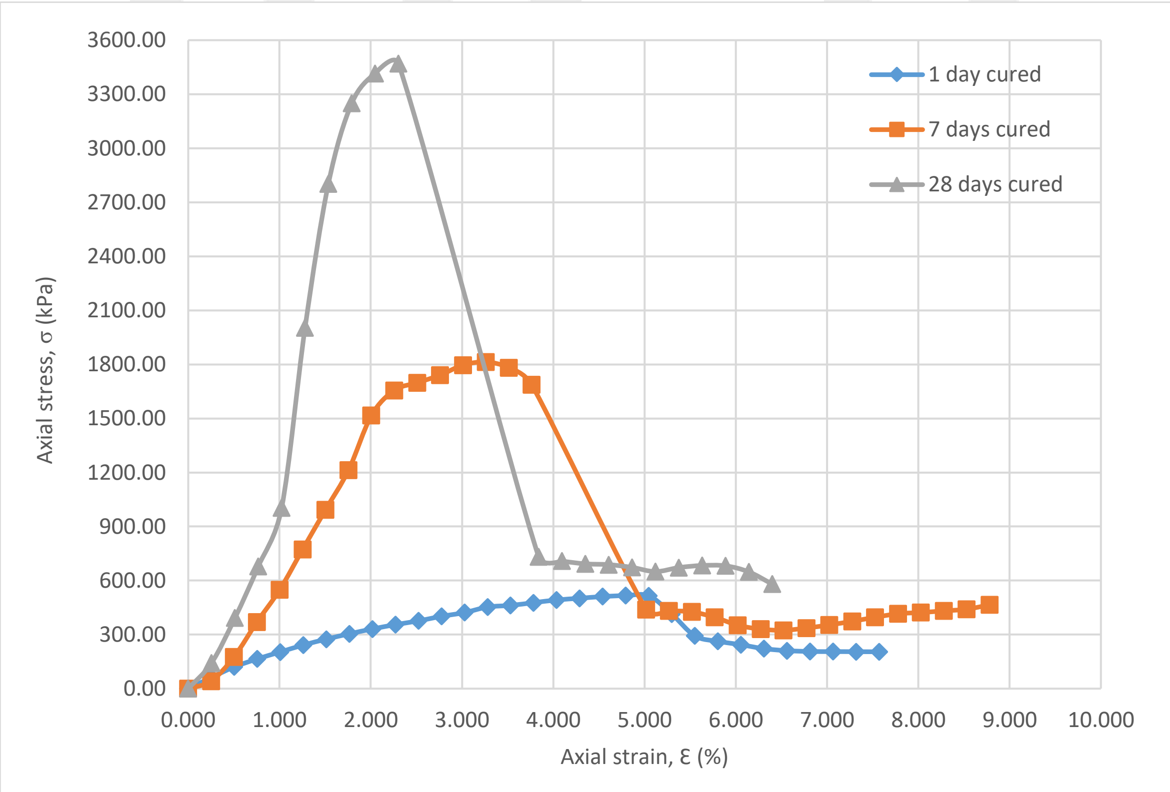

Figure 3.21. Stress-strain curves for SP20L8M12

Based on the analysis of the Taguchi method for UCS that is shown in figures 3.22-3.27, the best ratios for additive to enhance the strength for 1-day curing time, are 25% for pumice, 8% for lime, and 15% for marble. However, the analysis of 7 days and 28 days curing time yields 12% as the best ratio for marble. It is clear that the most efficient parameter to affect the UCS at different curing times is pumice since the UCS increases with increasing pumice ratio. On the other hand, with the increase of marble content, there is a decrease in the UCS at 7 and 28 days. Generally, it is clear that the optimum mixture for UCS is SP25L8M12.

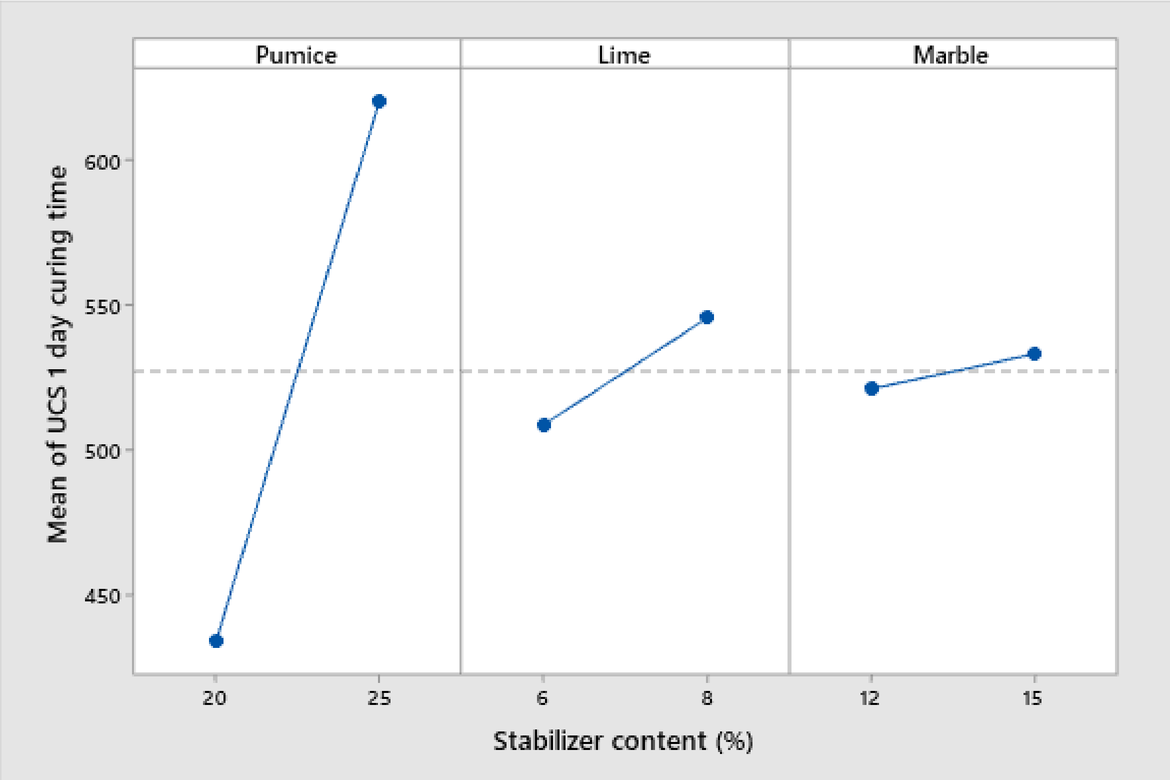

Figure 3.22. Parameter effects on mean UCS for 1 day curing time

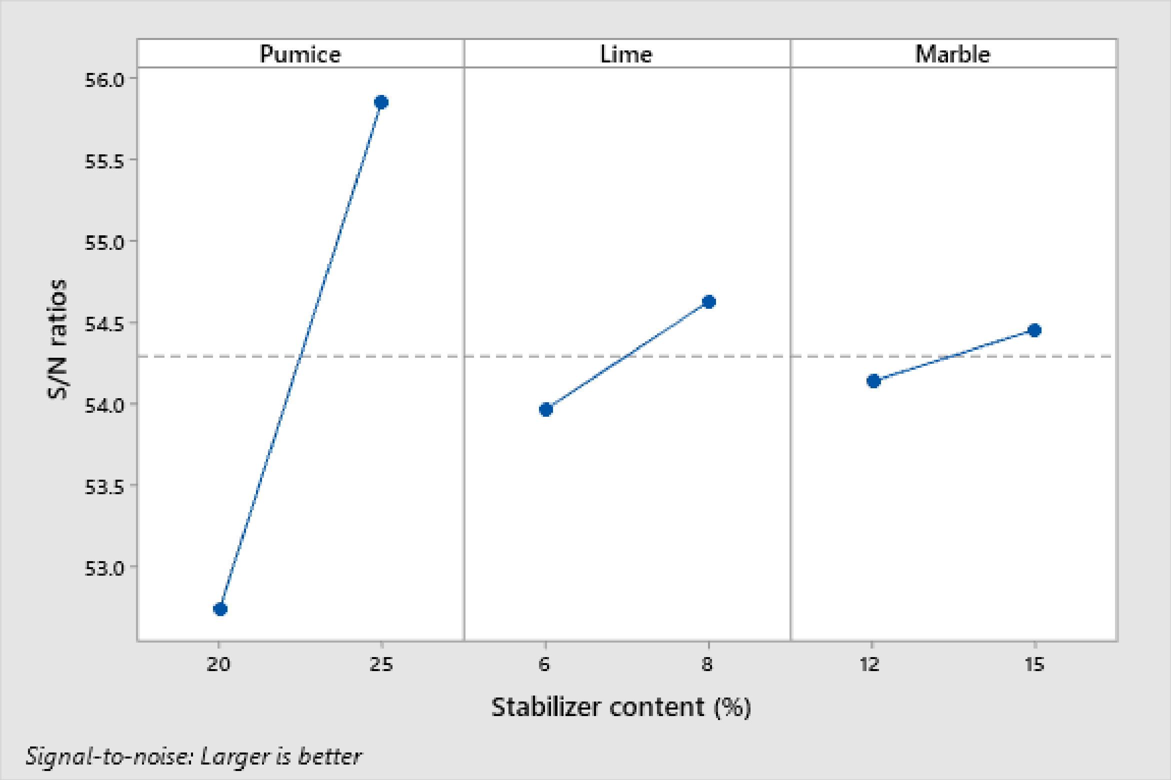

Figure 3.23. Parameter effects on mean S/N ratio for the UCS for 1 day curing time

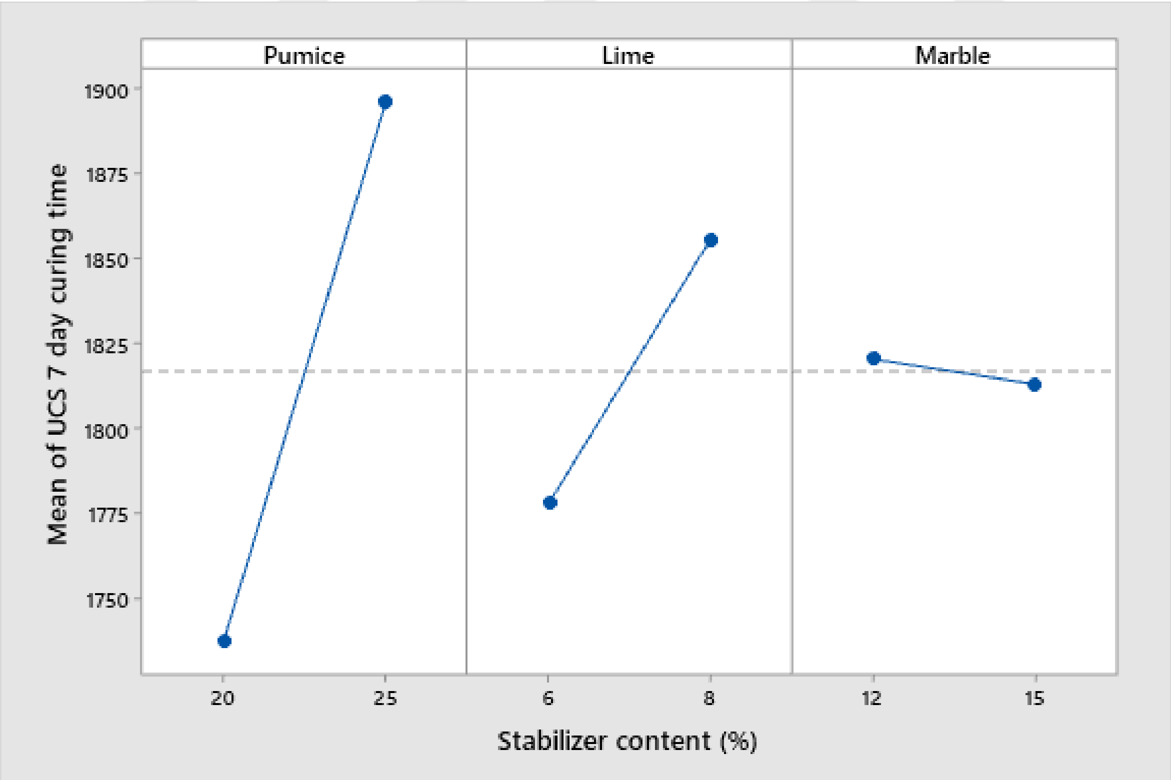

Figure 3.24. Parameter effects on mean UCS for 7 days curing time

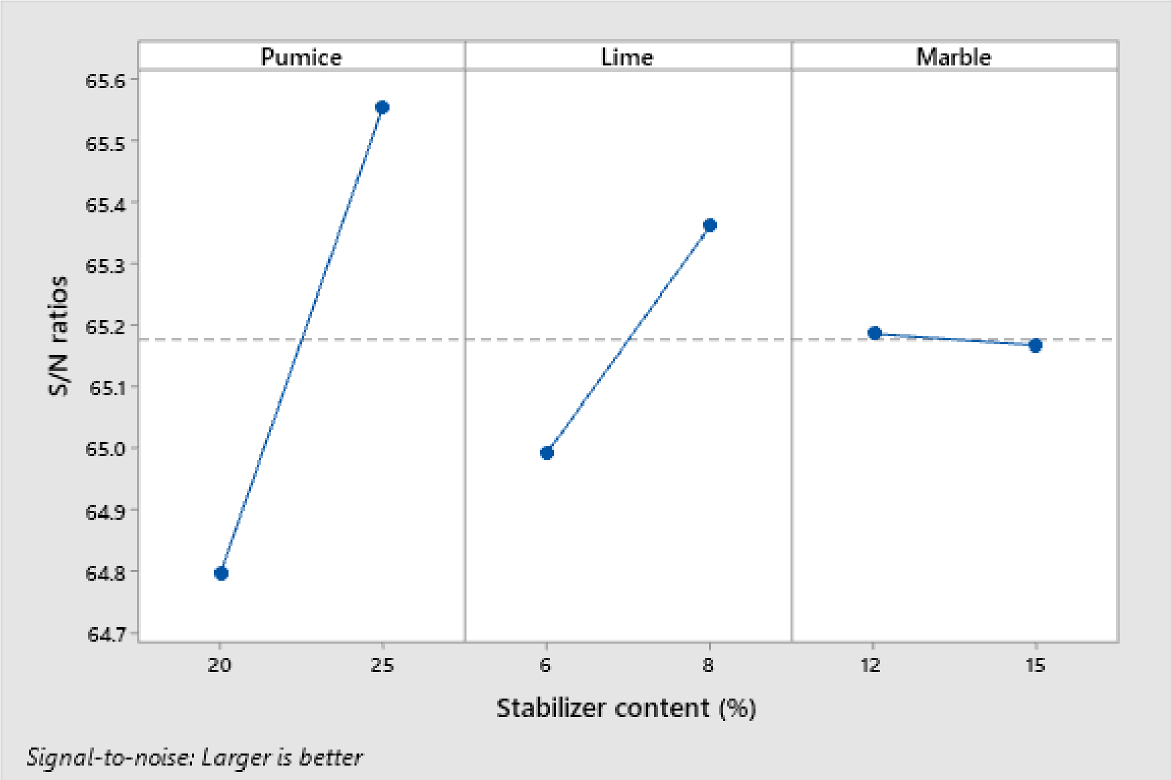

Figure 3.25. Parameter effects on mean S/N ratio for the UCS for 7 days curing time

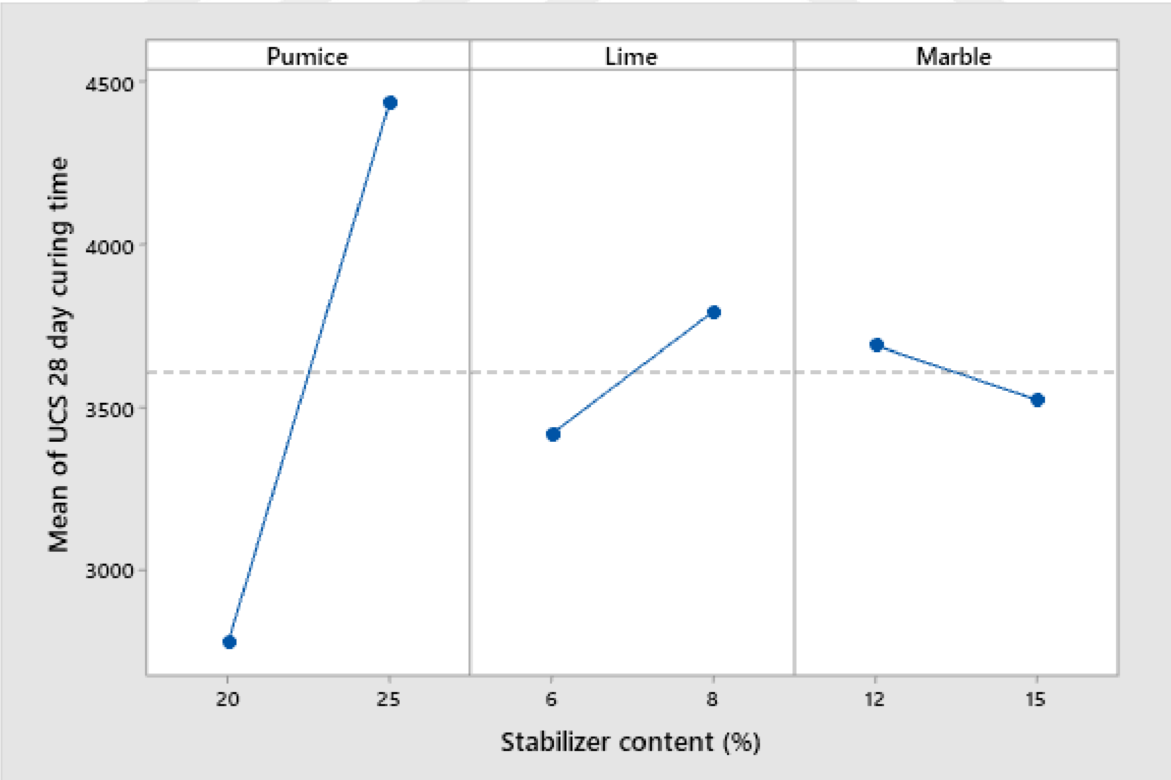

Figure 3.26. Parameter effects on mean UCS for 28 days curing time

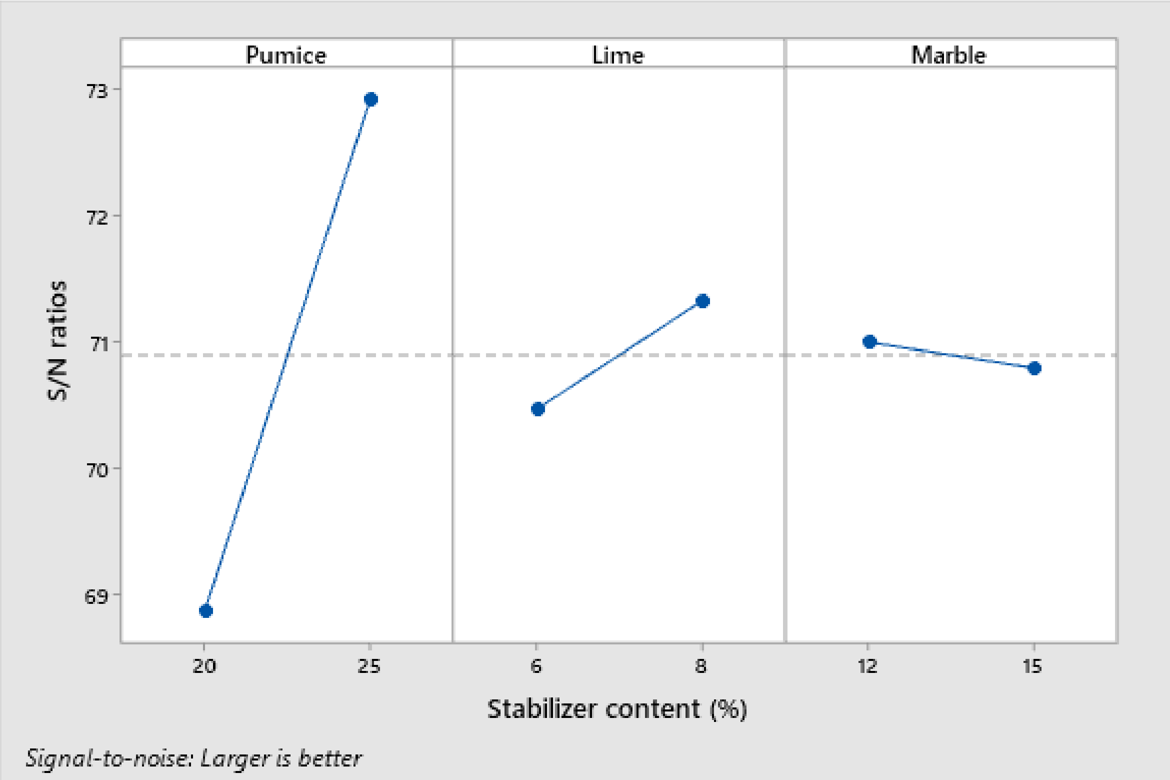

Figure 3.27. Parameter effects on mean S/N ratio for the UCS for 28 days curing time

3.4. Effect of Stabilizers on Swelling Parameters

The swell percentage and swelling pressure were determined using the oedometer apparatus. The results for soil and mixtures are presented in table 3.6 and figures 3.28 and 3.29. In addition, the time-swell curves for each mixture are shown in figure 3.30, while the vertical stress vs. void ratio curves are shown in figures 3.31-3.35. Based on the results, it is observed that all mixtures significantly enhanced the swell percentage as the minimum reduction in swell percentage is 97.7% for the SP20L8M12 mixture. The swell percentages of mixtures range between 0.51% and 2.05%. The percentage of swell is 1.47% for the SP20L6M12 mixture. This percentage decreases to 0.51% for the SP25L8M12 mixture when the lime and pumice contents are increased by 2% and 5%, respectively. Additionally, it decreases to 1.31% for the SP20L8M15 mixture when the lime and marble contents are increased by 2% and 3%, respectively. Moreover, it decreases to 0.65% for the SP25L6M15 mixture when the marble and pumice contents are increased by 3% and 5%, respectively. Notwithstanding, it increases to 2.05% when the lime content is increased to 8% for the SP20L8M12 mixture.

The mixtures decreased the swelling pressure for untreated soil, which was 550 kPa. The swelling pressure for the SP25L6M15 mixture and the SP25L8M12 mixture was 17 kPa and 18 kPa, respectively. However, the swelling pressure was higher for the other mixtures compared to their swell percentage which might be due to their high UCS that led to unexpectedly high swelling pressure compared to their low swell percentage.

Table 3.6. Swelling parameters for soil and mixtures

Name	**Swell percentage (%)**	**Swell pressure (kPa)**
S	62.17	580
SP20L6M12	1.47	130
SP20L8M15	1.31	95
SP25L6M15	0.65	17
SP25L8M12	0.51	18
SP20L8M12	2.05	500

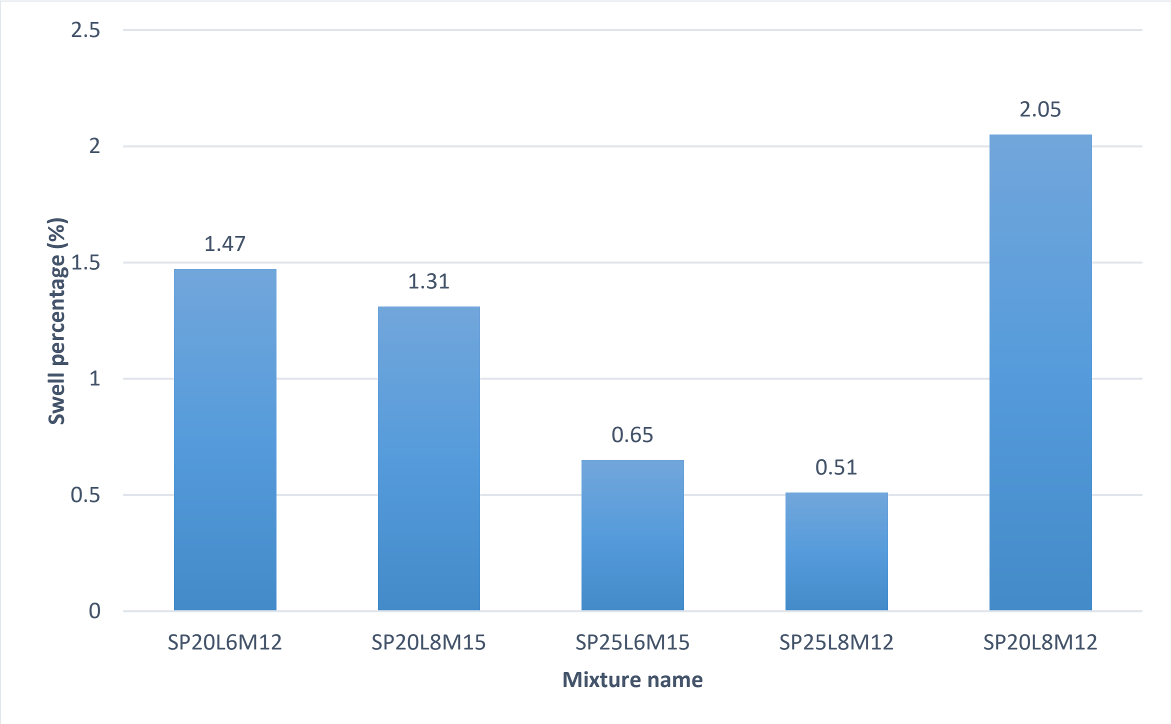

Figure 3.28. Swell percentage results for soil with additives

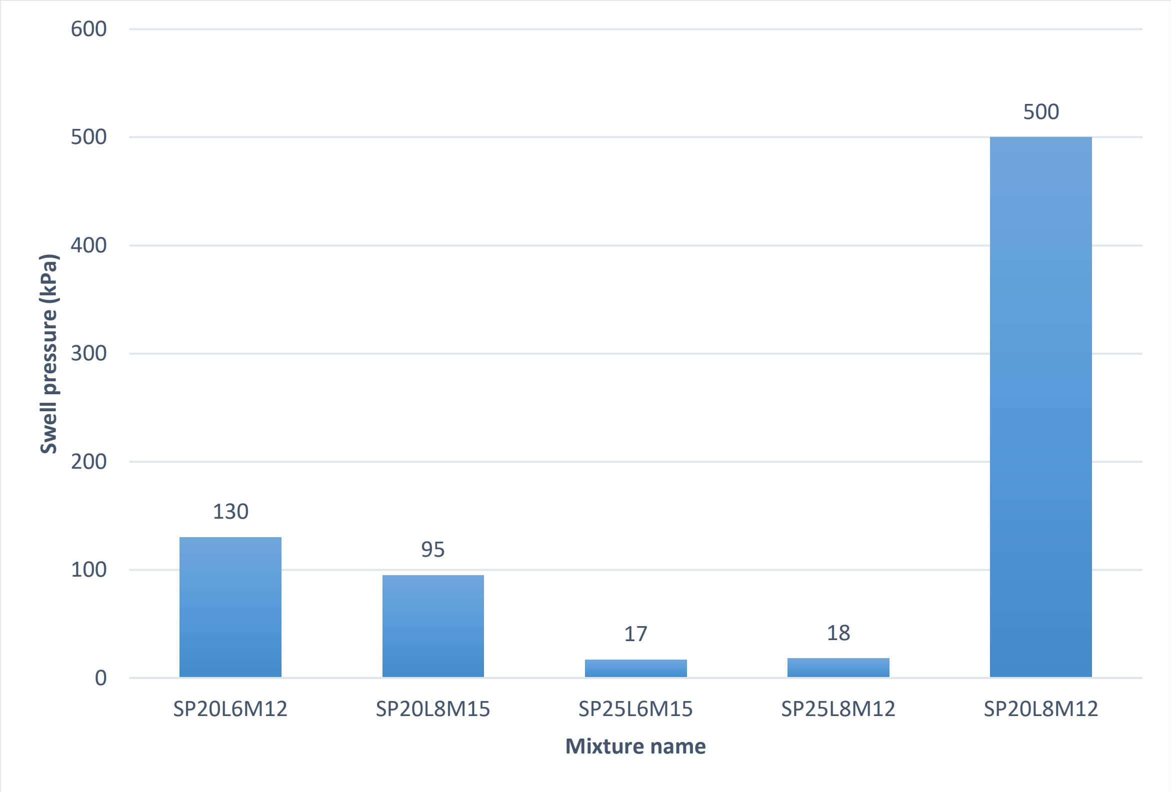

Figure 3.29. Swell pressure for additives

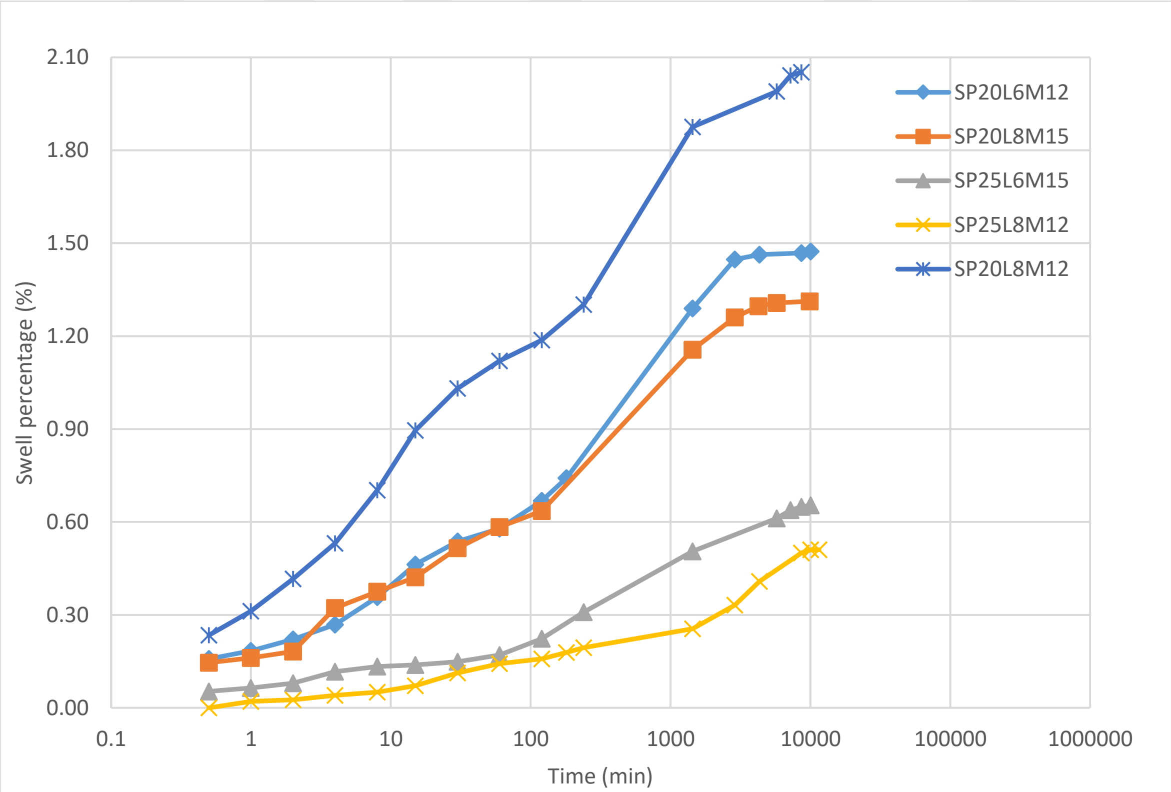

Figure 3.30. Time- swell curves for mixtures

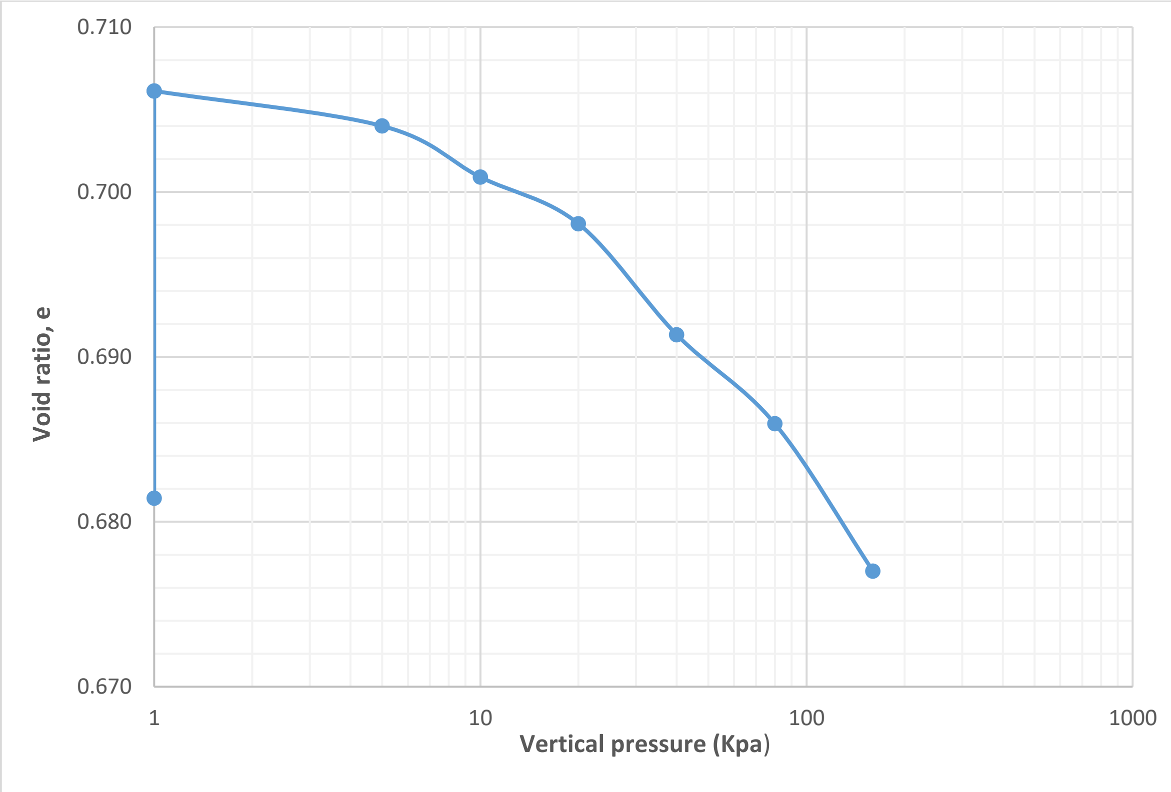

Figure 3.31. Vertical pressure vs. void ratio for SP20L6M12

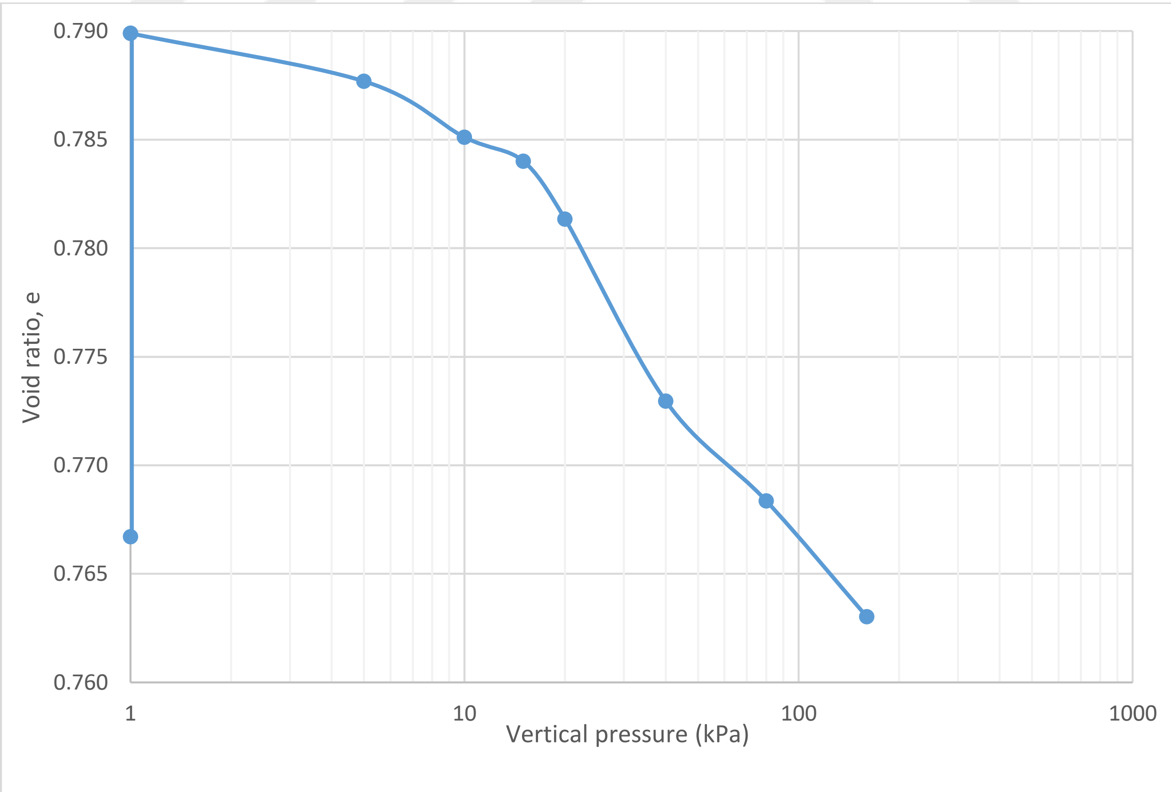

Figure 3.32. Vertical pressure vs. void ratio for SP20L8M15

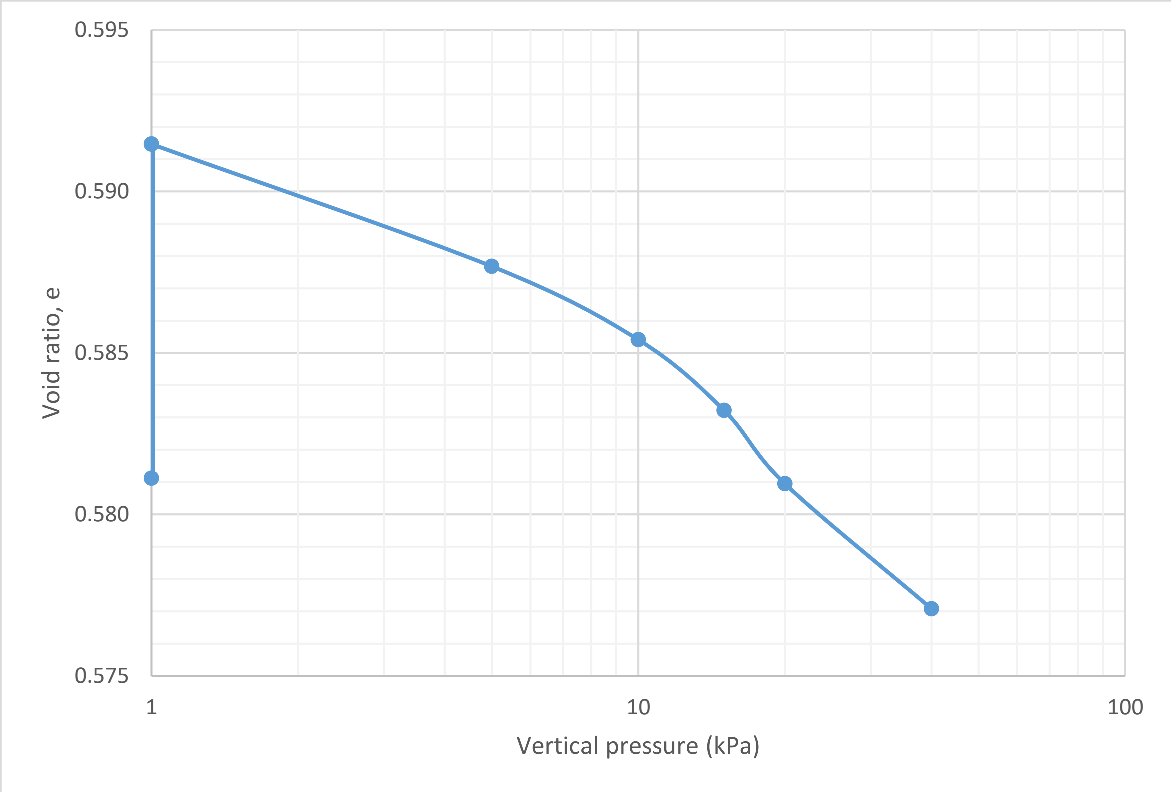

Figure 3.33. Vertical pressure vs. void ratio for SP25L6M15

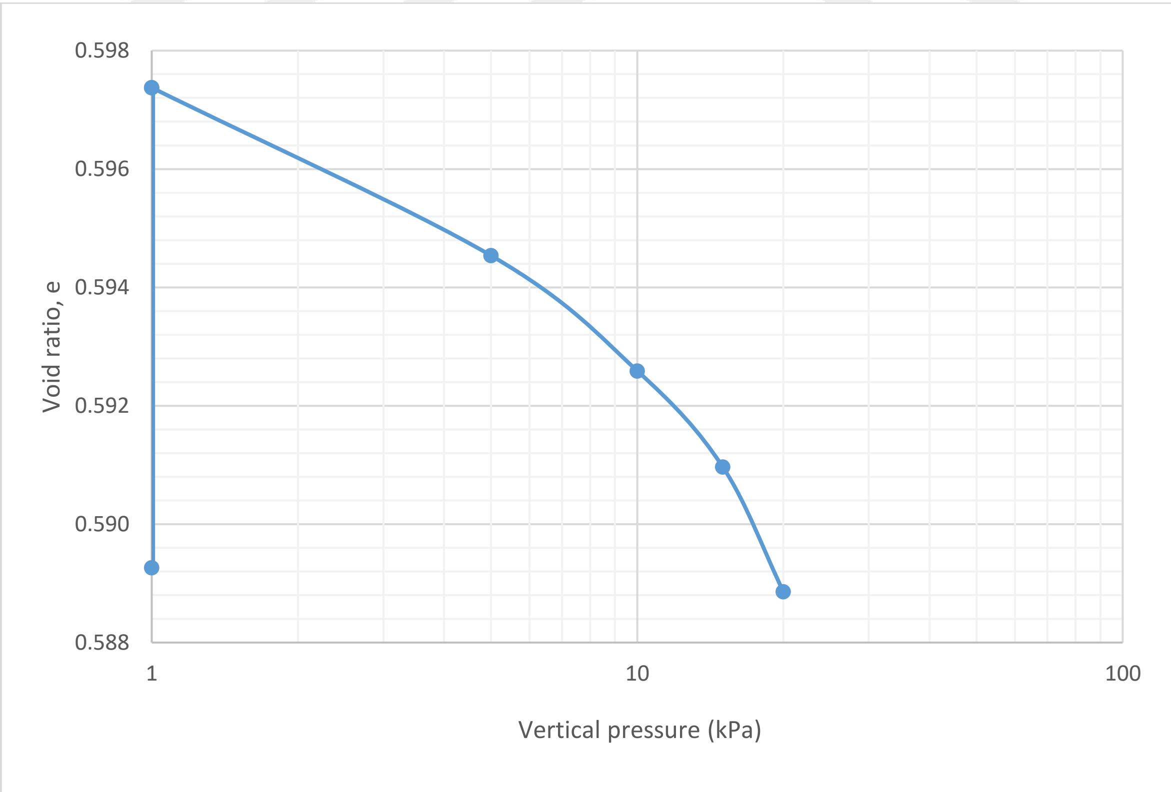

Figure 3.34. Vertical pressure vs. void ratio for SP25L8M12

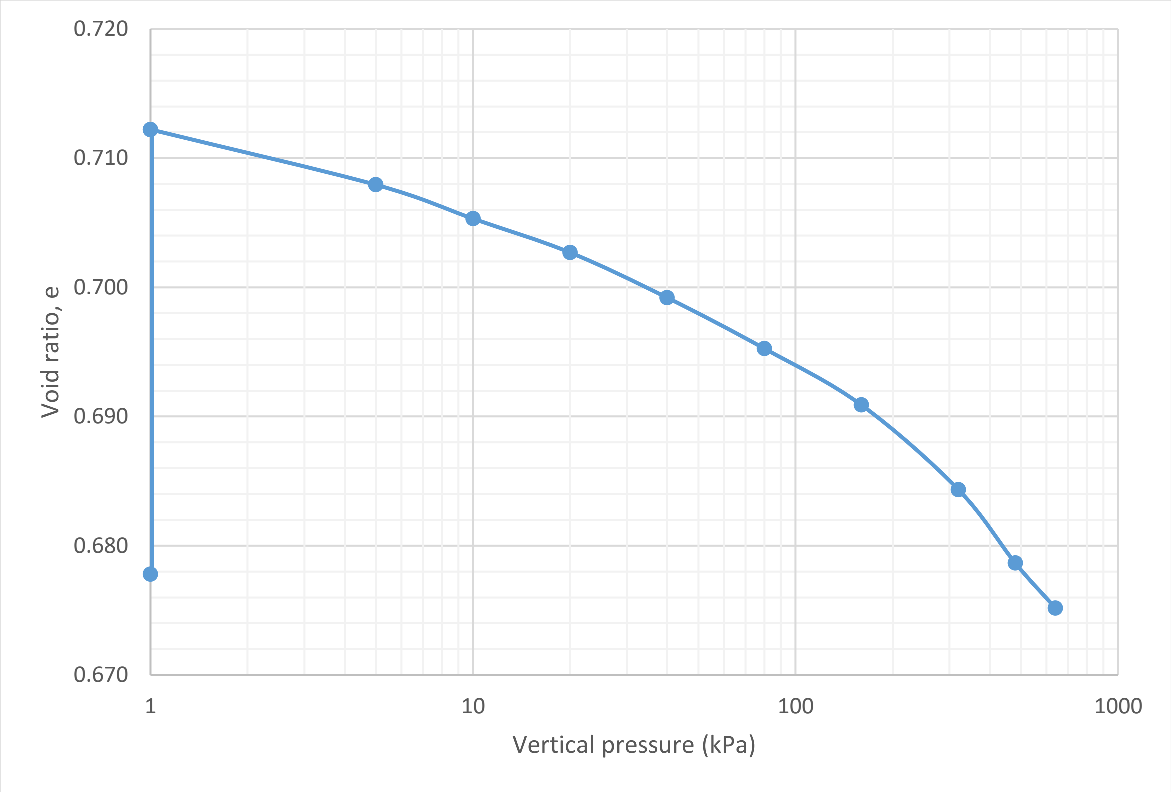

Figure 3.35. Vertical pressure vs. void ratio for SP20L8M12

Based on the analysis of the Taguchi method for swelling parameters shown in figures 3.36-3.39, the best ratios for additive to decrease the swell percentage are 25% for pumice, 8% for lime, and 12% for marble. It is clear that the most effective parameter to enhance the swell percentage is pumice since the swell percentage reduces with increasing the pumice ratio. On the other hand, an increase in the marble content results in an increase in the swell percentage. Generally, it is observed that the optimum mixture for enhancing swell percentage is SP25L8M12. In addition, for the swelling pressure pumice is the most effective parameter to decrease the swelling pressure while the effect of lime is insignificant as shown in figure 3.39. However, increasing marble content leads to a decrease in the swelling pressure as shown in figures 3.38 and 3.39 in contrast with its effect on swell percentage.

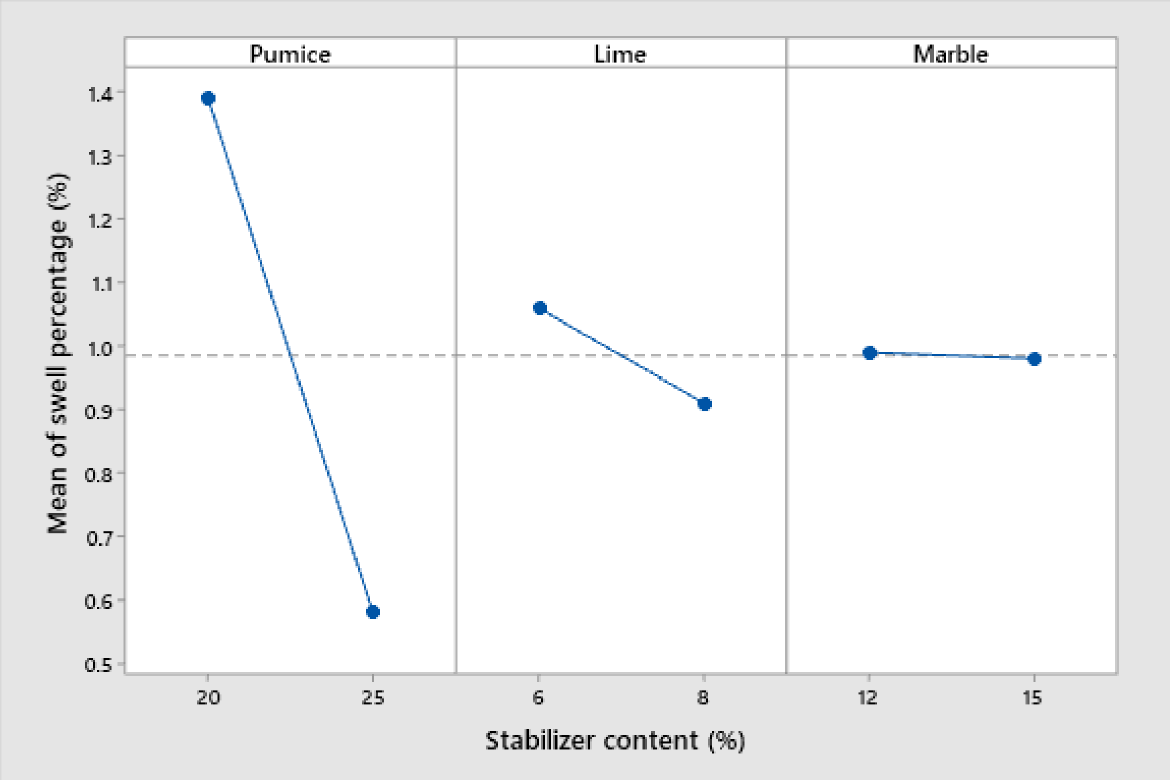

Figure 3.36. Parameter effects on the mean for swell percentage

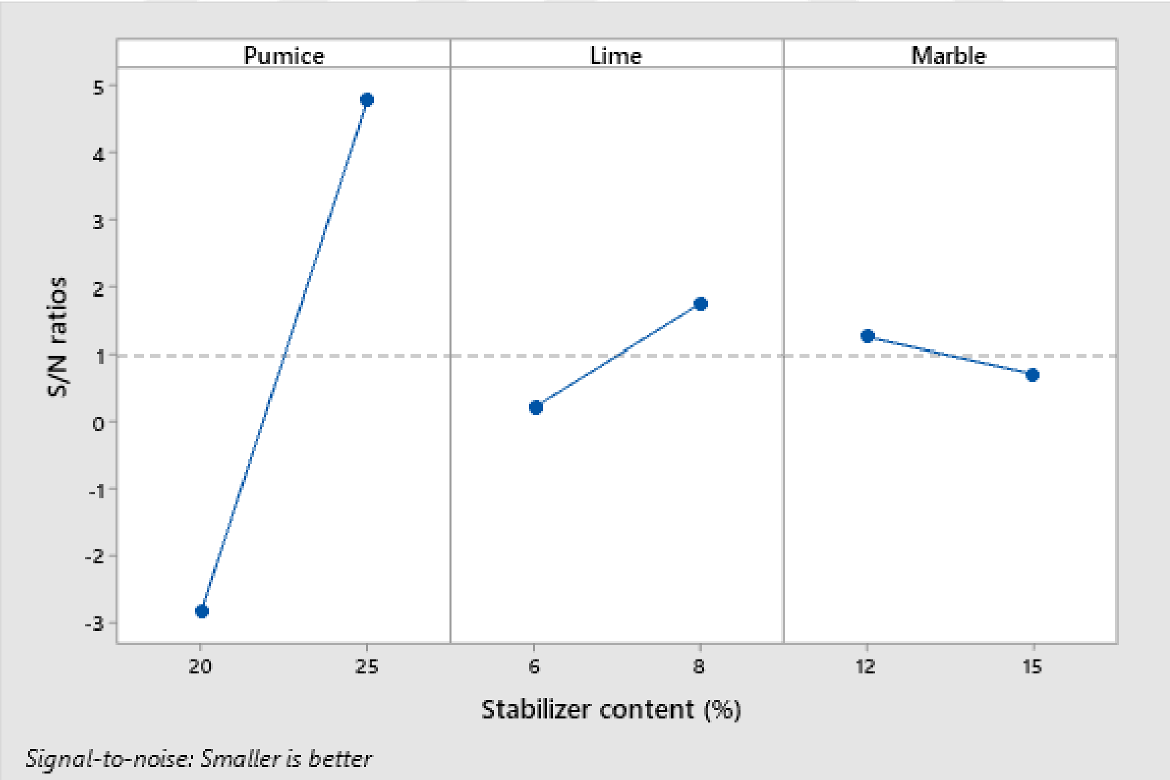

Figure 3.37. Parameter effects on mean S/N ratio for swell percentage

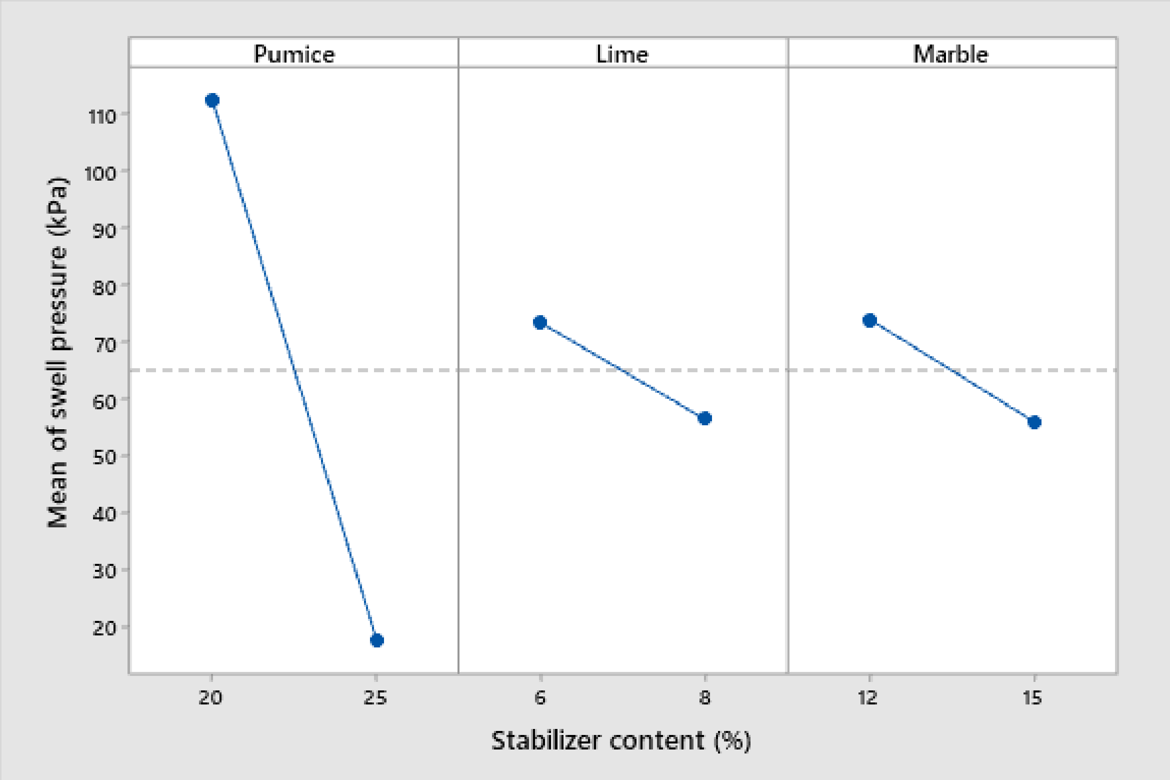

Figure 3.38. Parameter effects on the mean for swell pressure

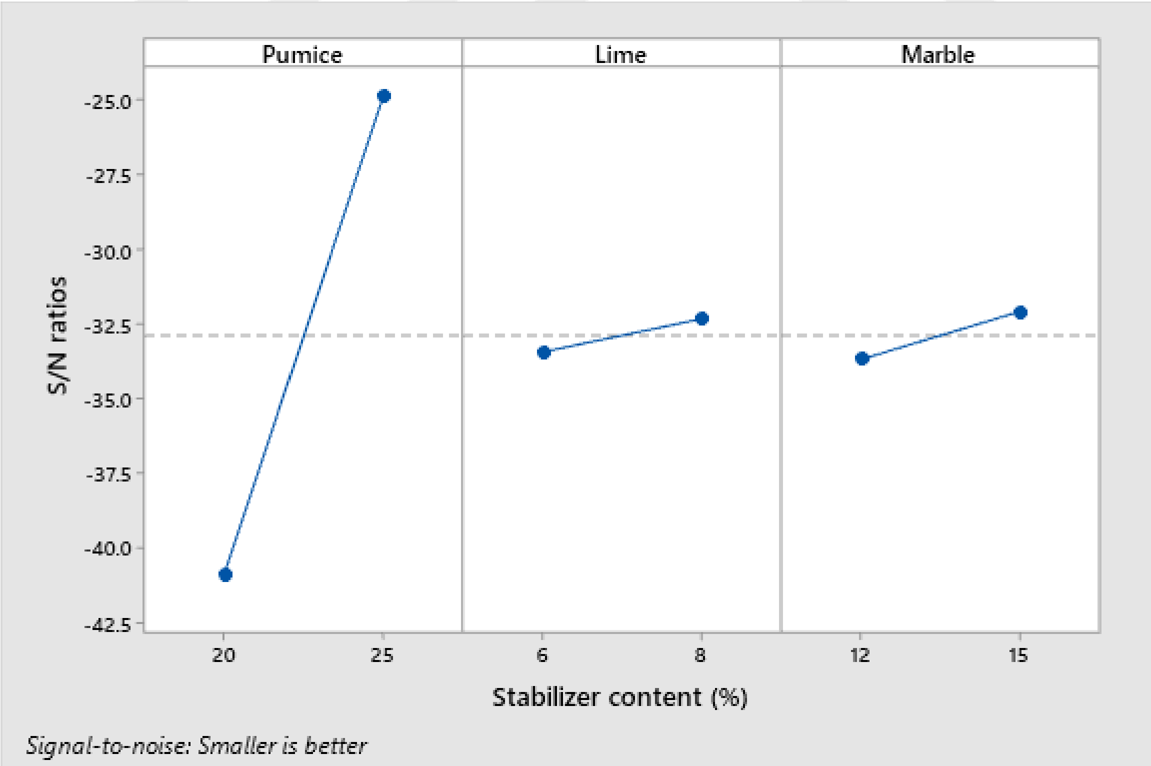

Figure 3.39. Parameter effects on mean S/N ratio for swell pressure

3.5. Effect of Predicting Results Using Taguchi Method

Taguchi method was also used to predict the results of mixtures that were not tested but consist of similar material levels as those used in the design experiments. In order to ascertain the applicability of this method for the prediction of the results, confirmatory tests were conducted using 9 trials for the first stage (table 3.7) and 1 trial for the second stage (table 3.8).

Table 3.7. Results of the confirmatory tests for the first stage

Name	**LL (%)**			**PL (%)**			**PI (%)**		
	Actual	predicted	Error (|%|)	Actual	predicted	Error (|%|)	Actual	predicted	Error (|%|)
SP10	133.07	134.05	0.74	36.71	35.87	2.29	96.36	98.18	1.89
SP15	127.97	129.65	1.31	40.57	36.56	9.88	87.4	93.09	6.51
SP20	116.67	120.16	2.99	38.53	37.51	2.65	78.14	82.65	5.77
SP25	112.51	118.26	5.11	33.59	38.88	15.75	78.92	79.38	0.58
SP20L6M12	74.03	73.16	1.18	56.56	60.02	6.12	17.47	13.14	24.79
SP20L8M15	70.31	66.65	5.21	54.62	56.61	3.64	15.69	10.04	36.01
SP25L6M15	72.25	68.25	5.54	57.93	60.71	4.80	14.32	7.54	47.35
SP25L8M12	71.82	67.77	5.64	57.21	58.66	2.53	14.61	9.11	37.65
SP20L8M12	74.3	69.66	6.24	55.9	57.3	2.50	18.4	12.36	32.83

Table 3.8. Results of the confirmatory tests of the SP20L8M12 mixture for the second stage

	q_u (kPa)			**Swelling parameters**	
	1 day cured	7 days cured	28 days cured	Swell percentage (%)	Swell pressure (kPa)
Actual	489.09	1798.05	3476.5	2.05	500
Predicted	446.21	1779.79	3048.27	1.32	113
Error (|%|)	8.77	1.02	12.32	35.61	77.40

It can be observed from the obtained values predicted by the Taguchi method that the absolute error of the tested and the predicted results changed for LL between 0.74% and 6.24%, and for PL between 2.29% and 6.12%. As for the S15 and the S25 mixtures, absolute error was 9.88% and 15.75%, respectively. However, the absolute errors for the PI ranged between 0.58% and 6.51% for the mixtures that contain pumice only, but they were too high for mixtures that contain three additives.

Based on the results of the second stage shown in table 3.8, the absolute error for UCS was 8.77% and 12.32% for 1 day and 28 days curing time, respectively. However, it was 1.02% for 7-day curing time. Regarding the swelling parameters, the results showed high errors.

The unexpected high swelling pressures of some mixtures could be the reason for the high error in predicting swell pressure using the Taguchi method. Additionally, the Taguchi method provides a linear approach for analysing and predicting the results, which could be the reason for some high errors. Furthermore, the relationship between increasing the content of an additive and its effect on any experiment is not linear. Moreover, the Taguchi method assumes that the results between successive two levels are continuously linear which is not always right. In addition, the resulted values of swell percentage and plasticity index are too small, therefore any small difference in the predicted and actual results leads to high errors. In addition, the plasticity limit is the difference between the liquid limit and the plastic limit, which leads to accumulative error. However, Taguchi method serves as a good indicator which saves time and efforts instead of conducting a large number of trials.

4. CONCLUSION

Soil stabilization with alternative materials such as pumice, lime, and marble was investigated by several laboratory experiments such as the consistency limits test, UCS, compaction test, and one-dimensional swell test. At first, the high plasticity soil (CH) was mixed with pumice at ratios of 5%, 10%, 15%, 20%, and 25%, lime at ratios of 0%, 3%, 5%, 6%, and 8%, and marble at ratios of 0%, 5%, 7%, 12%, and 15%. The consistency limits were evaluated using each additive alone with soil. Afterwards, consistency limits were tested in the first stage for 25 different mixtures using the additives based on the Taguchi method, which uses an orthogonal array to build the design experiments. The data obtained from the test results were analysed according to the Taguchi method and it was determined that the best ratios obtained are 20% and 25% for pumice, 6% and 8% for lime, and 12% and 15% for marble.

In the second stage, using the best 2 ratios of each additive, 4 different mixtures using an orthogonal array (L4) of Taguchi method were tested for swelling parameters and compaction parameters. In addition, the unconfined compressive strength was tested for 1, 7, and 28 days curing time. In light of the obtained results, the following conclusions can be drawn:

- There was a decrease in LL and PI values, which are the indicators of plasticity properties for all pumice mixtures where only pumice was used as an additive, with an increasing percentage of pumice used.
- There was a reduction in the PI & LL values when 12% marble was used alone. The values of PI & LL then suddenly surged upon increasing marble content up to 15%.
- Using lime alone decreased the PI values up to 6% and then the values insignificantly increased. On the other hand, the LL values decreased with increasing the lime content up to 5% and then increased at a lime content of 6% after which LL values returned to decrease again.
- The maximum percentage of the reduction in LL values when each additive was used alone was 21.7% at 25% pumice, 32.2% at 8% lime, and 19.8% at 12% marble content. However, the maximum percentage of the decrease in PI values

was 25.5%, 59.9%, and 24.4% at ratios of 20% pumice, 6% lime, and 12% marble, respectively.

- ❖ Using the additives together significantly decreased the PI and LL values since the optimum mixtures for reducing LL and PI values were SP20L8M15 and SP25L6M15, respectively. The SP20L8M15 mixture reduced the LL from 143.6% to 70.31% by a percentage of 51.04% while the SP25L6M15 mixture reduced the PI from 104.8% to 14.32% by a percentage of 86.34%.
- ❖ It was noticed that the maximum dry density of mixtures increased while the optimum water content values decreased. Using the SP20L8M12 mixture increased the maximum dry density from 1.23 to 1.29 g/cm^3, and reduced the optimum moisture content from 37.4% to 29.6%.
- ❖ According to the results obtained from laboratory experiments, mixing pumice, lime, and marble with high plasticity soil significantly increased their unconfined compressive strength. At the same time, it will be beneficial to use them to decrease the swelling percentage in chemical stabilization applications of expansive soils.
- ❖ The UCS increased to the maximum when the SP25L8M12 mixture was used since it enhanced the strength of untreated soil from 327.17 to 632.9 kPa for curing time of 1 day, from 367.07 to 1938.86 kPa after 7 days curing time, and from 348.1 to 4709.46 kPa for curing time of 28 days.
- ❖ The swell percentage reduced from 62.17% to 0.51% by a percentage of 99.18% when the SP25L8M12 mixture was used. In addition, the swell pressure decreased by 97.07% (from 580 kPa to 17kPa) when the SP25L6M15 mixture was utilized. However, unexpected high results of swell pressure were obtained for some mixtures which could be referred to their high strengths.
- ❖ The Taguchi method was an indicator in the analysis and prediction of the results. However, high errors were obtained in predicting the plasticity index of the mixtures which could be due to the linear analysis followed by the Taguchi method. In addition, the small values of PI and swell percentage may lead to high errors if any small amount of difference between the actual and predicted results occurred.

5. RECOMMENDATIONS

Within the scope of the thesis, it has been observed that pumice, marble, and lime can be used as alternative additives in stabilization studies for expansive soils. The fact that these additives are used in soil stabilization and have great environmental and economic benefits, makes them appropriate to be used in geotechnical engineering applications.

In order to adequately understand the effects of pumice, marble, and lime on the geotechnical properties of expansive soils, future works could consider conducting experimental studies using different clays and different types of pumice and marble. In addition, the effect of using each additive alone, dual mixtures of two additives, and triple mixtures of three additives with soil can be investigated at the same time to sufficiently understand the effect of these additives alone or together on soil stabilization.

Furthermore, following are suggestions provided for future studies:

- ❖ The Taguchi method can be applied using more levels and its applicability on different levels and under different conditions can be investigated. In addition, it is recommended to make the first level to be zero for all additives in order to be able to predict the results using an additive alone or with other additives.
- ❖ The one-dimensional swell test is one of the most acceptable techniques for measuring the swell percentage. However, for the full-scale representation of the soil performance, X-Ray diffraction and scanning electron microscope capabilities can be explored to study the mineralogical properties of treated and untreated soil.
- ❖ For the swelling pressure, it can be found using different methods and can be predicted using empirical equations.
- ❖ Economic and environmental aspects of utilizing pumice, marble, and lime can be studied to find the optimum mixture.

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RESUME

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