

Ö. Y. ÇİĞDEM

UTILIZATION OF POWDERED GLASS AND EXPANDED POLYSTYRENE
BEADS (EPS) AS ADDITIVES IN CLAYEY SOILS

THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
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ÖYKÜ YAĞMUR ÇİĞDEM

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Approval of the Graduate School of Natural and Applied Sciences, Atilim University.

Prof. Dr. Ender Keskinliç
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of **Master of Science in Civil Engineering, Atilim University.**

Prof. Dr. Hasan Umur Akay
Head of Department

This is to certify that we have read the thesis UTILIZATION OF POWDERED GLASS AND EXPANDED POLYSTYRENE BEADS (EPS) AS ADDITIVES IN CLAYEY SOILS submitted by ÖYKÜ YAĞMUR ÇİĞDEM and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Asst. Prof. Dr. Ebru Akış
Supervisor

Examining Committee Members:

Prof. Dr. Berna Unutmaz
Civil Engineering Department, Hacettepe University _____

Asst. Prof. Dr. Ebru Akış
Civil Engineering Department, Atilim University _____

Asst. Prof. Dr. Bahram Lotfisadigh
Manufacturing Engineering Department, Atilim University _____

Date: 08 June 2022

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name : iğdem, Öykü Yağmur

Signature :

ABSTRACT

UTILIZATION OF POWDERED GLASS AND EXPANDED POLYSTYRENE BEADS (EPS) AS ADDITIVES IN CLAYEY SOILS

Çiğdem, Öykü Yağmur

M.Sc., Department of Civil Engineering

Supervisor : Asst. Prof. Dr. Ebru Akış

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Waste management is gaining importance since the impact of climate change on human life becomes more apparent. In this study, it is aimed to investigate the effect of waste materials on the improvement of high plasticity clay. Glass powder and expanded polystyrene beads (EPS) are selected as waste materials since both additives have the lowest recycling rate (4.43% and 4.47%, respectively) among solid wastes. The effect of these materials is evaluated both individually and together with various experiments such as the Atterberg limit test, standard proctor test, one-dimensional swell, and unconfined compressive strength tests. Additive percentages are set at 0.3%, 0.9%, and 2% of the dry weight of the soil sample for EPS and 2%, 4%, and 6% of the dry weight of the soil for glass powder. Test results show that the addition of waste glass powder causes an increase in unconfined compressive strength and a decrease in free swell values. When EPS is used as the only additive, a decrease in the unconfined compressive strength and free swell values is observed. Furthermore, the combination of 4% of glass powder and 0.9% of EPS addition causes the most effective improvement in both unconfined compressive strength and free swell values.

In addition to the experimental studies, the obtained data from this study, and the test results of related studies in the literature are used together to build up datasets. These datasets are used to perform multilinear regression (MLR) and artificial neural network (ANN) analyses. As a result of MLR analyses, prediction equations are proposed to

estimate the free swell or unconfined compressive strength of the improved soils depending on the additive contents, free swell, or unconfined compressive strength of untreated soils. When the same dataset is used for ANN analysis, similar results with the MLR analyses are observed. Furthermore, if the provided data is limited to the Atterberg limits only, the ANN method provides reliable predictions of those parameters. A 5-fold cross variation is performed to verify that the results of ANN analysis do not depend on the selection of the data sets. Both MLR and ANN methods show that it is possible to predict the free swell and unconfined compressive strength of treated soil samples with high correlation coefficients.

Keywords: Soil improvement, high plasticity clay, waste material, multilinear regression analysis, artificial neural network.

ÖZ

KİLLİ ZEMİNLERDE KATKI MADDESİ OLARAK CAM TOZU VE GENLEŞTİRİLMİŞ POLİSTREN (EPS) KULLANILMASI

Çiğdem, Öykü Yağmur

Yüksek Lisans, İnşaat Mühendisliği Bölümü

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İklim değişikliğinin insan yaşamı üzerindeki etkisinin daha belirgin hale gelmesiyle atık yönetimi önem kazanmaktadır. Bu çalışmada, atık malzemelerin yüksek plastisiteli kil zemin iyileştirmesi üzerindeki etkisinin araştırılması amaçlanmıştır. Atık malzeme olarak, katı atıklar arasında en düşük dönüşüm oranına sahip olan cam tozu (%4.43) ve genişletilmiş polistiren (EPS) (%4.47) seçilmiştir. Cam tozu ve EPS, tek tek ve birlikte kullanılarak zemin parametreleri üzerindeki etkisi Atterberg limit, standart proktor, şişme yüzdesi tayini ve serbest basınç testleri yürütülerek değerlendirilmiştir. Katkı yüzdeleri, EPS için kuru numune ağırlığının %0.3, %0.9 ve %2'si olarak seçilirken, cam tozu için kuru numune ağırlığının %2, %4 ve %6'sı olarak belirlenmiştir. Test sonuçları, katkı maddesi olarak sadece cam tozu kullanıldığında malzemenin serbest basınç dayanımında artışa ve şişme yüzdelerinde azalışa neden olduğunu göstermiştir. Ancak, sadece EPS kullanıldığında hem şişme yüzdeleri hem de serbest basınç dayanımı değerlerinde azalma görülmüştür. Her iki katkı malzemesinin %4 cam tozu ve %0.9 EPS olarak belirlenmesi durumunda ise dayanım ve şişme yüzdesi en etkili iyileştirme ile sonuçlanmıştır.

Deneysel çalışmaya ek olarak, bu çalışmadan elde edilen veriler ve literatürdeki benzer çalışmaların sonuçları ile veri dosyaları oluşturulmuştur. Söz konusu veriler kullanılarak regresyon analizi ve Yapay Sinir Ağları (YSA) analizleri yürütülmüştür. Regresyon analizi sonucunda, iyileştirilmiş zeminlerin şişme yüzdesi veya serbest

basınç dayanımlarını, iyileştirme öncesi şişme yüzdesi veya serbest basınç dayanımı ile katkı yüzdelere bağlı olarak tahmin eden denklemler önerilmiştir. Bununla birlikte, regresyon analizlerinde kullanılan girdiler ile YSA analizleri yapılmış olup benzer sonuçlar elde edilmiştir. Ayrıca, verilerin sadece Atterberg limitleri ile kısıtlı olması durumunda YSA metodu ile söz konusu parametrelerin güvenli bir şekilde tahmin edilebileceği gösterilmiştir. YSA analiz sonuçlarının test verilerinin seçimine bağlı olmadığını göstermek için 5 katlı çapraz geçerlilik teorisi kullanmıştır. Hem regresyon analizi hem de YSA analiz sonuçları, yüksek korelasyon katsayısı değerleri ile katkı malzemesi ile iyileştirilmiş zemin numunelerinin serbest şişme ve serbest basınç dayanımını tahmin etmenin mümkün olduğunu göstermiştir.

Anahtar Kelimeler: Zemin iyileştirme, yüksek plastisiteli kil, atık malzeme, regresyon analizi, yapay sinir ağı.



To my grandfathers...

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CHAPTER 1

INTRODUCTION

The main characteristics of high plasticity clay soil samples may not be always sufficient to support structures or to use as construction materials. If the properties of the problematic soil samples are not improved, there can be deformation problems and even failures. At this point, the solution may be the improvement of the engineering properties of the soil and the purpose of the soil improvement can be listed as below [1]:

- Reducing the settlement of the structures
- Increasing the bearing capacity of the soil
- Decreasing the voids inside the soil
- Increasing the factor of safety against slope failures
- Controlling shrinking and swelling

There are several ways to stabilize soils. According to Hausmann [1], four soil improvement methods are valid: mechanical modification, hydraulic modification, modification by inclusion and confinement, and physical and chemical modification. These stabilization methods must be selected according to the type of soil, and the project since all these methods are focused on different problems.

Mechanical modification aims to increase the density of the soil by implementing short terms external forces such as impact rollers, plate vibrators, and deep compaction by heavy tamping. On the other hand, hydraulic modification uses for lowering the groundwater level of soils. For fine-grained soils, this method requires external force and electrical loads application in the long term, while it can be applied to coarse-grained soils by using boreholes or trenches directly. As mentioned, one of the soil improvement methods is a modification by inclusion and confinement. The objective of this method is to provide tensile strength to soil mass by using reinforcement fibers, strips, and fabrics. Lastly, physical, and chemical modification focuses on improving the engineering properties of the soil by using additives. Waste materials can already

be used as additives [1]. Using waste materials for soil improvement is highly recommended since it is an environmentally friendly solution. Besides the ecological advantages, waste materials provide saving costs. There is an extensive range of waste materials that can be used for soil improvement. In this study, high plasticity clay, bentonite, is treated with waste materials. Waste glass powder (WGP) and expanded polystyrene beads (EPS) are used as additives.

1.1.Objective and Scope of the Thesis

The objective of the thesis is to investigate the effect of waste glass powder and EPS beads separately and together on the free swell and unconfined compressive strength values of the treated high plasticity soil sample, bentonite. In addition, the free swell and unconfined compressive strength of treated clayey soils are predicted by establishing empirical relationships by multiple linear regression analyses and using previously developed ANN modeling.

The scope of this thesis is limited by the tests performed on bentonite soil samples obtained from Eczacıbaşı Esan Industrial Raw Material Company at Bozüyük, Bilecik.

1.2.Thesis Description

The study consists of seven chapters and the contents of the chapters are as follows:

- Chapter 1: Introduction. The main objective of the study is explained, and the organization of the thesis is presented.
- Chapter 2: Literature Review. This chapter is a summary of the previous studies on soil improvement with additives.
- Chapter 3: Materials. This chapter presents the basic properties of the soil and the additives used in this study.
- Chapter 4: Laboratory Tests. In this chapter of the thesis, the experiments carried out in this study are explained according to the reference standards.

- Chapter 5: Experimental Studies, Test Results, and Discussions. The chapter presents the sample preparation and obtained test results including the discussions of the findings.
- Chapter 6: Statistical Analyses and ANN Model. In this chapter, the results of multilinear regression analyses and artificial neural network analyses are presented.
- Chapter 7: Conclusions and Recommendations. The last chapter of the thesis contains a summary of the findings of this study, as well as recommendations for future research.



CHAPTER 2

LITERATURE REVIEW

In this chapter of the thesis, a literature review, mostly about soil stabilization by using Expanded Polystyrene beads (EPS) and glass powder, is presented. There are numerous studies about the improvement of the engineering properties of clays by using waste materials as additives. Relevant studies are discussed as follows.

2.1. Soil Stabilization by Using Various Materials

Waste material usage is an economical solution for soil improvement. There are a lot of studies about the effect of various waste materials on the engineering properties of soils.

Çanakçı et al. [2] used waste beverage cans for low plastic clay (CL) stabilization. Cans were cut into 5 mm thicknesses, then mixed with the soil in different percentages (2%, 4%, 6%, 8%, and 10% of the dry weight of the soil sample). As a result of the conducted tests, it was concluded that there was a decrease in the optimum moisture content with the additional beverage can strips. Additionally, the effect of additive on the California bearing ratio (CBR) was examined as the optimum percentage was reported as 8%.

Özdemir [3] used fly ash and investigated the bearing capacity improvement of a soft soil which was classified as CL according to USCS. The fly ash was (Class C) obtained from Soma Thermal Power Plant and used as the additive. Fly ash contents were selected as 3%, 5%, 7%, and 10% of the dry weight of soil. An experimental study was conducted by performing a modified proctor test, CBR, and unconfined compressive strength test. In addition to a scanning electron microscope energy dispersive X-ray analysis was conducted to obtain the microstructures of the observed soil samples. The results showed that plasticity index values of the observed samples

were nearly the same with and without any additives. Secondly, with the additional fly ash content, a decrease in the maximum dry density was seen, but the change in the optimum moisture content was not consistent. Lastly, it was reported that, after 7 and 28 days of curing, a decrease in the free swell, and an increase in the CBR were observed.

Daryati and Ramadhan [4] performed a study about the improvement of expansive soils (CL), stabilized with rice husk ash (RHA). RHA was mixed with the soil by 3%, 6%, and 9% by dry weight of the sample. The performed tests were listed as specific gravity test, standard proctor compaction test, direct shear, and CBR. Compaction test results showed that there was an increase in both optimum moisture content and maximum dry density values with the addition of RHA content. Moreover, as a result of CBR tests, it was observed that the CBR value was increased by 130% with the addition of 6% RHA. After that point, when the percentage of the RHA increased, CBR values showed a decrease. The study concluded that with the addition of RHA content there was an increase in cohesion, CBR value, plasticity index, optimum moisture content, and maximum dry density, while there was a decrease in internal friction angle, and specific gravity.

Kılıç et al. [5] conducted a study about the effect of lime and gypsum on the engineering properties of the soil. The importance of this study was investigating the effect of the lime, and gypsum both separately and together. The used soil sample was classified as clay with high plasticity. The additives were mixed with the soil in 3%, 6%, 9%, 12%, and 15% by the dry weight of the soil. The swelling and unconfined compression tests were performed after 90 days of preparation of the soil samples. As a result of the conducted study, the effect of the lime alone showed the maximum decrease in both swell pressure and swelling percent. Also, the increase in the unconfined compressive strength was the maximum when only lime was used as an additive.

Different type of additives was used to improve the physical and mechanical properties of clays. For example, in related studies, the effect of EPS and sand [6], copper slag and lime [7], lime and artificial pozzolan [8], polypropylene and lime [9], lime, calcium chloride, and rice husk ash [10], rice husk ash and fly ash [11], cement and EPS [12] on these parameters are investigated.

2.2. Soil Stabilization by Using Expanded Polystyrene (EPS)

In this section, the previous studies about the treatment of clayey soils by using expanded polystyrene (EPS) are presented.

Shirazi et al. [13] performed unconfined compressive strength (UCS) tests on the clayey soil (CL) samples, that were mixed with different contents of EPS beads (0.02%, and 0.03% of the dry weight of the soil sample). In the study, not only the effect of the EPS beads on the engineering properties of the soil was investigated, but also, the effect of the EPS sizes was examined by using coarse ($\phi > 4.75$ mm) and fine ($\phi < 4.75$ mm) EPS beads. UCS tests were conducted to observe the effect of EPS beads. Change in the compressive strength values of clay samples according to EPS percentages are given in Table 2.1 as follows.

Table 2.1 Properties of the Untreated Clay and Effect of the EPS Beads [13]

| | | | |
|----------------------------------|-------|------|------|
| Optimum Water Content (%) | 11.80 | | |
| Maximum dry density (g/cm^3) | 1.71 | | |
| Liquid Limit (%) | 30.00 | | |
| Plastic Limit (%) | 15.00 | | |
| Plasticity Index (%) | 15.00 | | |
| Coarse EPS Percentages (%) | 0.0 | 0.02 | 0.03 |
| Compressive Strength (kPa)* | 140 | 78 | 75 |
| Fine EPS Percentages (%) | 0.0 | 0.02 | 0.03 |
| Compressive Strength (kPa)* | 140 | 76 | 70 |

* These values are obtained from the related figures in the reference study.

As it is seen in Table 2.1, with the additional coarse and fine EPS percentages, there was a decrease in the compressive strength (q_u) of the clayey soil. When coarse EPS beads were used, the compressive strength value of the sample dropped by 46% with the additional 0.03% coarse EPS. On the other hand, a reduction in compressive strength was observed by %50 when 0.03% fine EPS beads were used. The authors stated that the adhesion between clay particles and EPS beads was smaller than the cohesion between clay particles, resulting in a reduction in UCS. In addition, since

fine EPS beads had more contact with the soil than coarse EPS beads, the reduction in the unconfined compressive strength values was higher when fine EPS beads were used.

Illuri [14] focused on the sustainability of expansive clay and recycled EPS mixture as a backfill material. For that manner, an artificial soil was reconstituted by mixing fine sand and sodium bentonite. Bentonite contents were selected as 16%, 24%, and 32%. Thus, three different artificial clays are obtained and named SB16, SB24, and S32 according to their bentonite percentages. Change in the free swell was observed with and without EPS beads. The result of the test and the basic properties of expansive clay are presented in Table 2.2.

Table 2.2 Properties of the Untreated Soil and Effect of the EPS Beads [14]

| SB16 | | | | |
|----------------------------------|-------|-----|-----|-----|
| Bentonite Content (%) | 16 | | | |
| Optimum Water Content (%) | 14.00 | | | |
| Maximum Dry Density (g/cm^3) | 1.73 | | | |
| Liquid Limit (%) | 43.00 | | | |
| Plastic Limit (%) | 21.00 | | | |
| Plasticity Index (%) | 22.00 | | | |
| EPS Content (%) | 0.0 | 0.3 | 0.6 | 0.9 |
| Free Swell (%) * | 29 | 23 | 19 | 12 |
| SB24 | | | | |
| Bentonite Content (%) | 24 | | | |
| Optimum Water Content (%) | 13.00 | | | |
| Maximum Dry Density (g/cm^3) | 1.74 | | | |
| Liquid Limit (%) | 60.00 | | | |
| Plastic Limit (%) | 22.00 | | | |
| Plasticity Index (%) | 38.00 | | | |
| EPS Content (%) | 0.0 | 0.3 | 0.6 | 0.9 |
| Free Swell (%) * | 47 | 43 | 38 | 27 |
| SB32 | | | | |
| Bentonite Content (%) | 32 | | | |
| Optimum Water Content (%) | 12.50 | | | |

Table 2.2 (continued)

| SB32 | | | | |
|----------------------------------|-------|-----|-----|-----|
| Maximum dry density (g/cm^3) | 1.71 | | | |
| Liquid Limit (%) | 77.00 | | | |
| Plastic Limit (%) | 24.00 | | | |
| Plasticity Index (%) | 53.00 | | | |
| EPS Content (%) | 0.0 | 0.3 | 0.6 | 0.9 |
| Free Swell (%) * | 72 | 63 | 58 | 43 |

* These values are obtained from the related figures in the reference study.

The mixtures were prepared at the optimum moisture content of the expansive soil. For the standard proctor test, the test was performed immediately after the soil is mixed with the additives and water in order to hinder any segregation and maintain the consistency of the soil sample. However, for determining the volumetric shrinkage, the prepared samples waited about 4 hours at room temperature and were oven-dried for 48 hours at 70°C. As concluded from the volumetric shrinkage test result, the recycled EPS beads reduced the shrinkage potential by different percentages according to the EPS contents and the type of the prepared soil samples. The reduction in the free swell was maximum when EPS content was selected as 0.9% for all three specimens. The maximum reductions in the free swell values can be noted as 59%, 43%, and 40% for SB16, SB24, and SB32, respectively.

In the same study [14], the effects of EPS beads and lime on the engineering properties of a dredged soil were also investigated. It was found that the addition of 2.0% EPS at a moisture content of 45.0% gave the most effective compaction. While the EPS percentage was kept constant at 2.0% of the dry weight of the soil sample, the lime percentages were selected as 3.0%, 5.0%, and 7.0% of the dry weight of the soil sample. The results of the tests carried out are presented in tabular form (Table 2.3).

Table 2.3 Effect of the EPS Beads on the Observed Engineering Properties [14]

| Maximum Dry Unit Weight (kN/m ³) | | 12.85 | | | |
|--|---------------------|------------------|----------------|---------------------------------------|------------------|
| Optimum Moisture Content (%) | | 39.00 | | | |
| Liquid Limit (%) | | 119.00 | | | |
| Plastic Limit (%) | | 38.00 | | | |
| Plasticity Index (%) | | 81.00 | | | |
| Lime Percentages (%) | EPS Percentages (%) | Unsoaked CBR (%) | Soaked CBR (%) | Unconfined Compressive Strength (kPa) | CBR Swelling (%) |
| 0.0 | 0.0 | 1.00 | 1.50 | 58.30 | 0.00 |
| 0.0 | 2.0 | 1.50 | 1.50 | - | - |
| 3.0 | 2.0 | 6.00 | 1.50 | 247.00 | 3.00 |
| 5.0 | 2.0 | 10.00 | 1.50 | 383.00 | 3.00 |
| 7.0 | 2.0 | 8.00 | 1.00 | 295.00 | 3.00 |
| 3.0 | 0.0 | - | - | 1710.00 | 0.00 |
| 5.0 | 0.0 | - | - | 1754.00 | 0.00 |
| 7.0 | 0.0 | - | - | 1197.00 | 1.00 |

The author claimed that EPS addition alone was not sufficient. Therefore, the addition of lime may be considered necessary. While EPS addition of only 2.0% resulted in a 0.5% increase in unsoaked CBR value and did not cause any change in the soaked CBR, the change in unsoaked CBR value peaked at the top when there was a lime addition of 5.0%. On the other hand, up to a lime addition of 7.0%, there was no change in the soaked CBR values. At 7.0% lime and 2.0% EPS addition, both unsoaked and soaked CBR values decreased. Although lime was added to improve the soil, it did not cause any change in soaked CBR values. The author explained that this could be due to the sulfate content in soils. In the unconfined compressive strength test results, the effect of the additives peaked again at 5.0% lime and 2.0% EPS addition. At these two specified percentages, the unconfined compressive strength value was almost four times higher than the unconfined compressive strength value of the untreated soil. Similar to the values for the unsoaked and soaked CBR values, a decrease in unconfined compressive strength was seen at 7.0% lime and 2.0% EPS addition. The effect of EPS on unconfined compressive strength values was not observed in this

particular study. For the change in swelling percentages, the effect of EPS alone was not investigated. In addition, the author finds that the increase in swelling percentage was related to the change in lime content rather than EPS. Since the materials with soluble sulfates were treated with calcium-based stabilizers, this had a negative effect on potential swelling due to the chemical reaction between the minerals.

Rocco [15] conducted a study about the effect of EPS beads on the engineering properties of Expanded Polystyrene (EPS) and cohesive mixtures. In this study, kaolin was used as the soil sample and classified as CH/MH according to USCS. EPS percentages were taken as 0.5, 1.0, and 1.5 by the dry weight of the soil sample. The results of the author's study are summarized in Table 2.4.

Table 2.4 Effect of the EPS Beads on the Observed Engineering Properties [15]

| | | | | |
|--|-------|-------|-------|-------|
| Liquid Limit (%) | 56.00 | | | |
| Plastic Limit (%) | 27.00 | | | |
| Plasticity Index (%) | 29.00 | | | |
| Clay Fraction (%) | 85.00 | | | |
| EPS Content (%) | 0.0 | 0.5 | 1.0 | 1.5 |
| Optimum Water Content (%) | 27.50 | 27.10 | 26.50 | 26.80 |
| Maximum Dry Density (g/cm^3) | 14.20 | 12.70 | 11.80 | 10.40 |
| Free Swell (%) | 7.7 | 6.5 | 6.5 | 3.2 |
| Swell Pressure (kPa) | 36.40 | 32.60 | 38.30 | 28.70 |
| Cohesion (kPa) | 25.50 | 16.50 | 8.30 | 16.50 |
| Internal Friction Angle ($^{\circ}$) | 8.80 | 9.40 | 10.1 | 9.60 |
| Effective Cohesion (kPa) | 35.90 | 26.20 | 6.20 | 24.80 |
| Effective Internal Friction Angle ($^{\circ}$) | 15.20 | 15.50 | 19.00 | 15.90 |

Before the compaction test, the soil samples were kept for at least 24 hours in air-tight containers. With the increase in the EPS contents, both maximum dry density values and optimum moisture content values were decreased (Table 2.4). For the swelling tests, it was stated that all the tests were performed at the optimum moisture content of the kaolin and kaolin-EPS mixtures. As shown in Table 2.4, when EPS was added up to 1.0% of the dry weight of the soil sample, there was a slight change in the free swell and swell pressure. On the other hand, at the EPS addition of 1.5%, the change

in the free swell and swell pressure was significant. The decrease in the free swell value when the EPS content was 1.5 of the dry weight of the soil sample was about 60% compared to the untreated soil sample. Also, the decrease in swell pressure with EPS addition of 1.5% was 20% compared to the untreated soil. Lastly, as a result of CU triaxial strength tests, internal friction angle values and cohesion values were obtained and presented in Table 2.4. There was no exact relation between the EPS percentages and changes in both internal friction angle and cohesion values. On the other hand, the author explained the rapid decrease of the cohesion and internal friction angle values, also for the effective parameters, of the soil with a 6% EPS mixture due to the shearing rate that is affected by the loss of panel pressure during consolidation stage.

In 2019, another study on soil improvement was conducted by Soundara and Selvakumar [16] by using geobeads. The swelling properties of the treated and untreated soil samples were studied. An expansive soil was chosen as the soil sample due to its higher swell and shrinkage potential. The observed soil sample was classified as high plasticity clay (CH) according to USCS classification. The used EPS beads in the study were obtained from EPS waste blocks. The blocks are crushed into EPS beads by hand and waste EPS beads with a diameter of 2 to 6 mm were obtained. The optimum moisture content of both treated and untreated samples was determined by a standard proctor test and swelling compression tests were performed. The results obtained are shown in Table 2.5 in terms of the percentage of EPS beads.

Table 2.5 Effect of the EPS Beads on the Observed Engineering Properties [16]

| | | | | | |
|----------------------------------|--------|--------|--------|--------|--------|
| Liquid Limit (%) | 276.00 | | | | |
| Plastic Limit (%) | 33.00 | | | | |
| Plasticity Index (%) | 243.00 | | | | |
| EPS Content (%) | 0.0 | 0.25 | 0.5 | 0.75 | 1.0 |
| Optimum Water Content (%) | 37.33 | 37.15 | 36.98 | 36.94 | 36.92 |
| Maximum Dry Density (g/cm^3) | 1.38 | 1.32 | 1.27 | 1.25 | 1.21 |
| Free Swell (%) | 14.7 | 13.2 | 11.9 | 9.4 | 8.1 |
| Swell Pressure (kPa) | 530.00 | 452.00 | 328.00 | 273.00 | 210.00 |

As shown in Table 2.5, EPS beads led to a significant reduction in swelling potential. The reduction in the free swell and swell pressure was expressed in terms of the high compressibility properties of the EPS waste beads, similar to the related studies in the literature. The maximum reduction in free swell was observed when the percentage of EPS beads was highest. When the EPS beads represented 1% of the dry weight of the soil sample, a 45% reduction in the free swell was observed, while this reduction was only 10% when the percentage of EPS beads was 0.25%. As for the swell pressure, it decreased to half of its value when the EPS bead addition was increased from 0.25% to 1.0%. The author stated that a 1% EPS addition was sufficient to reduce both the free swell and the swell pressure of expansive soil.

2.3. Soil Stabilization by Using Glass Powder

Previous studies about the improvement of low plasticity clayey soils and high plasticity clayey soils with glass powder are presented in this section of the thesis.

Çanakçı et al. [17] and Al-Kaki [18] investigated the stabilization of low plasticity clay (CL) with waste soda-lime glass powder. For the mixtures, the soil was mixed with glass powder with 3, 6, 9, and 12% of the dry weight of the soil. The waste soda glasses were obtained by crushing and sieving through the #200 sieve. Conducted tests could be listed as standard proctor test, Atterberg limit test, California bearing ratio test, and unconfined compressive strength tests, and the test results are presented in Table 2.6

Table 2.6 Effect of the Glass Powder on the Observed Engineering Properties [17, 18]

| Glass Percentages (%) | 0.0 | 3.0 | 6.0 | 9.0 | 12.0 |
|----------------------------------|-------|-------|-------|-------|-------|
| Optimum Water Content (%) | 15.25 | 14.50 | 14.20 | 13.10 | 12.00 |
| Maximum Dry Density (g/cm^3) | 1.82 | 1.86 | 1.88 | 1.91 | 1.92 |
| Liquid Limit (%) | 46.50 | 35.50 | 31.00 | 29.90 | 27.90 |
| Plastic Limit (%) | 28.68 | 22.41 | 18.78 | 18.16 | 17.93 |
| Plasticity Index (%) | 17.82 | 13.09 | 12.22 | 11.74 | 9.97 |
| California Bearing Ratio (%) | 2.34 | 3.29 | 4.23 | 5.18 | 6.12 |
| Free Swell (%) | 5.5 | 4.4 | 3.35 | 2.33 | 1.65 |

Table 2.6 (continued)

| Glass Percentages (%) | 0.0 | 3.0 | 6.0 | 9.0 | 12.0 |
|--|--------|--------|--------|--------|--------|
| Unconfined Compressive Strength for 3 days of curing (kPa) | 220.00 | 350.00 | 510.00 | 400.00 | 200.00 |
| Unconfined Compressive Strength for 7 days of curing (kPa) | 240.0 | 419.6 | 582.5 | 500.3 | 299.8 |

As a result of the compaction tests, it was reported that with the glass powder percentage increase, maximum dry density values were increased while optimum moisture content values were decreased. The maximum increase in the maximum dry density (5.5%) was observed when the glass powder percentage was 12%. In addition to that optimum moisture, the content value decreased with the increase in the glass powder content. The maximum reduction in the optimum moisture content value (21.6%) was observed when the glass powder addition was 12%. The authors stated that these changes may be explained by the lower water absorption capacity of the glass powder. Additionally, the plasticity index value was decreased by 44% with the 12% of glass powder addition. On the other hand, CBR values were increased with the increasing glass powder addition. The maximum increase in the CBR value (140%) was observed when the glass powder content was 12% which was expressed as the pozzolanic-cementitious by the authors. As a result of the unconfined compressive strength test, two different curing times were selected as 3 and 7 days. Compared with the untreated soil, for all cases, an increase in the unconfined compression strength got its maximum value when the glass powder content was 6%. After that point, unconfined compressive strength values decreased. However, despite this decrease, overall unconfined compressive strength values increased for seven days of curing times.

Fauzi et al. [19] revealed a study that focused on improvement in the engineering properties of the soil by using cut waste plastic and crushed waste glass to reduce the pavement failures. Two different soil samples were used in the study and classified as clayey silt and silty clay. The additives were mixed with the soil at different percentages (4%, 8%, 12%) by dry weight of soil samples. The sieve analysis, Atterberg limit tests, standard compaction tests, California bearing ratio tests, and

triaxial tests were performed to evaluate the engineering properties of the soil with and without any additives. The test results are presented in a tabular form (Table 2.7).

Table 2.7 Effect of the Glass Powder on the Observed Engineering Properties [19]

| Clayey Silt (A7-6) | | | | |
|--|-------|-------|-------|-------|
| Glass Content (%) | 0.0 | 4.0 | 8.0 | 12.0 |
| Optimum Water Content (%) | 12.00 | 12.00 | 11.80 | 11.60 |
| Maximum Dry Density (g/cm^3) | 1.75 | 1.78 | 1.80 | 1.83 |
| Liquid Limit (%) | 34.00 | 34.00 | 33.90 | 32.00 |
| Plastic Limit (%) | 24.00 | 24.00 | 25.90 | 24.50 |
| Plasticity Index (%) | 10.00 | 10.00 | 8.00 | 7.50 |
| California Bearing Ratio (%) | 2.00 | 3.00 | 4.10 | 5.80 |
| Cohesion (kPa) | 38.00 | 30.00 | 13.00 | 10.00 |
| Internal Friction Angle ($^{\circ}$) | 12.00 | 13.00 | 15.00 | 24.00 |
| Clayey Silt (A6) | | | | |
| Glass Content (%) | 0.0 | 4.0 | 8.0 | 12.0 |
| Optimum Water Content (%) | 15.60 | 15.40 | 15.00 | 14.80 |
| Maximum Dry Density (g/cm^3) | 1.62 | 1.61 | 1.63 | 1.65 |
| Liquid Limit (%) | 34.00 | 32.00 | 30.00 | 29.00 |
| Plastic Limit (%) | 16.00 | 15.50 | 15.00 | 14.20 |
| Plasticity Index (%) | 18.00 | 16.50 | 15.00 | 14.80 |
| California Bearing Ratio (%) | 2.20 | 3.80 | 5.00 | 7.80 |
| Cohesion (kPa) | 19.00 | 11.00 | 8.00 | 6.00 |
| Internal Friction Angle ($^{\circ}$) | 9.00 | 15.00 | 19.00 | 24.00 |

For the compaction properties of the soil samples, increasing the glass powder percentages reduced, optimum water content percentages while increasing maximum dry density values in both cases. As a result of the Atterberg limit test results, for both soil samples, plasticity index values were decreased when the glass powder percentages were increased. Different than plasticity index values, with the increasing glass powder percentages, CBR values raised significantly. As a result of the triaxial tests, it was reported that the maximum increase in the internal friction angle was observed when the glass powder percentage was 12%. However, with the increase in the additive, cohesion values decreased.

In 2020, a study on the use of glass powder as an additive in clayey soils was conducted by Bilgen [20]. In this study, used waste glasses were obtained from the windows of collapsed buildings, and the obtained materials were sieved through a 425-micron sieve before being used as an additive. The percentage of powdered glass was set at 10%, 20%, and 25% of the dry weight of the soil. The effect of the powdered glass was observed on three different soil types: Bentonite clay, Alapli clay, and Ereğli clay. According to the Unified Soil Classification System (USCS), bentonite clay was classified as clay with high plasticity (CH) whereas Alaplı and Ereğli clay were classified as CL. Atterberg limit test, compaction tests, unconfined compressive strength, and California bearing ratio test were performed. The obtained results are presented in Table 2.8.

Table 2.8 Results of the Performed Tests for Bentonite Clay [20]

| Bentonite Clay | | | | |
|---|--------|--------|--------|--------|
| Glass Percentages (%) | 0.0 | 10.0 | 20.0 | 25.0 |
| Optimum Moisture Content (%) | 35.00 | 30.00 | 29.00 | 24.00 |
| Maximum Dry Density (kN/m ³) | 12.20 | 13.10 | 13.60 | 14.10 |
| Liquid Limit (%) | 215.00 | 210.00 | 198.00 | 196.00 |
| Plastic Limit (%) | 41.00 | 42.00 | 42.00 | 43.00 |
| Plasticity Index (%) | 174.00 | 168.00 | 156.00 | 153.00 |
| Unconfined Compressive Strength (kPa) after 0 days | 147.00 | 162.00 | 247.00 | 291.00 |
| Unconfined Compressive Strength (kPa) after 7 days | 164.00 | 512.00 | 556.00 | 731.00 |
| Unconfined Compressive Strength (kPa) after 28 days | 178.00 | 676.00 | 772.00 | 795.00 |
| California Bearing Ratio (%) after 0 day | 5.00 | 6.00 | 7.00 | 8.00 |
| California Bearing Ratio (%) after 28 days | 7.00 | 28.00 | 32.00 | 35.00 |
| Alapli Clay | | | | |
| Glass Percentages (%) | 0.0 | 10.0 | 20.0 | 25.0 |
| Optimum Moisture Content (%) | 16.00 | 14.00 | 14.00 | 14.00 |
| Maximum Dry Density (kN/m ³) | 16.50 | 17.10 | 17.10 | 17.20 |
| Liquid Limit (%) | 35.00 | 34.00 | 32.00 | 30.00 |
| Plastic Limit (%) | 18.00 | 18.00 | 20.00 | 20.00 |

Table 2.8 (continued)

| Alapli Clay | | | | |
|---|--------|--------|---------|---------|
| Glass Percentages (%) | 0.0 | 10.0 | 20.0 | 25.0 |
| Plasticity Index (%) | 17.00 | 16.00 | 12.00 | 10.00 |
| Unconfined Compressive Strength (kPa) after 7 days | 42.00 | 66.00 | 80.00 | 105.00 |
| Unconfined Compressive Strength (kPa) after 28 days | 63.00 | 70.00 | 115.00 | 128.00 |
| California Bearing Ratio (%) after 0 day | 9.00 | 13.00 | 14.00 | 24.00 |
| California Bearing Ratio (%) after 28 days | 63.00 | 70.00 | 115.00 | 128.00 |
| Eregli Clay | | | | |
| Glass Percentages (%) | 0.0 | 10.0 | 20.0 | 25.0 |
| Optimum Moisture Content (%) | 20.00 | 16.00 | 15.00 | 13.00 |
| Maximum Dry Density (kN/m ³) | 16.10 | 17.10 | 17.20 | 17.30 |
| Liquid Limit (%) | 42.00 | 40.00 | 38.00 | 37.00 |
| Plastic Limit (%) | 18.00 | 21.00 | 21.00 | 23.00 |
| Plasticity Index (%) | 24.00 | 19.00 | 17.00 | 14.00 |
| Unconfined Compressive Strength (kPa) after 0 days | 410.00 | 461.00 | 503.00 | 560.00 |
| Unconfined Compressive Strength (kPa) after 7 days | 481.00 | 843.00 | 907.00 | 1016.00 |
| Unconfined Compressive Strength (kPa) after 28 days | 515.00 | 907.00 | 1328.00 | 1563.00 |
| California Bearing Ratio (%) after 0 day | 11.00 | 16.00 | 15.00 | 13.00 |
| California Bearing Ratio (%) after 28 days | 18.00 | 33.00 | 59.00 | 62.00 |

As can be seen in Table 2.8, with the increase of glass powder addition, there was a decrease in the optimum moisture content values, while the maximum dry density values increased slightly for all three observed soil types. Table 2.8 also showed the change in liquid limit, plastic limit, and plasticity index. The liquid limit values decreased by almost 9.0% when the glass powder addition was 25.0% for bentonite while these reductions were approximately 14.0%, and 12.0% for Alapli and Eregli clay, respectively. Similar to the values for liquid limit, the values for plasticity index also decreased with an increase in glass powder addition for bentonite, Alapli clay, and Eregli clay. When the glass powder addition was 25.0% of the dry weight of the soil sample, the maximum decrease in the plasticity index could be found to be 12.0%

of the plasticity index of the untreated soil for bentonite, 41.1% for Alapli clay, and 41.7% for Eregli clay. However, as shown in Table 2.8, the plastic limit values for untreated and treated soil samples were almost constant for all the soil types. The overall consideration of the effect of glass powder on the Atterberg limit index values of the soil sample did not result in any change in the classification of the soil sample according to the Unified Soil Classification System. In evaluating the effects of glass powder on the strength properties of the soil sample, with 25.0% glass powder addition, the value of the California bearing ratio increased by 60% after 0 days. The increase in CBR values for 0 days of curing is even higher for Eregli and Alapli clays which can be noted as 166% and 191%, respectively. Similarly, for the unconfined compressive strength values, the value almost doubled with the additional 25.0% glass powder after 0 days for bentonite, while the increase in the unconfined compressive strength values of Alapli clay can be noted as approximately 107% of its initial value, and 37% of Eregli clay. After 28 days, the maximum increase in the value of unconfined compressive strength was observed when the glass powder addition was the maximum (25%) for three of the soil types. Additionally, as seen in Table 2.8, it was found that both the California bearing ratio values, and unconfined compressive strength values increased with aging and glass powder addition. Also, Bilgen [20] stated that all the specimens for these two tests were prepared at the optimum moisture content of the respective specimen and the author concluded that glass powder had potential use for stabilizing clayey soils.

Ibrahim et al. [21] investigated the effect of glass powder on the soil samples collected from the Erbil city of Iraq which were classified as high plasticity clays (CH) according to Unified Soil Classification Systems. The glass powder used in this study was crushed and sieved through a #200 sieve. Atterberg limit test, standard proctor test, free swelling, and unconfined compressive strength test were performed. The results obtained in this study are presented in Table 2.9.

Table 2.9 Results of the Conducted Tests with respect to Glass Powder Percentages [21]

| Glass Percentages (%) | 0.0 | 6.0 | 12.0 | 18.0 | 27.0 | 36.0 |
|--|-------|-------|-------|-------|-------|-------|
| Liquid Limit (%) * | 51.00 | 49.00 | 47.50 | 42.50 | 40.00 | 39.00 |
| Plastic Limit (%) * | 22.60 | 22.50 | 22.20 | 21.00 | 20.00 | 19.50 |
| Plasticity Index (%) * | 28.40 | 26.50 | 25.30 | 21.50 | 20.00 | 19.50 |
| Optimum Moisture Content (%) | 20.50 | 19.15 | 18.45 | 18.00 | 17.60 | 17.50 |
| Maximum Dry Density (g/cm ³) | 1.65 | 1.67 | 1.69 | 1.71 | 1.73 | 1.74 |
| Unconfined Compressive strength (kPa) | 282.3 | 345.3 | 361.9 | 410.1 | 560.1 | 517.6 |
| Free Swell (%) * | 7.5 | 5.8 | 5.5 | 3.6 | 3.6 | 3.6 |

* These values are obtained from the related figures in the reference study.

As a result of the standard proctor test, the relation between the optimum moisture content, the maximum dry density, and the glass powder addition was obtained. With the increase in the glass powder addition, the values of optimum moisture content decreased while the values of maximum dry density increased as shown in Table 2.9. The maximum decrease in optimum moisture content (14.6%) was observed when glass powder addition was highest (36%). Also, the maximum increase in maximum dry density (5.4%) was obtained when the glass powder addition was 36% of the dry weight of the soil sample. Since the surface area of the silica, which was highly involved in the glass powder used, was smaller than the surface area of the clay, the optimum moisture content values decreased with the additional glass powder [21]. As a result of the unconfined compressive strength test, the value of unconfined compressive strength peaked and almost doubled at 27% glass powder addition. Further increase in glass powder addition ended with a decrease in unconfined compressive strength. In addition, as the glass powder content was increased, both the liquid limit values and the plasticity index values decreased. When 36.0% glass powder was added, the liquid limit decreased by 23.5%. Also, the value of the plasticity index decreased by 31.3 % at 36.0 % glass powder addition. On the other hand, the plasticity index values did not show much change after 18.0 % glass powder addition. The plasticity limit values also remained constant for untreated and treated soil samples.

In another study, Mujtaba et al. [22] investigated the effect of powdered glass on fat clay by performing a modified compaction test, unconfined compression tests, one-

dimensional consolidation tests, and California bearing ratio (CBR) tests. The soil sample used is classified as high plastic clay (CH) and the powdered glass is classified as poorly sand (SP). The basic properties of the observed soil and the test results are given in Table 2.10.

Table 2.10 Results of the Conducted Tests with respect to Glass Powder Percentages [22]

| Glass Percentages (%) | 0.0 | 2.0 | 4.0 | 6.0 | 8.0 | 10.0 | 12.0 | 14.0 |
|--|--------|--------|--------|--------|--------|--------|--------|--------|
| Liquid Limit (%) * | 51.80 | 50.00 | 46.50 | 42.22 | 39.30 | 36.55 | 33.60 | 28.40 |
| Plastic Limit (%) * | 21.40 | 20.70 | 18.62 | 17.11 | 16.90 | 16.55 | 15.60 | 14.29 |
| Plasticity Index (%) * | 30.40 | 29.30 | 27.88 | 25.11 | 22.40 | 20.00 | 18.00 | 14.11 |
| Clay Fraction (%) * | 50.00 | 49.22 | 48.44 | 47.66 | 46.88 | 46.10 | 45.32 | 44.54 |
| Optimum Moisture Content (%) * | 18.20 | 17.40 | 17.10 | 16.70 | 16.30 | 16.00 | 15.40 | 14.90 |
| Maximum Dry Unit Weight (kN/m ³) * | 18.30 | 18.40 | 18.5 | 18.8 | 18.9 | 19.1 | 19.2 | 18.9 |
| Soaked CBR Values (%) * | 1.80 | 2.30 | 3.65 | 4.80 | 5.60 | 6.40 | 7.70 | 7.25 |
| Swell Pressure (kPa) * | 94.00 | 88.00 | 70.00 | 52.00 | 44.00 | 37.00 | 16.00 | 14.00 |
| Free swell (%) * | 3.9 | 3.5 | 2.9 | 2.2 | 1.9 | 1.4 | 0.9 | 0.5 |
| Unconfined Compression Strength (without curing) (kPa) * | 171.40 | 180.36 | 185.96 | 197.50 | 199.25 | 203.10 | 211.20 | 184.21 |

* These values are obtained from the related figures in the reference study.

As shown in Table 2.10, all consistency limits (liquid limit, plastic limit, plasticity index) were decreased with the increase in glass powder content. The maximum decrease in liquid limit value was found to be 45.2% when the glass powder addition was 14% of the dry weight of the soil sample. For the plastic limit values, the maximum decrease (a reduction of 33.2%) was obtained when the glass powder

addition was 14%. The decrease in both plastic and liquid limits eventually led to a decrease in the plasticity index. The author stated that the decrease in liquid limit and plasticity index due to the addition of glass powder was related to the non-plastic property of the glass powder used. Moreover, due to the effect of the additive on the Atterberg limits, the classification of the observed soil sample changed from fat clay (CH) to lean clay (CL). When the results of the modified standard proctor test were evaluated, an increase in maximum dry density was observed up to an addition of 12% glass powder. The increase in density up to 3.2% addition was related to the chemical reaction between the SiO_2 component in the glass powder addition and the Al_2O_3 component in the observed soil sample. The authors explained the decrease in the maximum value of dry density at 14% glass powder addition with the coarser glass powder particles that were not mobilized in the reaction in the soil sample and caused a decrease in the density values [22]. Another test performed in the study was the California bearing ratio (CBR). The CBR values increased from 1.80% to 7.70% when 12% glass powder was added. After a 12% addition of glass powder, the observed value decreased. In addition, the free swell and swell pressure of the untreated and treated soil were determined by one consolidation test. The addition of 14% of the glass powder (based on the dry weight of the soil sample) resulted in an 85% decrease in both the swell pressure and the free swell whereas the addition of up to 2% glass powder had no significant effect on the observed characteristics of the soil. Finally, the unconfined compressive strength test was performed by curing the samples with and without additives at different times. It can be seen that the values of unconfined compressive strength increase up to an addition of 12% glass powder and then decrease. The authors stated that, due to the chemical bond between the SiO_2 component in the glass powder addition and the Al_2O_3 component in the observed soil sample, unconfined compressive strength values showed an increase as time passes [22].

CHAPTER 3

MATERIALS

The study aims to investigate the changing behavior of the treated high plasticity clay, bentonite, through the addition and the waste expanded polystyrene beads and waste glass powder. In order to understand the effect of these materials on the engineering properties of bentonite, the basic properties of the materials are presented in this chapter.

3.1. Bentonite

The first settlers in the U.S. state of Wyoming discovered a clay-like material and called it mineral soap since the material was formed by the deposits on the surface. Over the years, the name of this material was changed a few more times and eventually renamed bentonite [23]. Today, bentonite is used in many different fields such as engineering geology, mining, and agriculture for various purposes [23]. In this study, bentonite is used as the main soil type representative of highly plastic clays, and the effect of selected additives on the soil properties is investigated. Therefore, it is very important to know the basic properties of bentonite used in this study in particular.

Bentonite can be defined as montmorillonite clay, formed by the decomposition of volcanic ash, and lava. The material is very rich in aluminum and magnesium. According to their montmorillonite compounds, bentonites can be divided into three main groups: Sodium bentonite, calcium bentonite, and sodium-calcium bentonite. One of the factors that necessitate the classification of bentonites is the change in their behavior when they are exposed to water [24]. For example, while sodium bentonite can swell to 8 to 10 times its volume upon contact with water, calcium bentonite can only swell to 2 to 3 times its volume in the same condition. On the other hand, the swelling potential of sodium-calcium bentonite is in between the swelling potential of

sodium and calcium bentonite, as sodium-calcium bentonite is obtained by mixing the other two types of bentonites [25]. Similar to their swelling potential, the usage areas of bentonite also show changes according to their types. Sodium bentonite is generally used in the drilling industry, foundry industry, and construction industry. In addition, calcium bentonite is usually used in the food industry for clarification, and the production of cat litter while sodium-calcium bentonite is used in the drilling industry [25].

Among these three types of bentonites, sodium bentonite has great commercial importance. However, the supply of sodium bentonite in nature is low. For this reason, other types of bentonites (calcium bentonite and sodium-calcium bentonite) are converted into sodium bentonite by some chemical methods [25]. Worldwide, bentonite reserves total 1870 million tons, with countries such as the United States, the United Kingdom, Japan, Greece, Italy, and Germany having the largest bentonite reserves. In Turkey, the total bentonite reserves are 281 million tons. Moreover, Edirne, Bilecik, Ankara, Eskişehir, Kütahya, and Çankırı are some of the cities that have the most important bentonite sources in the country.

In this study, commercially available bentonite is obtained from the Bilecik, the facility of Eczacıbaşı Esan Industrial Raw Material Company, and used. The chemical composition of the bentonite is determined by an X-ray fluorescence test at the facility of Earth Science Applications and Research Centre (YEBİM) of Ankara University and in Table 3.1. By conducting an experimental study, the basic properties of the bentonite used in this study are also determined and presented in Table 3.2 (See Appendix B for the datasheets of the parameters given in Table 3.2).

Table 3.1 Chemical Compound of the Used Bentonite

| Element | Value (%) | Element | Value (%) |
|--------------------------------|-----------|--------------------------------|-----------|
| SiO ₂ | 73.57 | TiO ₂ | 0.052 |
| Al ₂ O ₃ | 12.68 | MnO | 0.0691 |
| LoI | 7.84 | Na ₂ O | 0.039 |
| MgO | 2.259 | SO ₃ | 0.00687 |
| CaO | 1.318 | V ₂ O ₅ | 0.00093 |
| K ₂ O | 1.076 | Cr ₂ O ₃ | 0.00071 |
| Fe ₂ O ₃ | 0.741 | Cl | 0.0002 |
| P ₂ O ₅ | 0.0014 | | |

Table 3.2 Basic Properties of the Used Bentonite

| | |
|--|--------|
| Specific Gravity | 2.50 |
| Optimum Water Content (%) | 52.00 |
| Maximum Dry Density (g/cm ³) | 0.994 |
| Sand (%) | 0.00 |
| Fines (%) | 100.00 |
| Silt (%) | 19.00 |
| Clay (%) | 81.00 |
| Liquid Limit (%) | 161.69 |
| Plastic Limit (%) | 47.63 |
| Plasticity Index (%) | 114.06 |
| Soil Classification (According to USCS) | CH |

As shown in Table 3.2, the average specific gravity of the bentonite is obtained as 2.50, and as a result of the standard proctor test, the optimum moisture content of the sample is determined as 52.0%, and the maximum dry density of the soil is obtained as 0.994 g/cm³. In addition, the material completely consists of fine particles, which means all the tested soils have passed through the No. 200 sieve. By conducting a hydrometer test the percentages of silt and clay are obtained as 19%, and 81.00%, respectively.

The particle size distribution of the bentonite determined by the hydrometer test is shown in Figure 3.1 as follows.

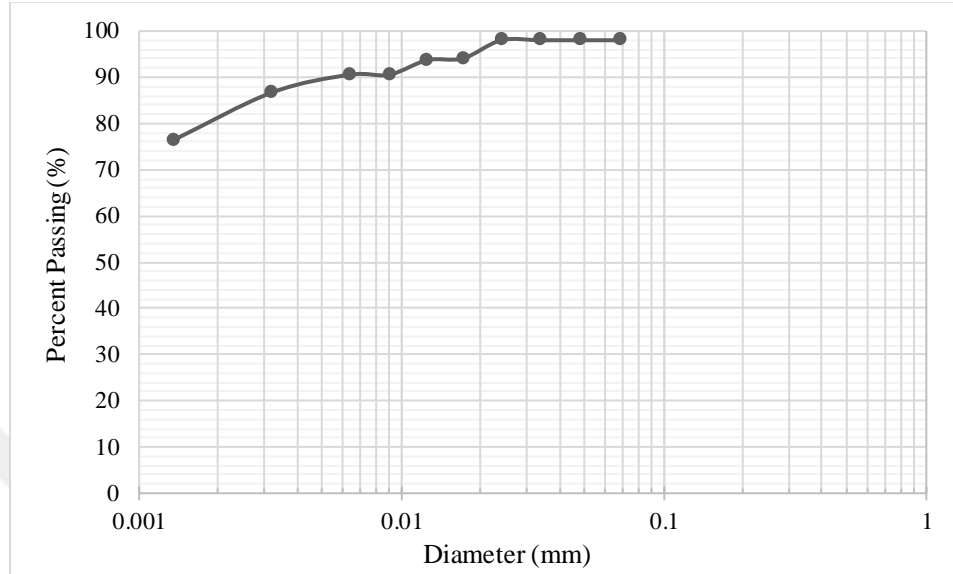


Figure 3.1 Particle Size Distribution of Bentonite (Hydrometer Test Result)

In addition, the Atterberg limit test is performed to determine the liquid limit, plastic limit, and plasticity index of the bentonite used. As a result, the liquid limit, plastic limit, and plasticity index of the material are obtained as 161.69%, 47.63%, and 114.06%, respectively (Table 3.2). Considering all this information about bentonite, the bentonite used is classified as high plasticity clayey soil (CH) using the Unified Soil Classification System [26].

3.2. Glass Powder

There are concerns about the environmental problems caused by waste glass since waste glass cannot decompose in nature. Air pollution, water pollution, high energy consumption, and high raw material consumption are just some of the environmental problems caused by waste glass material. One of the best ways to reduce these negative impacts of waste glass on the environment is to recycle this waste and create new uses

for it. In this way, not only the environment but also the natural sources are protected [27].

In Turkey, a document entitled "Regulation on Control of Solid Wastes" was published by the Ministry of Environment and Urban Planning in 1991. In this document, recycling was declared mandatory. However, while the total production of glass in Turkey is about half a million tons annually, only half of the production is put on the market and 32% of the remaining part is recycled [27]. Therefore, it can be concluded that recycling waste glass in Turkey requires further studies and efforts. Recycling waste glass can reduce energy consumption by 25%, air pollution by 20%, mining waste by 80%, and water consumption by 50% [28]. Considering these facts, waste glass is reused in various fields, including the construction sector. Waste glass can be combined with cement, concrete, and even bricks. Studies show that the durability of the final version of the materials increases with the increase of glass content [27].

Waste glass powder from a glass and ceramics factory in Sinop (Boyabat Osmanlı Cam Mozaik) is used in this study. The X-ray fluorescence test is performed at the facility of Earth Science Applications and Research Centre (YEBİM) of Ankara University. The test results are shown in Table 3.3 as follows.

Table 3.3 Chemical Composition of the Used Glass Powder

| Element | Value (%) | Element | Value (%) |
|--------------------------------|-----------|--------------------------------|-----------|
| SiO ₂ | 66.75 | TiO ₂ | 0.0511 |
| Na ₂ O | 10.35 | Al ₂ O ₃ | 0.009 |
| LoI | 9.75 | Cr ₂ O ₃ | 0.00891 |
| CaO | 8.258 | MnO | 0.00504 |
| MgO | 3.533 | P ₂ O ₅ | 0.0021 |
| Fe ₂ O ₃ | 0.2952 | Cl | 0.00131 |
| K ₂ O | 0.2205 | V ₂ O ₅ | 0.00091 |
| SO ₃ | 0.1977 | | |

The basic properties of the glass material used are determined by performing laboratory tests such as the specific gravity test, sieve analysis, and hydrometer test (See Appendix C for the datasheets of the tests performed). The results are presented

in tabular form in Table 3.4. As a result of the performed tests, the specific gravity of the glass is obtained as 2.56. On the other hand, a sieve analysis, and hydrometer test is performed and the particle size distribution of the glass powder is shown in Figure 3.2 below.

Table 3.4 Basic Properties of the Used Glass Material

| | |
|------------------|-------|
| Specific Gravity | 2.56 |
| Sand (%) | 73.00 |
| Fines (%) | 27.00 |
| Silt (%) | 26.4 |
| Clay (%) | 0.6 |

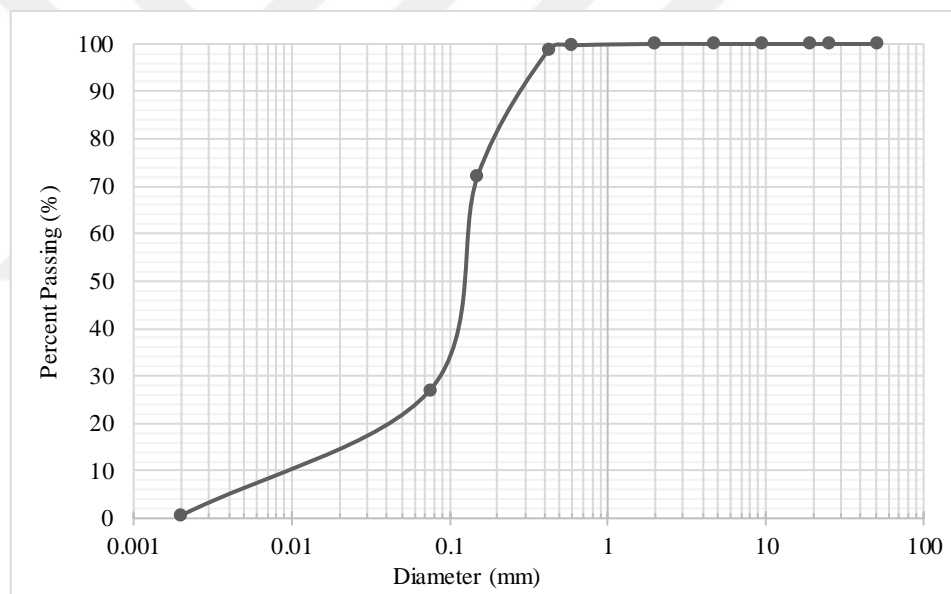


Figure 3.2 Particle Size Distribution of Glass

As a result of sieve analyses, the percent passing of sieve No. 30 is 99.7%. Therefore, it is decided to use the waste glass by sieving through sieve No. 30 to remove the particles whose diameter is larger than the opening width of sieve No. 30. It is also found that the sand content of the waste glass is 73.00% (Table 3.4). In addition to the sieve analysis, the particle size distribution of the fine particles is determined by

performing a hydrometer test. The silt and clay content in the waste glass powder is 26.4% and 0.6%, respectively.

Another important point to be mentioned about the glass, in general, the material provides a hydraulic bonding feature to the materials when they are combined with calcium hydroxide ($\text{Ca}(\text{OH})_2$) and water [29]. This combination can be named a pozzolanic reaction and is described by the following equations (Eqs. 3.1, and 3.2).



In Eqs. 3.1, and 3.2, the reaction takes place by combining silicate (SiO_2) and/or aluminate (Al_2O_3) with calcium hydroxide ($\text{Ca}(\text{OH})_2$) and water [29]. There are numerous studies in the literature on the pozzolanic effect of glass powder [30, 31, 32, 33, 34]. In the study by Shi et al. [34], the pozzolanic activity of four different sizes of glass powder on concrete is evaluated. The study shows that the pozzolanic activity of the glass increases as the size of the glass powder decreases. As a result, the strength also increases. Time is another factor on which pozzolanic behavior depends and, the strength at constant temperature increases with time due to the pozzolanic activity [34]. Furthermore, the SiO_2 and Al_2O_3 compounds in the glass powder have a positive effect on increasing the strength of the stabilized clays [34].

3.3. Expanded Polystyrene Beads (EPS)

Expanded polystyrene is a foamy, closed-cell, and typically white thermoplastic material derived from petroleum. The material is 98% dry air and 2% polystyrene. Even though the material is considered environmentally friendly and 100% recyclable, the negative impact of this material on nature should not be neglected. Recycling the material is not a common practice because it requires a lot of space due to its lightweight. For this reason, transporting the material from the place of origin to the place of recycling is very costly [35]. Thus, it is obvious that a large amount of EPS

waste is present in nature, which may cause serious problems. Since a large amount of EPS material may be needed for soil improvement, using the material for soil improvement can directly reduce EPS waste in nature. The waste EPS beads obtained from a company in İstanbul (İstanbul Strafor) are used in the study to reduce the swelling potential of the bentonite. The size of EPS used in soil sample treatment ranges from 1.00-4.75 mm as shown in Figure 3.3.



Figure 3.3 EPS Beads' Size Variation

CHAPTER 4

LABORATORY TESTS

An experimental study is carried out and the effect of waste materials as additives is investigated in this chapter. The specific gravity tests, sieve analyses, hydrometer tests, Atterberg limit tests, standard proctor tests, one-dimensional swell tests, and unconfined compressive strength tests are performed. The objective of these tests, the preparation of specimens for each test, the procedure, and the calculation steps are presented in this section of the thesis, respectively. All the tests are performed according to the standards given in Table 4.1.

Table 4.1 Used Standards for Each Test

| Test Method | Reference Standard |
|--------------------------------------|---|
| Specific Gravity Test | ASTM D854 [36] |
| Sieve Analysis Test | ASTM D6913 [37] |
| Hydrometer Test | ASTM D7928 [38] |
| Atterberg Limit Test | BS 1377: Part 2 [39] ASTM D4318 [40] |
| Standard Proctor Test | ASTM D698 [41] |
| One Dimensional Swell Test | ASTM D4546 [42] |
| Unconfined Compressive Strength Test | ASTM D2166 [43] |

4.1. Specific Gravity Test

Specific gravity can be described as the mass ratio of a unit volume of soil solids to the mass of the same volume of distilled air free water at 20 °C. The parameter is usually denoted by G_s . The calculated value for specific gravity can also be used to determine the phase relation of soils, such as void ratio, degree of saturation, and density of the soil [36].

In this study, the test is performed according to ASTM D854 [36]. The standard presents two different test methods: Method A and Method B. Method A is a procedure for moist specimens, whereas Method B is a procedure for oven-dry specimens. In this study, Method B is chosen as the test method. Therefore, sample preparation, test procedure, and calculation are performed to meet the requirements of ASTM D854, Method B. In this section, only information on Method B is presented, starting with the test equipment [36]. The test equipment can be listed as follows.

The test apparatus:

- Pycnometers (with a minimum capacity of 250 mL)
- Stoppers
- Balance (with 0.01 g readability)
- Drying oven (with a uniform temperature of $110\pm 5^{\circ}\text{C}$)
- Desiccator
- Entrapped air removal apparatus
- Vacuum system
- Sieve (No. 4 (4.75 mm) or finer)
- Water bath

For sample preparation, the material should generally be sieved through the No. 4 sieve (4.75 mm). The sieved sample should be placed in the oven for 24 hours at a constant temperature of $110 \pm 5^{\circ}\text{C}$. Similarly, pycnometers and stoppers should also be dried in the oven at $110 \pm 5^{\circ}\text{C}$. To determine the amount of material, the size of the pycnometer used for the specific gravity test should be considered according to ASTM D854, Table 1 [36]. The table used is given in Table 4.2 below.

Table 4.2 Recommended Mass of Test Specimen [36]

| Soil Type | Specimen Dry Mass for 250 mL Pycnometer (g) | Specimen Dry Mass for 500 mL Pycnometer (g) |
|---------------|---|---|
| SP, SP-SM | 60±10 | 100±10 |
| SP-SC, SM, SC | 45±10 | 75±10 |
| Silt or Clay | 35±5 | 50±10 |

As a first step towards performing a specific gravity test, the mass of the dry and clean pycnometers should be determined and noted. Then, the pycnometer should be filled with airless water completely. The air inside the water can be removed by either boiling, vacuuming, or a combination of these two methods. The pycnometer should be placed in a water bath at a constant temperature of $20\pm 1^\circ\text{C}$ for about one hour. The weight of the water-filled, air-free pycnometers should also be measured and recorded. By using the obtained data, the volume of each pycnometer should be calculated by using Eq 4.1, which is required for further calculations. In addition, by using Eq. 4.2, mass of the used pycnometers should be also determined and noted. If the test is going to be performed by using at least two pycnometers, the same procedures should be followed for each pycnometer.

As a next step, the samples, pycnometers, and stoppers should be dried in an oven at $110\pm 5^\circ\text{C}$ for 24 hours. A representative figure of the needed apparatus is presented in Figure 4.1a as follows. The dried soil should be poured through the funnels into the pycnometers (Figure 4.1b). All soil particles on the inside surface of the pycnometers should be rinsed with water. Until the half depth of the pycnometer is reached, water should be added to the top of the dried soil. By using a vacuum pump, the air inside the soil should be removed (Figure 4.1c). After all the air particles are removed from the soil, water should be fully-added to the pycnometers and stoppers should be inserted. The pycnometers should be placed in a water bath with a constant temperature of $20\pm 1^\circ\text{C}$ for one hour. Then, the pycnometers should be removed from the water bath, wiped, and dried. Next, the weight of the air-free water and soil should be determined and recorded. By transferring the soil slurry to a dish of known weight, the sample should be dried for 24 hours at a constant temperature of $110\pm 5^\circ\text{C}$ and allowed to cool. The dry mass of the soil slurry should also be noted and by using Eq.

4.3, which is presented below, specific gravity of the specimen should be determined. The average value for at least two specific gravity values should be reported as the real specific gravity of the specimen.

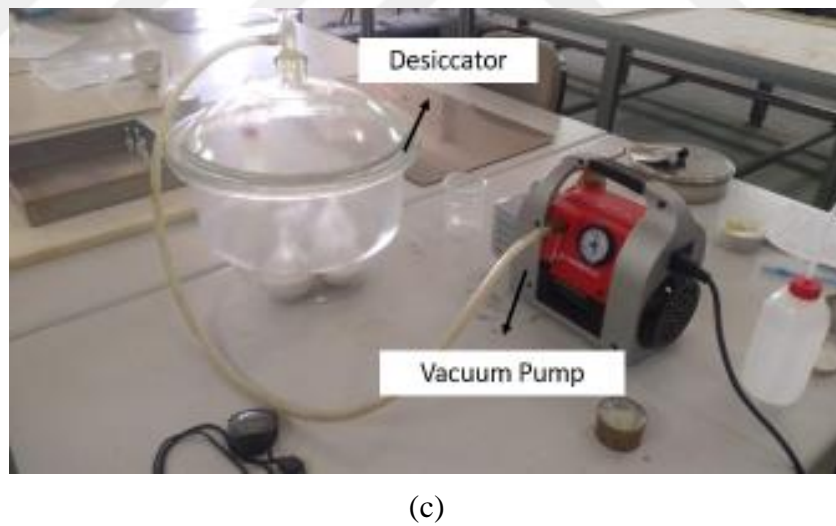
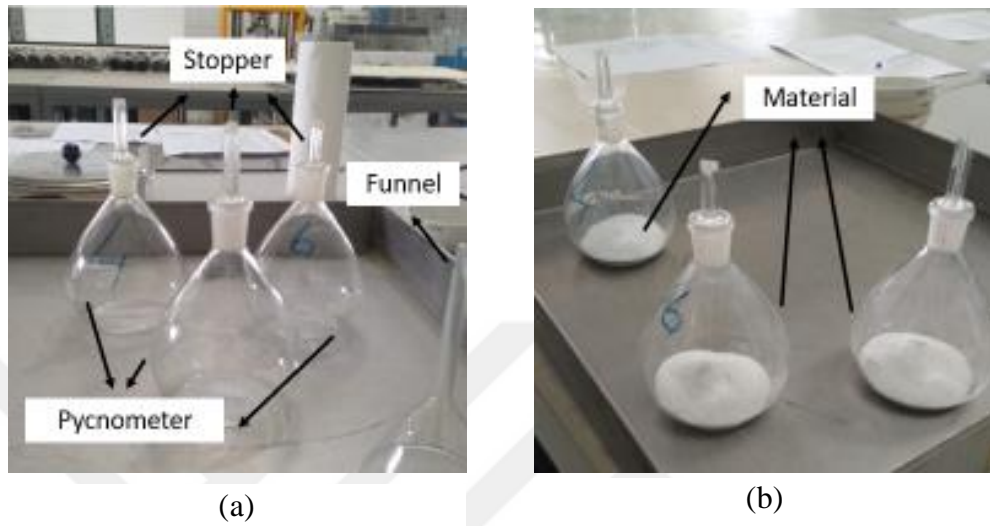


Figure 4.1 (a) Test apparatus (b) Placing the materials to the pycnometers
(c) Vacuuming the pycnometers

$$V_{\rho} = \frac{(M_{\rho w,c} - M_{\rho})}{\rho_{w,c}} \quad (4.1)$$

Where:

V_{ρ} = volume of pycnometer (mL)

$M_{\rho w,c}$ = the mass of pycnometer which is filled with water (g)

M_{ρ} = the mass of the dry pycnometer (g)

$\rho_{w,c}$ = density of water at test temperature (g/mL)

$$M_{\rho w,t} = M_{\rho} + (V_{\rho} \times \rho_{w,t}) \quad (4.2)$$

In the above equation:

$M_{\rho w,t}$ = mass of the pycnometer at test temperature (g)

M_{ρ} = mass of the dry pycnometer (g)

V_{ρ} = volume of the pycnometer (mL)

$\rho_{w,t}$ = the density of the water at test temperature (g/mL)

$$G_s = \frac{\rho_s}{\rho_{w,t}} = \frac{M_s}{(M_{\rho w,c} - (M_{\rho w,t} - M_s))} \quad (4.3)$$

Here:

G_s = specific gravity

ρ_s = the density of the soil (g/cm³)

$\rho_{w,t}$ = the density of the water at tested temperature (g/cm³)

$M_{\rho w,c}$ = the mass of the pycnometer that is filled with water (g)

M_s = the mass of the oven dry soil (g)

$M_{\rho w,t}$ = the mass of the pycnometer at tested temperature (g)

4.2. Sieve Analysis Test

To determine the particle size distribution of the soils, the sieve analysis test and the hydrometer tests are used. The sieve analysis tests are carried out to determine the distribution of the soil particles that are larger than 0.075 mm (No. 200 sieve).

The gradation curve can be used to determine the soil classification of the tested sample. According to USCS [26], soil samples passing through 3 in. (75 mm) and retained on the No. 4 sieve (4.75 mm) are defined as gravel while the soil samples passing through the No. 4 (4.75 mm) sieve and retained on the No. 200 (0.075 mm) sieve are defined as sand. Furthermore, the fine particles (silt and clay) are defined as the ones that pass through sieve No. 200 (0.075 mm). The standard sieve sizes other than those specified are listed in Table 4.3 with the corresponding standard opening sizes [37].

Table 4.3 Standard Sieve Sizes

| Alternative | Standard |
|-------------|-------------|
| 3 in. | 75.0 mm |
| 2 in. | 50.0 mm |
| 1-1/2 in. | 37.5 mm |
| 1 in. | 25.0 mm |
| 3/4 in. | 19.0 mm |
| 3/8 in. | 9.5 mm |
| No. 4 | 4.75 mm |
| No. 10 | 2.00 mm |
| No. 20 | 850 μ m |
| No. 40 | 425 μ m |
| No. 60 | 250 μ m |
| No. 100 | 150 μ m |

Table 4.3 (continued)

| Alternative | Standard |
|-------------|-------------------|
| No. 140 | 106 μm |
| No. 200 | 75 μm |
| Pan | |

According to ASTM D6913 [37], there are two methods for performing sieve analyses: Method A and Method B. Method B is applicable only when the maximum particle size is less than or equal to sieve No.4 (4.75 mm), and in this study, method B is used since all the soil sample passes through sieve No. 4 (4.75 mm). In addition, the dry method is chosen for soil preparation. The test equipment for the corresponding methods is listed below.

The test apparatus:

- Sieve set
- Mechanical sieve shaker
- Drying oven (with a uniform temperature of $110\pm 5^{\circ}\text{C}$)
- Balances (with four significant digits)
- Brushes

For the sieve analysis test, sieves of various sizes are stacked from the largest to the smallest opening, as shown in Table 4.3. According to the dry method, initially, the mass of a container, that is going to place the sample, should be determined and noted. After placing the sample in the container, it should be oven dried for 24 hours at $110\pm 5^{\circ}\text{C}$. Then, the dried weight of the sample should be determined and recorded. A sufficient number of sieves with various opening sizes should be assembled to use in the test. While deciding on the sieve's sizes, it should be considered that 100% of the material will pass through the largest sieve size. In addition, a pan should be placed at the bottom of the sieve stack and the sieves should be sorted from the largest sieve openings to the smallest ones. Following, the dried material should be poured into the sieve set and any retained material on the container should be brushed to make sure that all the recorded amount of the soil is tested. After placing a lid on the top of the

sieve set, using a sieve shaker, the sieves should be shaken for a 10 to 20 minutes period. Once the shaking is completed, the lid should be removed to check that there is no material retained on the largest sieve. Then, each sieve should be removed, and the retained samples on each sample should be transferred to the container of known mass. If any material is retained on the sieves, using a brush, the retained samples should be carefully removed from the sieves. After determining the mass retained on each sieve and performing necessary calculations, the gradation curve can be plotted.

The first step in plotting the gradation curve is to determine the cumulative mass retained on each sieve using the mass retained on each sieve during the test. Eq. 4.4 can be used to calculate the cumulative mass retained.

$$CMR_N = CMR_{N-1} + MR_N \quad (4.4)$$

Where:

CMR_N = cumulative mass retained on the Nth sieve (g)

MR_N = mass retained on the Nth sieve (g)

CMR_{N-1} = cumulative mass retained on above the Nth sieve (g)

Once the cumulative mass retained on each sieve is calculated from the above equation, the percent passing for each sieve can be determined using Eq. 4.5 as follows.

$$PP_N = 100 \times (1 - CMR_N / S, M_d) \quad (4.5)$$

Here:

PP_N = percent passing on the Nth sieve (%)

CMR_N = cumulative mass retained on Nth sieve (g)

S, M_d = dry mass of the specimen (g)

Then, the percent passing (x-axis) versus sieve opening (in a logarithmic scale) (y-axis) graph should be plotted to obtain the gradation curve of the tested soil or the material.

4. 3. Hydrometer Test

As mentioned earlier, the hydrometer test is used for the particle size distribution of soil particles smaller than sieve No. 200 (0.075 mm). The test method is also called sedimentation analysis since it is based on the sedimentation of the tested particles. The main principle of the hydrometer (sedimentation) test is Stoke's law. According to Stoke's law, larger particles fall through the liquid faster than smaller particles. There are several assumptions for Stoke's law. These assumptions can be listed as the particles are smooth and spherical; there is no interface between the particles; the flow is laminar, and the particles have the same density [38]. In this test, the density of the liquid soil mixture is measured considering Stoke's law and using a hydrometer at specific time intervals to determine the distribution of particle size of the observed soil. The hydrometer test is performed according to ASTM D7928 [38], and the particle size distribution of the fine samples is determined. The equipment used for the test can be listed as follows.

The test apparatus:

- Hydrometer (type 151H)
- Sedimentation cylinder (at least two sedimentation cylinders having a height of 457 mm, an inside diameter between 55 mm to 64 mm and a capacity of 1000 mL)
- Temperature device (with readability of 0.5 °C)
- Timing device
- Balance (with readability of 0.01 g)
- Drying oven (with a uniform temperature rate of $110 \pm 5^\circ\text{C}$)
- Plate

- Specimen mixing container
- Soil suspension oven-drying container
- Separation sieve
- Mechanical stirrer
- Squirrt bottle
- Sodium hexametaphosphate ((NaPO₃)₆)
- Desiccator (optional)
- Test water (distilled water)
- Porcelain evaporation dish

According to ASTM D7928 [38], there are two methods of specimen preparation: moist preparation and air-dried preparation. The moist preparation method is recommended for materials that cannot be air-dried. In addition, this method is recommended for soils such as organic soils, highly plastic fine-grained soils, tropical soils, and soils containing halloysite, because their properties are changed by drying. In this study, the moist soil preparation method is used. Therefore, only the information on this method is presented in this section.

In the hydrometer test, resolving the cohesion between the soil particles is important since each particle must be individual to observe its settlements at different velocities. Therefore, a dispersant such as sodium hexametaphosphate ((NaPO₃)₆) must be used. The agent prevents the fine particles from coalescing or flocculating. The combination of the soil sample tested, and the dispersant is called the sedimentation sample. About 50±10 g of the material to be tested should be taken for the test. Also, about 5±0.1 g of sodium hexametaphosphate should be added to the tested sample to ensure the separation of soil particles. The exact amount of dispersant added should also be determined and recorded. Test water may be added to the sedimentation sample, if necessary, to obtain a uniform mixture. The sedimentation sample containing the dispersant should be stored in a desiccator for 24 hours prior to testing. In addition, a reference solution should be prepared 24 hours before the test to obtain a correction for temperature density. To do this, the same amount of dispersant (5±0.1 g sodium

hexametaphosphate) should be added to another sedimentation cylinder filled with 1000 ± 1 mL water. The reference solution should stand overnight, and the top of the cylinder should be covered to prevent evaporation. During the test, the readings of the reference solution should also be noted at the same temperatures as those of the tested solution. These noted values are used in the calculations for the hydrometer test.

Another important point in the calculation of hydrometer tests is the meniscus correction. The main principle of the hydrometer is to read the values on the upper surface of the liquid. However, it is almost impossible to read the values at the top of the liquid because the liquid wets the glass and forms a concave shape around the hydrometer. While the correct reading must be taken at the lowest point of the concave shape, the readings are taken from the top of the meniscus during the test. Taking the reading slightly above the correct reading, causes the third digit of the particle size to change. Therefore, a meniscus correction should be made. This meniscus correction is given as C_m . A representative figure showing the main reason for the correlation can be found in Figure 4.2 [38].

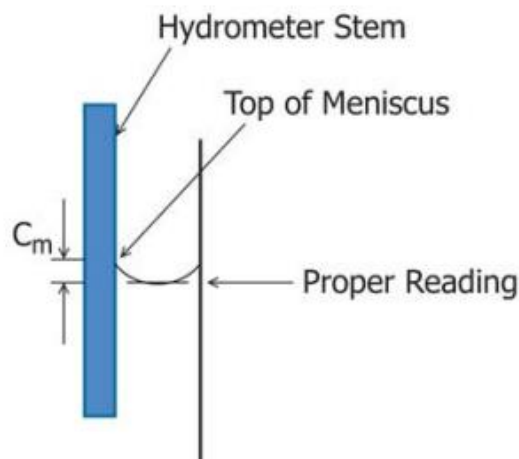


Figure 4.2. Meniscus Correction Diagram [38]

Firstly, the sedimentation sample with dispersant should be placed into a mechanical sifter. If necessary add water to the sample at this point of the test, and the sample should be mixed for about 1 minute, using the mechanical sifter. After a minute of mixing, the entire sample should be transferred to the sedimentation cylinder and the sedimentation cylinder should be filled with $1000 \text{ mL} \pm 1$ water. Prior to testing, the

slurry should be mixed by using the tipping method. In other words, the sedimentation cylinder should be turned upside down and back for about one minute to obtain a uniform suspension. Once, the last inversion is completed, the timer should be started and the hydrometer readings should be taken at 1, 2, 4, 8, 15, 30, 60, 240, and 1440 minutes. Additional readings may also be taken. Approximately, 15 to 20 seconds before taking the readings, the hydrometer should be placed in the cylinder to a level that approximately matches the level of the previous reading to provide a minimum amount of particle movement inside of the mixture. After taking the reading, the hydrometer should be removed and cleaned with the help of a wash cylinder as seen in Figure 4.3a. The hydrometer should be also dried before the next reading. Immediately after the hydrometer reading, the temperature of the suspension should be measured and recorded to the nearest 0.5°C . Once the last reading is taken, the soil suspension should be poured into an oven-dried container. The mass of the container and the suspension should be determined and noted, then dried for 24 hours in a drying oven at a constant temperature of $110\pm 5^{\circ}\text{C}$. After cooling down, the mass of the suspension with the container should be recorded (Figure 4.3b) and necessary calculations should be followed to obtain the particle size distribution of the fine-grained sample observed.



(a)



(b)

Figure 4.3 (a) Hydrometer in the wash bottle (b) Recording the weight of the oven dried sample

For the calculations, the first step should be to calculate the dry mass of the sediment sample by subtracting the mass of the dispersant from the mass of the dry soil with the dispersant [38].

$$M_d = M_{dd} - M_{disp} \quad (4.6)$$

Where:

M_d = mass of dry soil (g)

M_{dd} = mass of dry soil plus dispersing agency (g)

M_{disp} = mass of dispersant (g)

As mentioned earlier, if a reference solution is used, temperature-density corrections should be made to determine the offset of the hydrometer. The corrected hydrometer reading can be calculated for type 151H hydrometers using Eq. 4.7 as follows [38].

$$r_{d,m} = A - 7.784 \times 10^{-6} \times T_m - 4.959 \times 10^{-6} \times T_m^2 \quad (4.7)$$

In the above equation:

$r_{d,m}$ = 151H specific gravity hydrometer offset at reading m, nearest 0.0001

A = average specific gravity shift nearest 0.0001

T_m = temperature at reading m (°C)

In the third step, mass percent finer can be determined by using the following Eq. 4.8 for a type 151H hydrometer, as follows [38].

$$N_m = \left(\frac{G_s}{G_s - 1} \right) \times \left(\frac{V_{sp}}{M_d} \right) \times \rho_c \times (r_m - r_{d,m}) \times 100 \quad (4.8)$$

Where:

N_m = mass percent finer material at reading m

G_s = specific gravity of the soil nearest three digits

V_{sp} = volume of suspension (cm^3)

M_d = dry mass of the sedimentation specimen (g)

r_m = hydrometer reading in suspension at reading m

$r_{d,m}$ = hydrometer offset reading from reference solution at the same temperature

For each hydrometer measurement, the effective depth is utilized to calculate the particle fall distance. To calculate the effective depth, some parameters of the hydrometer must be known. These parameters are the distance between the center of the hydrometer and the maximum hydrometer reading (H_{r1}), the distance between the center of the hydrometer and the minimum hydrometer reading (H_{r2}), the maximum hydrometer reading (r_1), and the minimum hydrometer reading (r_2). These parameters are shown in Figure 4.4. In the figure (Figure 4.4), C_b represents the center of the hydrometer [38].

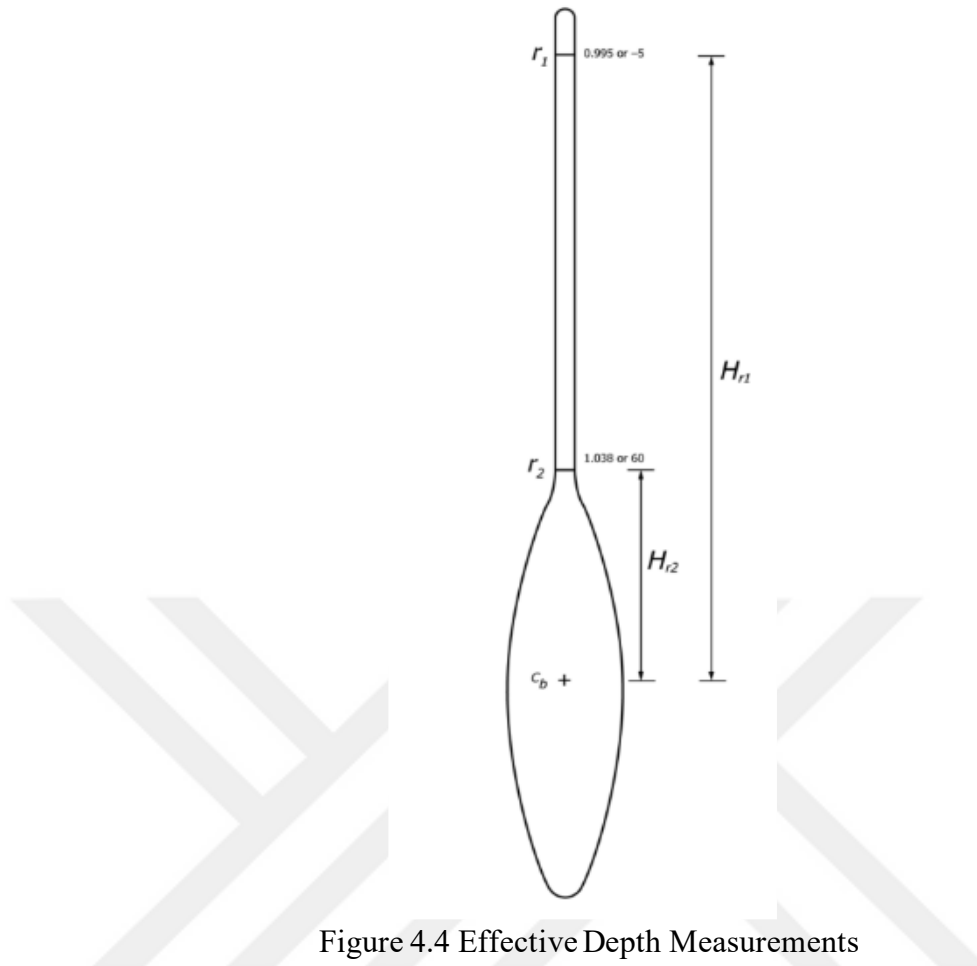


Figure 4.4 Effective Depth Measurements

To calculate the effective depth, Eq. 4.9 can be used as shown below [38].

$$H_m = H_{r2} + \left(\frac{(H_{r1} - H_{r2})}{(r_2 - r_1)} \times (r_2 - r_m + C_m) \right) - \left(\frac{V_h b}{2A_c} \right) \quad (4.9)$$

Where:

H_m = effective depth in 2 significant digits at reading m (cm)

H_{r1} = distance between the center of hydrometer to the maximum hydrometer reading (cm)

H_{r2} = distance between the center of hydrometer to the minimum hydrometer reading (cm)

r_1 = maximum hydrometer reading

r_2 = minimum hydrometer reading

r_m = hydrometer reading at reading m

C_m = meniscus correction

V_{hb} = volume of the hydrometer bump (cm³)

A_c = cross sectional area of the sedimentation cylinder (cm²)

For each hydrometer reading, the particle diameter of the soil in suspension is determined by using Eq. 4.10 as follows [38].

$$D_m = \left(\sqrt{\frac{18\mu}{\rho_w g (G_s - 1)}} \times \frac{H_m}{t_m} \right) \times 10 \quad (4.10)$$

Here:

D_m = particle diameter (mm)

μ = viscosity of water at 20°C, 0.01000 g/cm-s

ρ_w = density of the water at 20°C, 0.99821 g/cm³

g = acceleration of gravity, 980.7 cm/s²

G_s = specific gravity of the soil

H_m = particle fall distance at reading m (cm)

t_m = fall time at reading m (s)

Finally, the present passing sieve No. 200 (0.075 mm) can be determined using Eq.4.11 as follows [38].

$$P_p = 100 \times \left(1 - \frac{M_{dr}}{M_d}\right) \quad (4.11)$$

Where:

P_p = percent passing No. 200 (0.075 mm) sieve (%)

M_{dr} = dry mass retained on No. 200 sieve (g)

M_d = initial dry mass of the sedimentation specimen (g)

4.4. Atterberg Limit Test

The Atterberg limits, which refer to the degree of firmness, can be used to define the boundaries of the different consistency levels of soils liquid state, plastic state, semi-solid state, and solid state are the states of consistency of cohesive soils, and the water content corresponding to the transition from one state to another is known as Atterberg limits. The liquid limit is the water content at which soil passes from the liquid state to the plastic state. The plastic limit, on the other hand, is the water content that defines the boundary between the plastic and semi-solid phases. The Atterberg limit tests are carried out to determine the liquid limit, plastic limit, and plasticity index of soils. The test equipment, sample preparation, procedure, and calculations related to the determination of liquid limit and plastic limit are presented in this section of the thesis.

4.4.1. Liquid Limit Test

There are two methods for determining the liquid limit of the soil sample: the cone penetration method and the Casagrande method. In this study, the liquid limit of the samples is determined using the cone penetration method. Tests are performed according to BS 1377: Part 2: 1990 [39] and the required test equipment can be listed as follows and the test apparatus is presented in Figure 4.5.

The test apparatus:

- Palette knives
- Sieves
- Corrosion resistance container
- Flat glass plate
- Wash bottle
- Cone penetration device
- Metal cups (55 ± 2 mm in diameter, and 40 ± 2 mm in depth)
- Drying oven (at a uniform temperature rate of $110\pm 5^{\circ}\text{C}$)

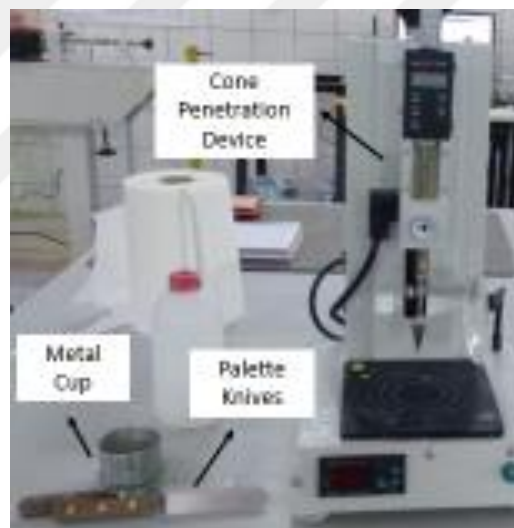
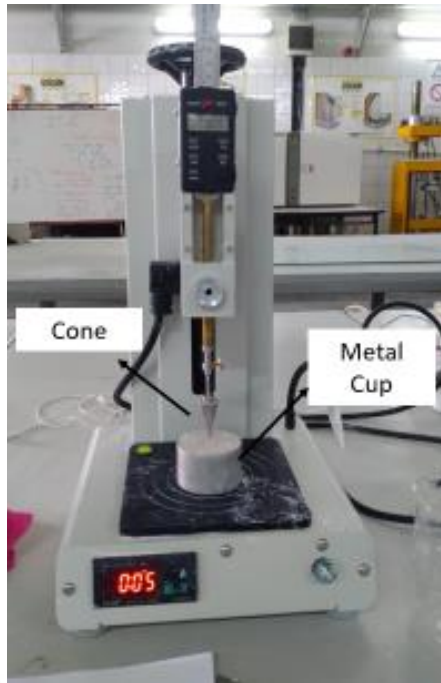


Figure 4.5 Liquid Limit Test Apparatus

Approximately 300 grams of the sample should be collected for sample preparation. If the particle size of the sample is larger than sieve No. 40 ($425\ \mu\text{m}$), it should be sieved before adding water. The amount of water added should be determined based on the previous experience to achieve a penetration depth greater than 15 mm. The sample should then be mixed with water and, the soil-water mixture is placed in a desiccator and kept for 36 hours [44]. Before the test, the sample should be mixed for at least 10 minutes but the recommended duration for highly plastic soils can be increased up to

40 minutes. By taking care to the removal of the excess air, the paste should be transferred into a metal cup. In addition, a smooth surface should be provided with the help of a spatula. Before releasing the cone, the tip of the cone should be moisturized, and the penetration cone should be aligned so that the tip of the cone just touches the surface layer of the sample as presented in Figure 4.6a and Figure 4.6b. Then, the penetration cone should be released for about 5 ± 1 seconds. The value on the dial gauge should be recorded as the penetration value (Figure 4.6c). Following, the cone should be removed, and the tip of the cone should be cleaned before the next reading. To obtain a smooth surface for the sample, a small amount of the sample should be added, and another measurement should be taken by following the same steps. One of the key points for the liquid limit test is if the difference between the two penetration values is not more than 0.5 mm, the two noted values can be used as the average penetration in determining the liquid limit values. However, if the difference between the two values is greater than 0.5 mm, but less than 1 mm, another measurement is needed. If the difference between the readings is even more than 1 mm, the soil should be removed from the cup and mixed again. Once the readings are consistent, by taking at least 10 grams of sample from the cup, the water content of the tested sample should be determined. To determine the liquid limit, the test should be repeated with different water contents. Therefore, water should be added to the tested material and mixed in a range of 10 minutes to 40 minutes by considering the soil type. After obtaining a uniform paste, the same test steps should be followed at least three times until the average penetration values between 15 mm and 25 mm are obtained. If this range is not achieved, further tests can be conducted. Then, necessary calculations can be performed to obtain liquid limit value of each sample.



(a)



(b)



(c)

Figure 4.6 (a) Initial reading settings (b) Arrangement of the cone (c) Dial gauge reading after penetration

For the calculations, the point relationship between the water content (x-axis) and the average penetration values (y-axis) should be plotted. Then, the best straight line that fits the points should be added. The water content value corresponding to the 20 mm of penetration from the best fit line should be determined and noted as the liquid limit of the observed sample [39].

4.4.2. Plastic Limit Test

The plastic limit of the specimens is determined by hand rolling according to ASTM D4318 [40]. The test equipment can be listed as follows.

- Ground glass plate
- Spatula
- Sieves
- Drying oven (at a uniform temperature rate of $110 \pm 5^\circ\text{C}$)

For sample preparation, at least 20 grams or more of soil samples sieved through the No. 40 sieve (425 μm) should be collected from the soil. Water should also be added to such a consistency that it can be easily rolled between the palm or finger and the ground glass plate. Thus, the amount of water should again be determined by trial and error or previous experience. After the preparation of specimens and curing time (at least 36 hours), the tests can be carried out. It has a great importance that during rolling mild pressure should be applied. Rolling should be continued until the sample reaches a uniform diameter of 3.2 mm, and crumble. If the soil crumbles without reaching 3.2 mm, it means that the sample has a water content lower than the plastic limits. If the sample reaches 3.2 mm in diameter without crumbling, it indicates that the water content of the sample is higher than its plastic limit. After providing satisfactory conditions for the plastic limit test, at least 6 grams of the sample should be taken for the determination of water content (Figure 4.7). The same steps should be followed one more time since the plastic limit values are obtained by taking the average of two test results. Also, these average water content values should be noted as the plastic limit of the samples.

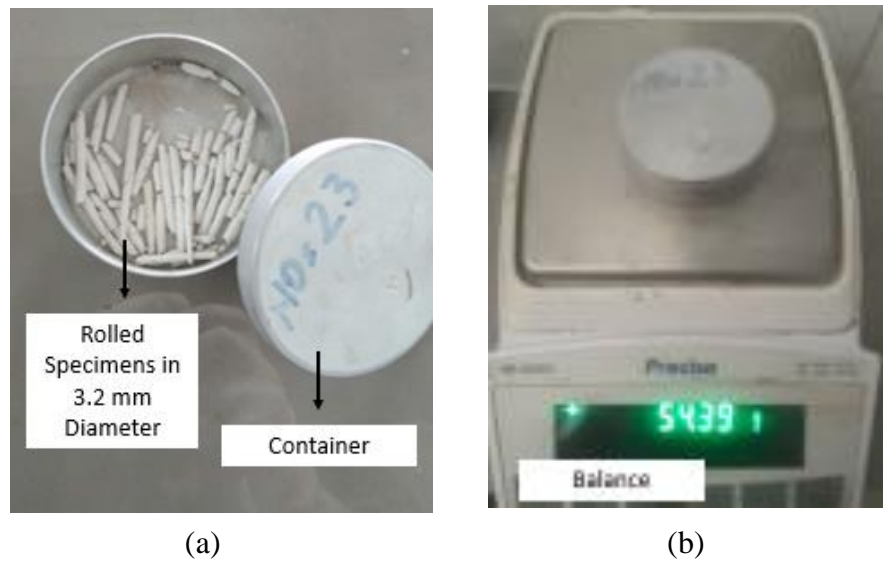


Figure 4.7 (a) Taking specimens for the water content determinations (b) Recording the dry weight of the sample with the container

4.4.3. Plasticity Index

The plasticity index (PI) can be defined as the range of water content in which the soil behaves plastically. It is calculated by subtracting the plastic limit from the liquid limit. The calculation of the plasticity index can also be represented in Eq. 4.11 as follows.

$$PI = LL - PL \quad (4.11)$$

Where:

PI = plasticity index (%)

LL = liquid limit (%)

PL = plastic limit (%)

4.5. Standard Proctor Test

Compaction is one of the most used soil improvement methods. It is aimed to decrease the air voids inside the soil by using different compaction methods such as smooth wheel rollers, sheep foot rollers, rubber-tired rollers, etc. Compaction also provides an increase in the unit weight and strength characteristics. At appropriate water content, soil compaction is optimal, and maximum dry density is obtained. To determine the optimum moisture content corresponding to maximum density, compaction tests should be performed. As a result of the compaction tests, the relation between the water content of the specimen and its dry density can be obtained. With increasing water content, the dry density values of the specimen also increase at a peak point and then, start to decrease. The water content corresponding to this maximum dry density is named optimum water content. In this study, the optimum moisture content and the maximum dry densities of the specimens are obtained by performing the standard proctor tests.

The standard proctor test is performed according to ASTM D698, Method A [41]. The information related to Method A is presented below, starting with the test apparatus (Figure 4.8).

The test apparatus:

- Mold (with 101.6 ± 0.4 mm inside diameter, and 116.4 ± 0.5 mm height)
- Mechanical rammer (which has a mass of 2.5 kg and has a drop of 30.5 mm)
- Sample extruder
- Balance
- Straightedge
- Drying oven (at a uniform temperature rate of $110 \pm 5^\circ\text{C}$)
- Sieves
- Mixing tools

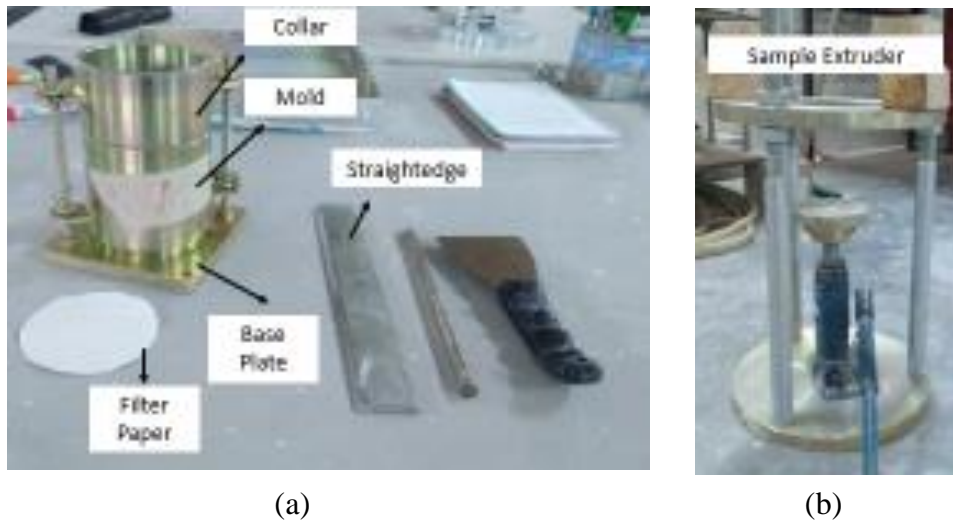


Figure 4.8 (a) Test apparatus (b) Sample extruder

For Method A, the number of compaction layers is three and the number of blows per layer is identified as 25. These blows should also follow a specific rammer compaction pattern as shown in Figure 4.9 [41].

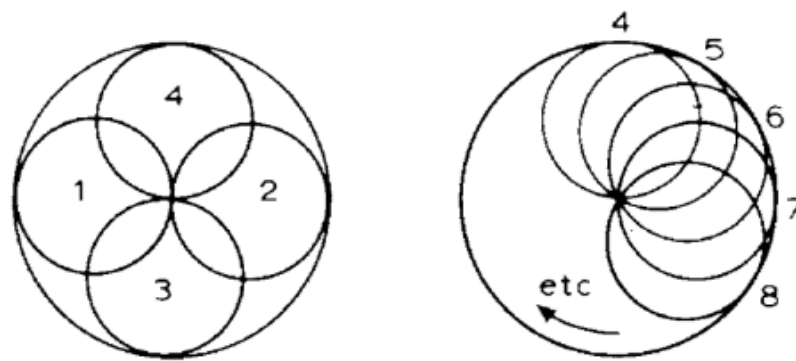


Figure 4.9 Rammer Compaction Pattern

The standard states that the previously compacted soil sample should not be reused while performing standard proctor test since the reused soil sample will give a higher value for maximum dry density due to yielding. For Method A, it is recommended that at least 16 kg of a soil sample should be prepared for the standard proctor test. In this study, the sample for the standard proctor test is prepared using the moist preparation method and for moist sample preparation, the sample should be sieved through a No.

4 (4.75 mm) sieve. At least four (preferably five) test samples with different water content values should be prepared. In selecting the water content values, the main objective should be to achieve water contents as close as possible to the optimum moisture content of the sample. For that manner, at least two of the selected water contents are below the optimum moisture content of the soil sample, while at least two of the points are above the optimum water content value. After deciding on the water contents, the specimens should be mixed thoroughly with water. However, the prepared mixtures should be cured before testing. The minimum standing time for the soils is given in the standard and shown in Table 4.4 for each soil sample [40]. But, in this study, as mentioned previously, the curing time for each sample is set at 36 hours since a CH soil is used [44].

Table 4.4 Minimum Curing Times for Standard Proctor Test [40]

| Soil Classification | Minimum Standing Time (h) |
|---------------------|---------------------------|
| GW, GP, SW, SP | No requirement |
| GM, SM | 3 |
| All other soils | 16 |

Before starting the test, the height, and diameter of the mold should be determined. Also, the weight of the mold with the base plate should be noted, too. After recording the necessary values, the mold, base plate, and collar should be assembled. The soil should be compacted into three equal layers by applying 25 blows for each layer (Figure 4.10a) by providing a uniform rate per minute by following the rammer pattern in Figure 4.9. Before the second, and third layers, the surface layer of the compacted soil should be trimmed. After the compaction of the last layer, by using a straightedge, compacted soil should be trimmed to obtain a plane surface. If needed, holes in the surface of the compacted soil should be filled. As a next step, the weight of the compacted soil with the mold and the base plate should be determined (Figure 4.10b). After recording the weight, the compacted soil should be removed from the mold (Figure 4.10c) and a small amount of soil sample should be taken to determine the water content of the compacted soil. By performing the same steps at least four times with different water contents, the compaction curve of the sample can be plotted.

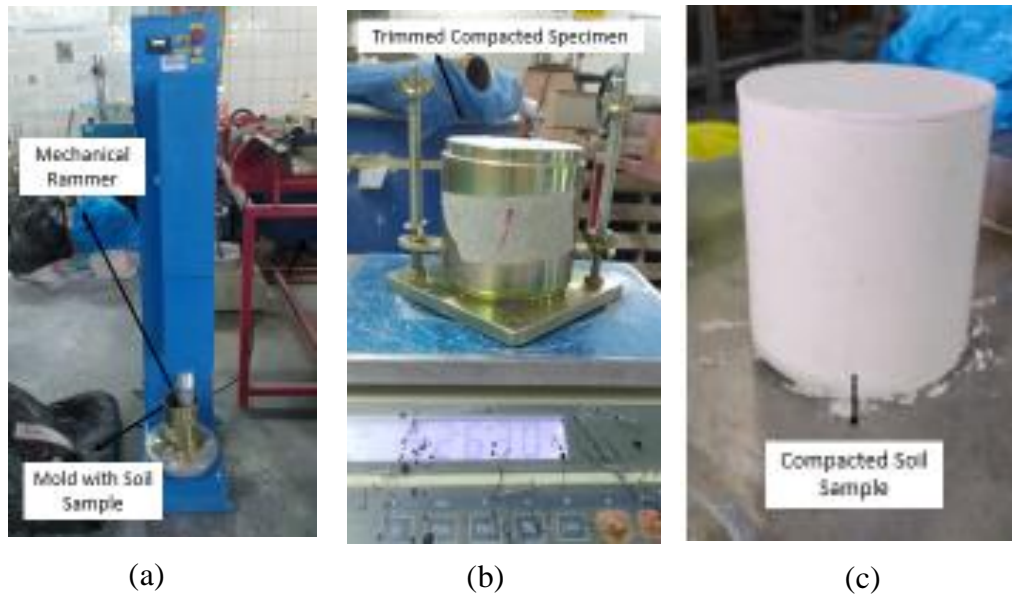


Figure 4.10 (a) Compacting the specimen by using mechanical rammer (b) Recording the weight of the trimmed compacted specimen (c) Compacted soil sample removed from the mold

Calculations are carried out according to ASTM D698 [40] and the relevant equations are presented as follows. As a first step, the moist density of each sample should be calculated using Eq. 4.12 below.

$$\rho_m = K \times \frac{(M_t - M_{md})}{V} \quad (4.12)$$

Where:

ρ_m = moist density of the compacted specimen (g/cm³)

M_t = mass of moist soil with the mold and the base plate (g)

M_{md} = mass of the compaction mold with the base plate (g)

V = volume of the compaction mold (cm³)

K = conversion constant, depending on density units and volume units which is equal to 1 for g/cm³ and volume in cm³

After calculating the moist density according to the Eq. 4.12, the dry density values can be determined using Eq. 4.13.

$$\rho_d = \frac{\rho_m}{1 + \frac{w}{100}} \quad (4.13)$$

Where:

ρ_d = dry density of compaction (g/cm³)

w = molding water content (%)

Before plotting the compaction curve, Eq. 4.14 can be used to determine the dry unit weight of the compacted sample.

$$\gamma_d = K_2 \times \rho_d \quad (4.14)$$

Here:

γ_d = dry unit weight of compacted specimen (kN/ cm³)

K_2 = conversion constant, depending on density units which is equal to 9.8066 for density in g/cm³

After these calculations, the compaction curve can be drawn using the dry unit weight and water content values. The value corresponding to the peak value of the compaction curve on the x-axis indicates the optimum water content. The value corresponding to the peak value of the compaction curve on the y-axis is noted as the maximum dry density.

4.6. One Dimensional Swell Test

Cohesive soils attract water molecules towards their particles due to their absorbing power. This causes the water molecules to form a layer around the soil particles, resulting in an increase in the volume of the soil. This volume change can be referred to as swelling. The minimum pressure applied to prevent this volume change is called the swelling pressure. On the other hand, the percentage swell after the absorption of water, swell under 1 kPa of seating pressure is called free swelling. It is possible to determine both the swelling pressure and the percentage of free swell of expansive soils by a laboratory experiment. In the experiment, the sample is loaded vertically in a rigid mold and only the vertical swell is observed. Therefore, the test method is also known as the one-dimensional swell test. In this study, the effect of additives on the free swell of the soil sample is investigated.

According to ASTM D4546 [41], three different test methods are valid: Method A, Method B, and Method C. These methods should be selected depending on the type of soil being tested. For example, Method A should be preferred for reconstructed samples, Method B for natural soils or existing fills, and Method C for the combination of both cases. In this study, Method A is chosen as the test method because the soil sample is reconstituted. The test apparatus can be listed below (Figure 4.11).

The test apparatus:

- Consolidometer
- Specimen ring (with a minimum diameter of 50 mm, and a minimum height of 20 mm)
- Porous disk
- Aluminum foil
- Micrometer
- Deformation indicator
- Drying oven (at a uniform temperature rate of $110 \pm 5^\circ\text{C}$)
- Balances

- Water content containers
- Test water

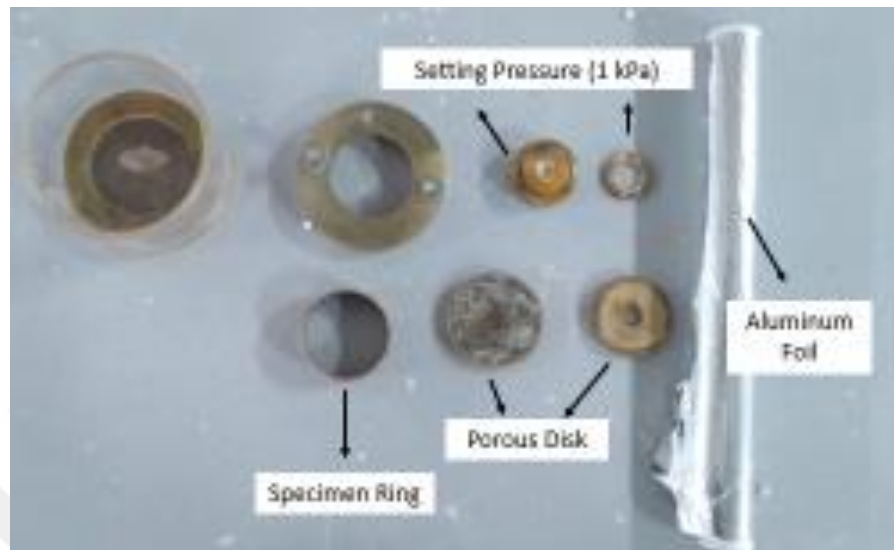


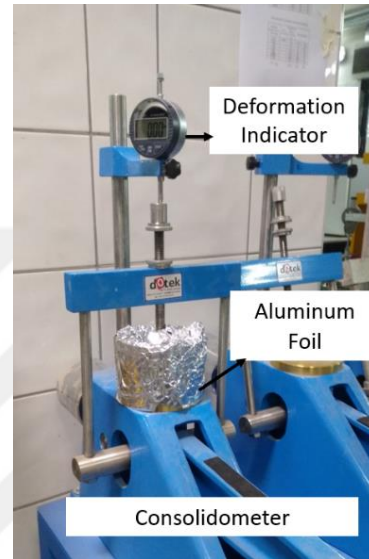
Figure 4.11 One dimensional swell test

Prior to testing, the weight of the specimen ring, also the diameter, and height of the ring should be determined. To obtain the sample for testing, at first, a standard proctor test should be performed per ASTM D698 [41] at the optimum moisture content and maximum dry densities of the sample. Using the sample ring, an undisturbed sample from the compacted soil should be extracted. Also, a small amount of sample should be taken for the initial water content determinations. As a next step, the mass of the specimen ring and the sample should be determined, as presented in Figure 4.12a, in addition to the initial height of the sample, which directly corresponds to the height of the ring. Then, the specimen can be placed in the consolidation device by applying a seating pressure of 1 kPa, including the mass of the top porous stone, and load plate. During testing, to minimize the change in the water content of the sample during the consolidation phase, the sample should be enclosed with aluminum foil (Figure 4.12b). The inundation pressure should not be applied for more than 1 hour. After reading the amount of compression, the sample should be inundated with distilled water and the readings should be taken at time intervals of 0.5 minutes, 1 minute, 2 minutes, 4 minutes, 8 minutes, 15 minutes, 30 minutes, 1 hour, 2 hours, 4 hours, 8 hours, and 24

hours (Figure 4.12c). Once the first 24-hour period is completed, the readings can be taken at 24-hour intervals until the two sequential readings are the same, which shows the swelling is completed. To end the test, test water, and the loads should be removed. Then, the weight of the sample with the ring should be noted after wiping out the remaining water on the sample with the help of a filter paper. Also, the final height and the final water content of the specimen should be also recorded.



(a)



(b)



(c)

Figure 4.12 (a) Recording the initial weight of the sample (b) Compression phase of the specimen (c) Taking readings of the observed specimens

For the calculation of free swelling, the relevant equations are presented according to ASTM D4546 [41]. The final height of the sample after wetting can be calculated using the following Eq. 4.15. Finally, free swell is calculated by Eq. 4.16.

$$\rho_1 = \frac{M}{V} \quad (4.15)$$

Where:

ρ_1 = bulk density (g/ cm³)

M = specimen mass (g)

V = specimen volume (cm³)

$$\rho_{d1} = \frac{\rho_1}{1+w_1} \quad (4.16)$$

Where:

ρ_{d1} = dry density (g/ cm³)

ρ_1 = bulk density (g/ cm³)

w_1 = initial water content (%)

4.7. Unconfined Compressive Strength Test

The unconfined compressive strength is a special type of tri-axial test in which the confining pressure is zero. The unconfined compressive strength can be defined as the compressive stress value at which the unconfined cylindrical specimen fails due to the applied axial load, and it is denoted as q_u . In this study, the unconfined compressive strength tests are performed according to ASTM D2166 [42]. The equipment used for the test can be listed as follows (Figure 4.13).

The test apparatus:

- Compression device
- Sample extruder
- Deformation indicator
- Dial comparator
- Balance
- Drying oven (at a uniform temperature rate of $110 \pm 5^\circ\text{C}$)

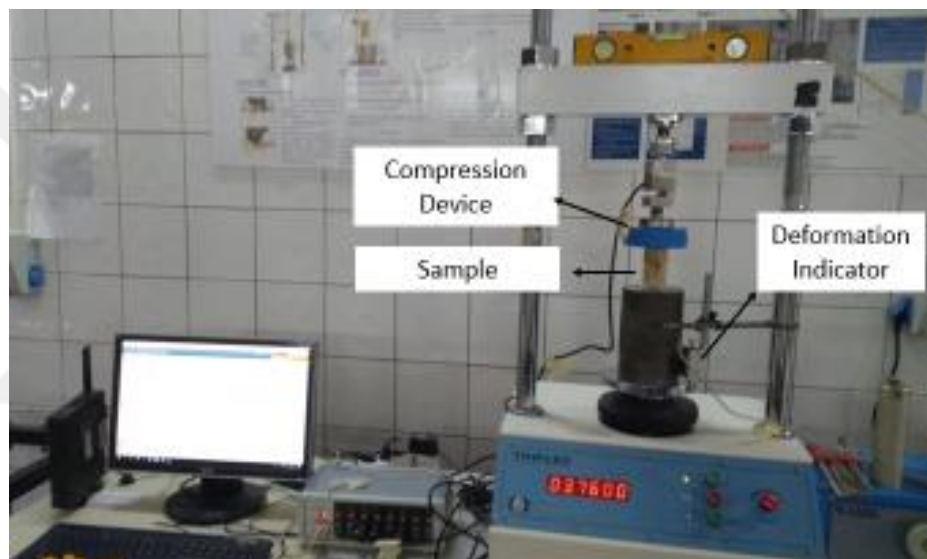


Figure 4.13 Unconfined compression strength test apparatus

Tests samples are extracted from standard proctor test mold in which the soil samples compacted at their optimum moisture contents and maximum dry densities according to ASTM D698 [40]. The extracted specimens must have the minimum diameter and height to diameter ratio. According to ASTM D2166 [42], the minimum diameter of the sample taken should not be less than 30 mm, while the height to diameter ratio must be between 2 or 2.5.

The diameter and the height of the extracted sample should be noted by taking the average of three different measurements. The specimen should be placed on the loading device and the specimen should be centered with the loading plate (Figure

4.14a). By adjusting the loading device so that it just touches the top of the specimen, the load can be applied at an axial strain of 2% per minute through the loading device. The test should be continued until the load values decrease with increasing stress or 25% of the maximum strain is reached (Figure 4.14b). By using the simultaneously recorded load, deformation, and time values, necessary calculations are carried out to obtain the unconfined compressive strength of each specimen. In order to calculate the strength, the maximum axial load recorded during the test is divided to the corrected cross-sectional area of the specimen. To calculate the corrected cross-sectional area, first the axial strain of the specimen during is calculated by using Eq.4.17.

$$\varepsilon_1 = \frac{\Delta L}{L_0} \times 100 \quad (4.17)$$

Where:

ε_1 = axial strain (%)

ΔL = length change of the specimen (mm)

L_0 = initial length of the specimen (mm)

Then, the corrected cross-sectional area for each applied load can be determined using Eq. 4.18. Finally, the value of the compressive stresses can be calculated by dividing the applied load by the corresponding corrected area, as shown in Eq. 4.19.

$$A = \frac{A_0}{\left(1 - \frac{\varepsilon_1}{100}\right)} \quad (4.18)$$

Where:

A = corrected area (mm²)

ε_1 = axial strain (%)

A_0 = initial average cross sectional area (mm^2)

$$\sigma_c = (P/A) \quad (4.19)$$

Where:

σ_c = compressive stress (kPa)

P = given applied load (kN)

A = corresponding corrected area (mm^2)

Once all the necessary calculations have been made, the compressive stress (ordinate) can be plotted against the axial strain (abscissa). From the plotted graph, the maximum compressive stress is reported as the unconfined compressive strength (q_u).

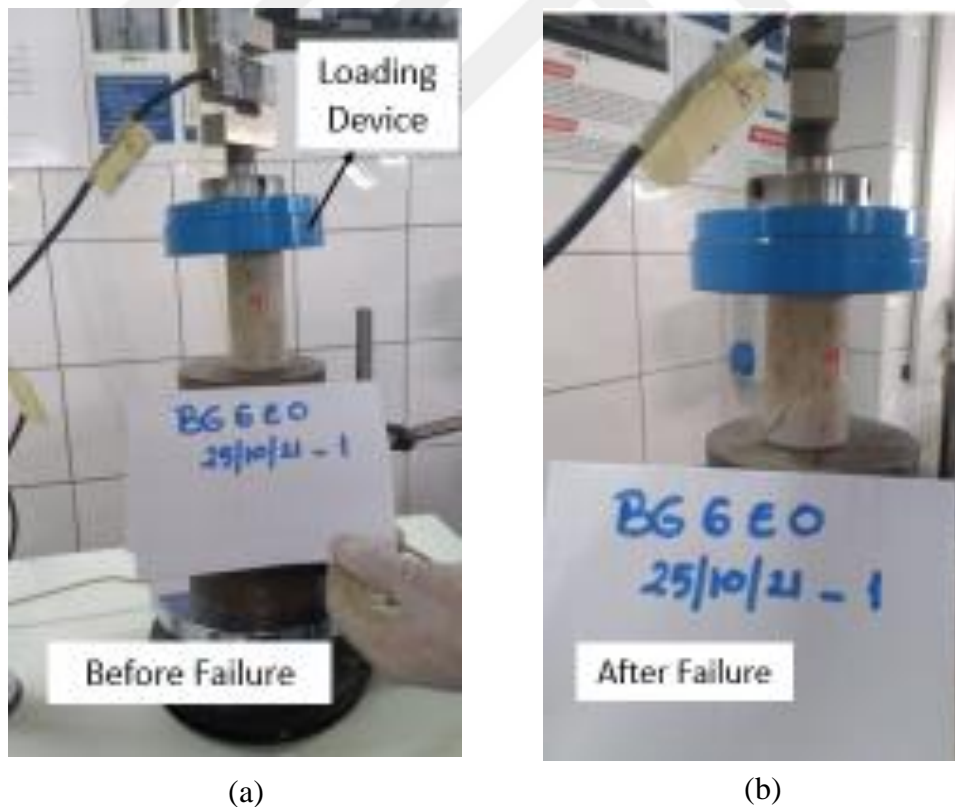


Figure 4.14 (a) Sample before failure (b) Sample after failure

CHAPTER 5

EXPERIMENTAL STUDIES, TEST RESULTS AND DISCUSSIONS

In this chapter, the effects of additives (EPS and/or glass powder) on the index and engineering properties of bentonite are investigated by laboratory tests. The sample preparation and the results of the experiments carried out on the bentonite used with the different percentages of the additives are presented. As stated, glass powder, has a positive effect on the unconfined compressive strength values of soil samples, whereas EPS beads have the potential to decrease the free swell of the soil samples when they are used as additives. In this study, in addition to investigating the effect of additives individually, the effect of both of them on the index and engineering properties of the soil sample is examined. The optimum additive percentages of glass powder and EPS beads are found to obtain the maximum increase in the compressive strength value of the soil sample and the maximum reduction in the swelling index.

5.1. Sample Preparation

The treated soil samples are prepared by mixing the bentonite with additives and distilled water and kept in plastic sealed bags for a minimum of 36 hours ASTM 3080 [44]. The index properties, optimum moisture content, and maximum dry density values for all treated soil samples are determined according to standards shown in Table 4.1. The untreated and treated soil samples for free swell and unconfined compression tests are extracted from the molds in which the soil is compacted according to the optimum moisture content and maximum dry densities of corresponding additives by standard proctor test apparatus.

A wide range of glass powder by the dry mass of the soil sample is used as an additive in the previous relevant studies [17, 18, 19, 20, 21, 22] while a narrower range for EPS usage is selected for treatment of clayey soils [13, 14, 15, 16]. In this study, a common

range for both of the stabilizing agents is selected that is ranging from 0 to 6% for glass powder and 0 to 2% for EPS. The effect of the additives is also examined by using the selected following percentages:

- Three different glass powder percentages are decided as 2%, 4%, and 6%, of the dry mass of the prepared specimens.
- Three different EPS bead percentages are decided as 0.3%, 0.9%, and 2% of the dry mass of the prepared specimens.
- Selected combinations of EPS beads and glass powder together.

The prepared samples are named according to the additive percentages of EPS and glass powder. For the naming, the symbol “BG_iE_j” is used where, B, G, and E represent bentonite, glass powder, and expanded polystyrene beads, respectively. The “i” and “j” after the letters G and E denote the percentage of glass powder and expanded polystyrene beads by the dry mass of the bentonite, respectively. Figure 5.1 shows an example of the naming of a sample containing 4% glass powder and 0.9% EPS, based on the dry weight of the sample. For the rest of the study, this naming is used for the identification of each sample.

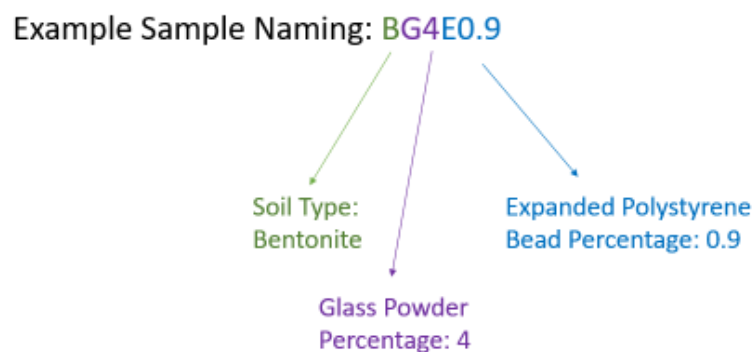


Figure 5.1 An Example of Sample Naming for Bentonite with 4% of Glass Powder and 0.9% of EPS Addition

5.2. Atterberg Limit Test Results

The Atterberg limit tests are performed to determine the liquid limit, plastic limit, and plasticity index of untreated and treated soil samples according to ASTM D4318 [40] and BS:1377 [39]. The test results are presented in Table 5.1.

Table 5.1 Atterberg Limit Test Results

| Sample Name | Liquid Limit (%) | Plastic Limit (%) | Plasticity Index (%) |
|-------------|------------------|-------------------|----------------------|
| BG0E0 | 161.69 | 47.63 | 114.06 |
| BG2E0 | 157.19 | 45.27 | 111.92 |
| BG4E0 | 153.46 | 44.44 | 109.02 |
| BG6E0 | 148.08 | 43.77 | 104.31 |
| BG0E0.3 | 154.24 | 44.45 | 109.79 |
| BG0E0.9 | 159.33 | 45.07 | 114.23 |
| BG0E2.0 | 159.40 | 43.37 | 116.03 |
| BG2E0.9 | 156.95 | 44.80 | 112.15 |
| BG4E0.9 | 152.57 | 45.92 | 106.65 |
| BG6E0.9 | 146.92 | 43.50 | 103.42 |

As shown in Table 5.1, when glass powder is used exclusively as an additive (BG2E0, BG4E0, BG6E0), both the liquid limit and the plastic limit decrease with increasing glass powder content, resulting in a decrease in the plasticity index. Since glass powder is a cohesionless material, a decrease in the values of the plasticity index is expected, and the effects of glass powder on the liquid limit, plastic limit, and plasticity index values are consistent with the relevant studies in the literature [17, 18, 19, 20, 21, 22]. It is even possible to see the effect of the non-cohesive property of the glass powder on the liquid limit, plastic limit, and plasticity index values when it is used together with a 0.9% EPS additive (BG2E0.9, BG4E0.9, BG6E0.9). These changes in the Atterberg limit parameters can be accepted as a sign that the strength parameters of the soil have improved.

On the other hand, it can be observed that the liquid limit of the treated soil samples with only EPS (BG0E0.3, BG0E0.9, BG0E2) first decreases and then increase and

remain the same with increasing additive percentage, whereas the plastic limit does not show a trend. Also, it should be noted that during the plastic limit tests of samples with EPS additives, even the EPS is sieved prior to mixing with the bentonite and special care is given during the plastic limit test, some of the EPS beads detached from the sample during rolling. The red circles in Figure 5.2 show the detached EPS beads. Although plasticity index values show a similar trend to liquid limit, the plastic limit values may also be affected by this issue. In view of this, it is believed that the assessment of the strength due to treatment should be evaluated after further tests.



Figure 5.2 Detachment of EPS Beads

5.3. Standard Proctor Test Results

In order to determine the maximum dry density and optimum moisture content of the treated and untreated samples, standard proctor tests are performed according to ASTM D698 [41]. All of the results are presented in Table 5.2 and the obtained test results are evaluated in three subgroups to assess the effects of each additive at a time and the effects of both additives.

Table 5.2 Standard Proctor Test Results

| Sample Name | Optimum Moisture Content (%) | Maximum Dry Density (g/cm ³) |
|-------------|------------------------------|--|
| BG0E0 | 52.00 | 0.994 |
| BG2E0 | 50.20 | 0.990 |
| BG4E0 | 51.22 | 1.003 |
| BG6E0 | 51.42 | 1.012 |
| BG0E0.3 | 52.00 | 0.914 |
| BG0E0.9 | 51.95 | 0.864 |
| BG0E2.0 | 48.00 | 0.745 |
| BG2E0.9 | 48.30 | 0.842 |
| BG4E0.9 | 50.60 | 0.843 |
| BG6E0.9 | 50.80 | 0.846 |

Figure 5.3 shows the effect of the addition of glass powder alone on the compaction curve of the bentonite.

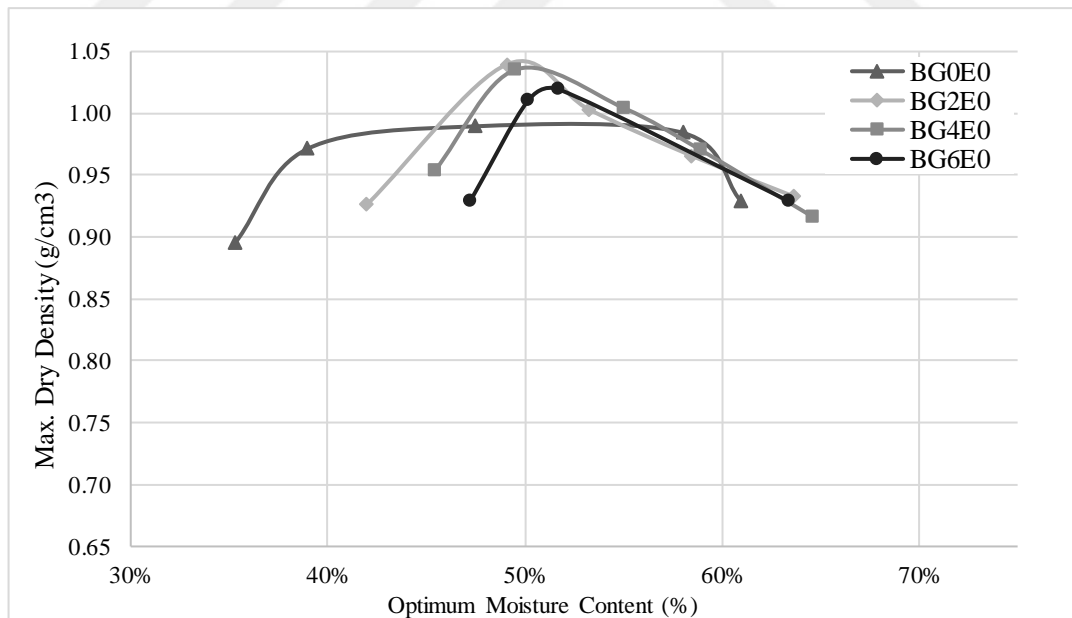


Figure 5.3 Compaction Characters of Only Glass Powder Treated Samples

As shown in Table 5.2 and Figure 5.3, although the change in the optimum water content is very small, it can be noted that the values begin to decrease and then increase

and remain constant as the glass powder content increases. Ibrahim et al. [21] stated that, since the SiO_2 element has a lower surface area than the soil samples, the addition of glass powder, which has a high SiO_2 content, decreases the specific surface area of the soil, resulting in a decrease in the optimum moisture content. However, in this particular study, the bentonite used has a higher SiO_2 content than the glass powder used (Tables 3.1 and 3.3). Therefore, the addition of higher proportions of glass powder may decrease the average SiO_2 content in the bentonite, increasing in the surface area of the sample. Additionally, the maximum dry density values increase with increasing glass powder content. Besides, previous studies in the literature showed that as the glass powder content increases, the optimum moisture content values of the samples decrease [17, 18, 19, 20, 21, 22]. It can be concluded that the results of these tests are in agreement with the previous findings in the literature.

In the second group, the effect of EPS on the compaction properties of the prepared specimens is evaluated. The variation of compaction properties with EPS admixture is shown in Figure 5.4.

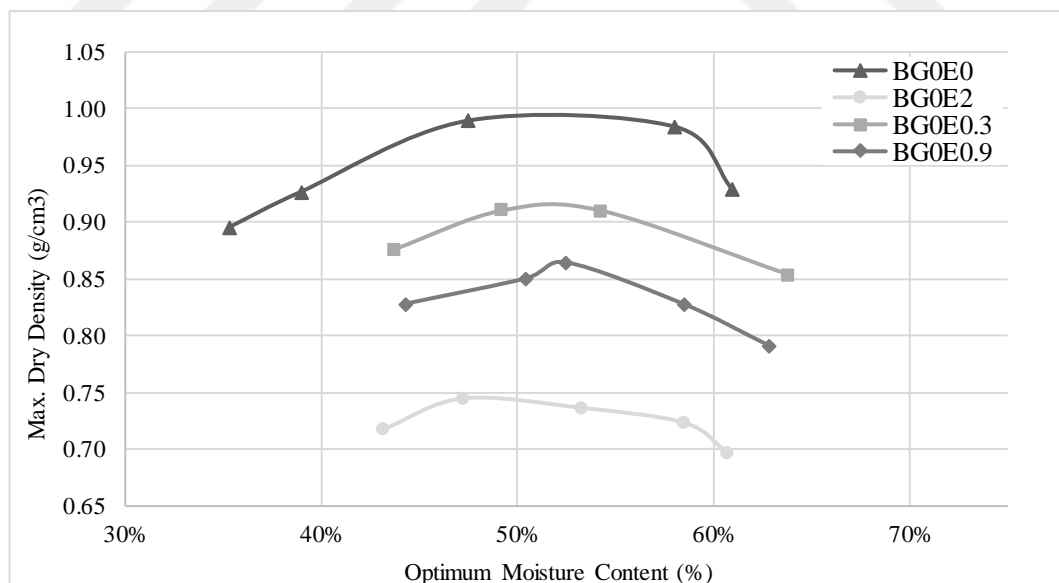


Figure 5.4 Compaction Characters of Only EPS Treated Samples

It is observed that, that both the optimum moisture content and the maximum dry density decrease with increasing EPS content. One of the reasons for the decrease in

maximum dry density may be the reduced compatibility of treated soil with EPS beads. This property ensures that the material can absorb more energy, resulting in a decrease in maximum dry density values. In addition, since the material has a low bulk density and weight, these properties also lead to a reduction in the maximum dry density [14]. Compared to the effect of EPS beads on the maximum dry density values, the effect of EPS on the optimum moisture content is less and due to the flat compaction curves, it is not easy to estimate the optimum moisture content values. However, as shown in Table 5.2 and Figure 5.4, the optimum moisture content values decrease as the percentage of the additive increases for this particular study, and the change in both of the compaction properties due to the effect of EPS beads gives similar results to the studies in the literature [14, 15].

In the last group, the effect of both of the additives on the compaction curves is examined together. The variation in compaction properties obtained by performing the standard proctor test is shown in Figure 5.5. The optimum moisture content values of soil samples initially decrease and then increase slightly and remain constant with 0.9% of EPS and increasing glass powder percentage. The maximum dry density values also increase with increasing glass powder addition. It can be concluded that the effect of the glass powder outweighs the effect of the EPS on the compaction properties and leads to similar changes as for the specimens treated with glass addition only.

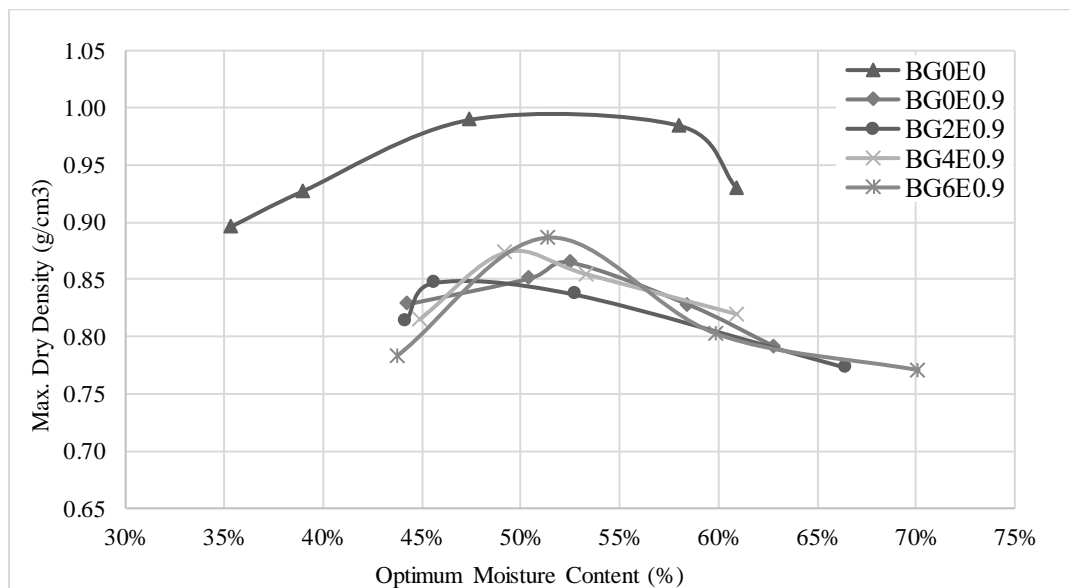


Figure 5.5 Compaction Characters of Both EPS and Glass Powder Treated Samples

5.3. One Dimensional Swell Test Results

The one-dimensional swell test is performed in according to ASTM D4546 [42] and the free swell values of the soil samples are shown in Table 5.3 (see Appendix J for datasheets for one-dimensional swell tests).

Table 5.3 One Dimensional Swell Test Results

| Sample | Free Swell (%) |
|---------|----------------|
| BG0E0 | 30.41 |
| BG2E0 | 31.31 |
| BG4E0 | 25.25 |
| BG6E0 | 24.46 |
| BG0E0.3 | 29.63 |
| BG0E0.9 | 22.08 |
| BG0E2 | 23.89 |
| BG2E0.9 | 24.02 |
| BG4E0.9 | 22.63 |
| BG6E0.9 | 23.63 |

It is reported that both of the additives (EPS beads or glass powder) have a positive effect on the swell potential of the treated soil samples [14, 15, 16, 20, 21, 22]. Therefore, it can be said that these results listed in Table 5.3 are in good agreement with the previous findings.

The test results are evaluated in three different groups to evaluate the effect of the additives individually and together, starting with the effect of only glass powder. Figure 5.6 shows the test results of the free swell of the untreated soil and the glass powder added soil samples at different time intervals. The free swell values decrease with increasing glass powder content. However, for untreated bentonite and bentonite with 2% glass powder addition, the values for free swell are almost the same. It can be concluded that a 2% glass powder addition is not sufficient to improve the swelling potential of the observed sample. On the other hand, with 4% and 6% glass powder addition, the free swell values decrease from 30.41% to 25.25% and 24.46%, respectively. This decrease is also related to the aforementioned decrease in plasticity index with increasing glass powder content. Since glass is a non-cohesive material, it

leads to a decrease in plasticity index values, which plays an important role in reducing the free swell values [20].

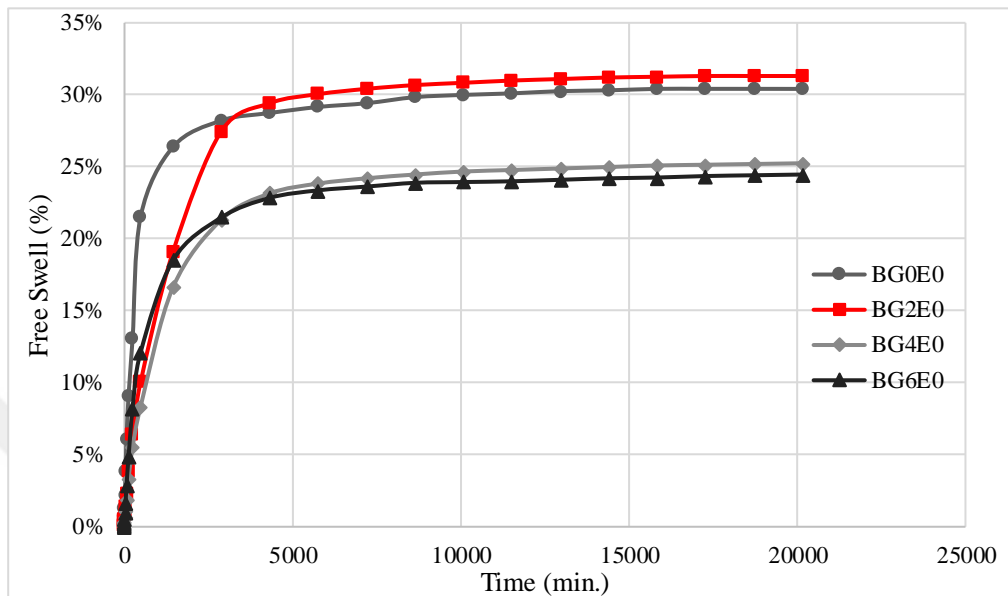


Figure 5.6 Free Swell Values of Only Glass Powder Treated Samples

For the next group, the effects of EPS beads on free swell values are evaluated. The test results are shown in Figure 5.7. While the free swell value of the untreated bentonite is 30.41 in percent, the values for 0.3%, 0.9% and 2.0% EPS addition are 29.63%, 22.08% and 23.89%, respectively. The free swell values decrease up to the addition of 0.9% of EPS beads and since this addition gives the best improvement in free swell values, the EPS proportion in the mixtures is kept constant at 0.9% for the following group. When EPS is used in the mixture, since it is a non-water absorbent and impermeable material, water cannot flow through the EPS beads, resulting in reduced free swell [14, 16]. Moreover, the reason for the increase after 0.9% EPS addition may be explained by the possible segregation caused by the addition of high EPS percentages [14].

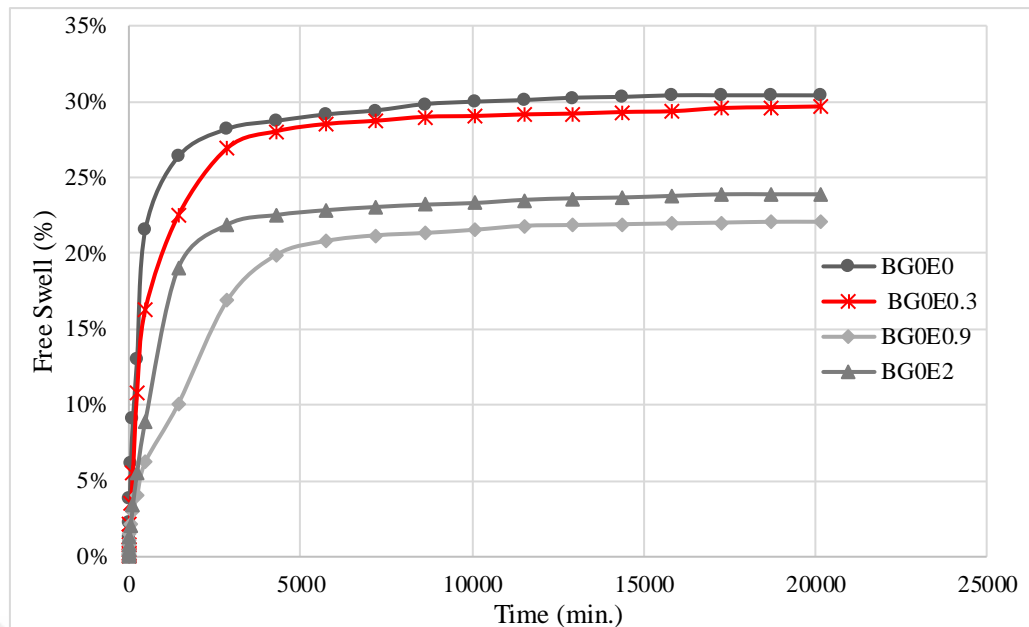


Figure 5.7 Free Swell Values of Only EPS Treated Samples

Finally, for the specimens with EPS and glass powder inclusions, the free swell values can be seen in Figure 5.8. The free swell values of both the EPS and glass powder added samples are lower than those of the untreated bentonite. Since both glass powder and EPS have a positive effect on reducing the free swell values of the observed samples, the reduction is expected to be higher when both additives are used together. On the other hand, the free swell value of BG0E0.9 is 22.08%, whereas, for BG2E0.9, BG4E0.9, and BG6E0.9 the values are equal to 24.02%, 22.63%, and 23.63%, respectively. These values are lower compared to BG0E0, as shown by the results of the one-dimensional swell test. Although, it is seen that the free swell of BG0E0.9 (22.08%) is the lowest one among all prepared mixtures, the free swell value of BG4E0.9 (22.63%) can also be accepted as the same since values are very close to each other. Since the EPS beads lead to a decrease in soil volume, resulting in less soil sample reacting with the glass powder and causing higher free swell values than expected [14]. And this issue can explain the slight increase in free swell with increasing glass powder content (BG6E0.9). Therefore, it can be concluded that the highest improvement of the soil sample containing both additives can be observed when the proportion of the glass powder is 4% of the dry mass of the soil sample (BG4E0.9).

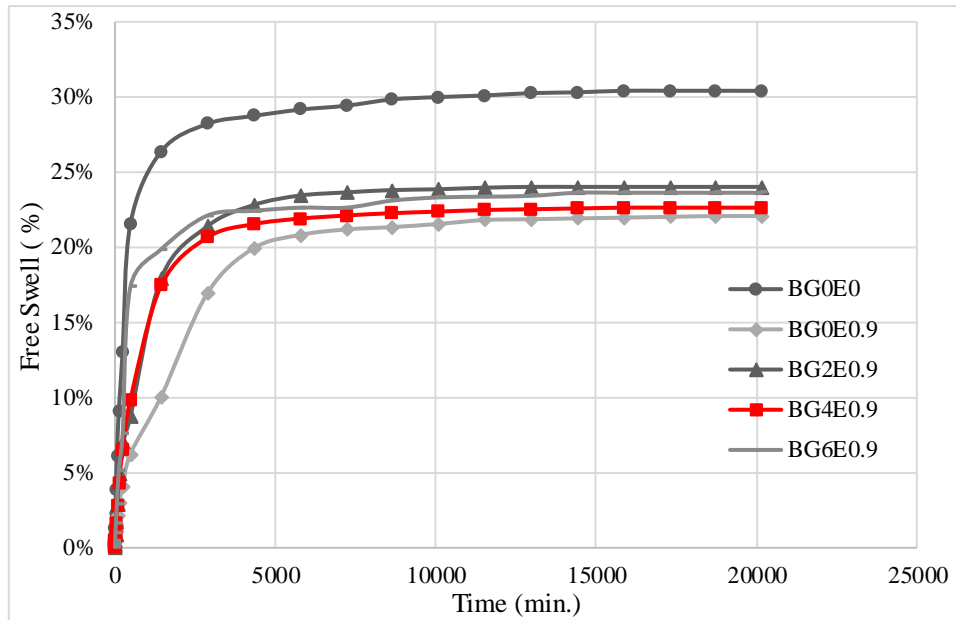


Figure 5.8 Free Swell Values of Both EPS and Glass Powder Treated Samples

5.4. Unconfined Compressive Strength Test

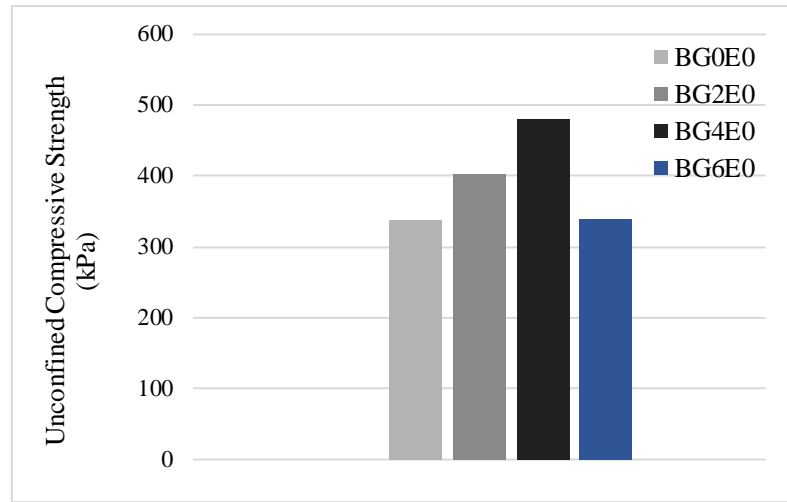
The unconfined compressive strength tests are carried out on the 0-day and 7-day cured specimens according to ASTM D2166 [43]. All the tests are performed two times and the average value is reported as the unconfined strength value, as shown in Table 5.4.

Table 5.4 Unconfined Compressive Strength Test Results

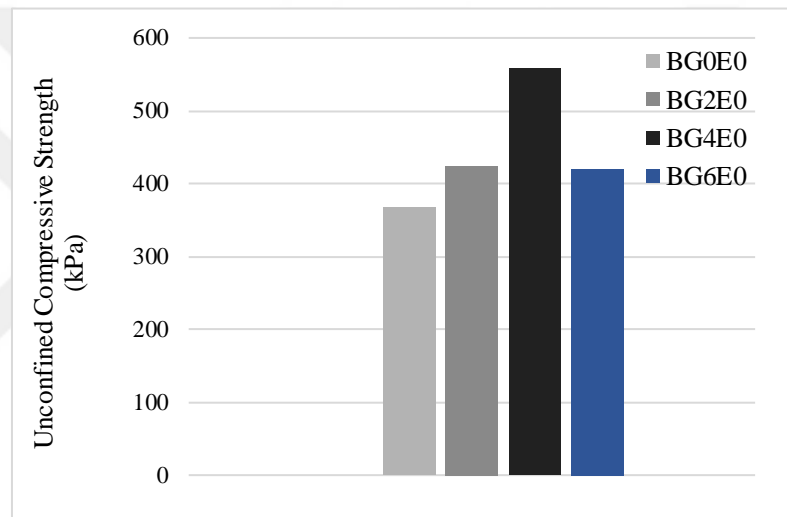
| Sample | Unconfined Compressive Strength (kPa) 0-day | Unconfined Compressive Strength (kPa) 7-days |
|---------|---|--|
| BG0E0 | 337.4 | 367.8 |
| BG2E0 | 403.8 | 423.6 |
| BG4E0 | 461.9 | 558.0 |
| BG6E0 | 340.4 | 420.6 |
| BG0E0.3 | 300.3 | 343.2 |
| BG0E0.9 | 259.6 | 291.2 |
| BG2E0.9 | 323.5 | 361.2 |
| BG4E0.9 | 355.3 | 378.1 |
| BG6E0.9 | 236.7 | 311.0 |

Similar to the previous subsections, the unconfined compressive strength test results are evaluated in three groups, starting with the effect of glass powder alone. As can be seen in Table 5.4 and Figure 5.9, the unconfined compressive strength of the treated bentonite increases with increasing glass powder content for both 0-day and 7-days curing up to a glass powder addition of 4%. The addition of 4% glass powder to the bentonite causes a 37%, and 52% increase in the strength at 0-day and 7-days curing time, respectively. It is also seen that the strength values increase with increasing curing time. The behavior of the mixtures is consistent with earlier research [20, 21, 22] Then, when the glass powder content is increased to 6%, a reduction in the strength value is observed. This decrease can be explained by the reduction in adhesion between the surface of the soil and the glass powder [21].





(a)



(b)

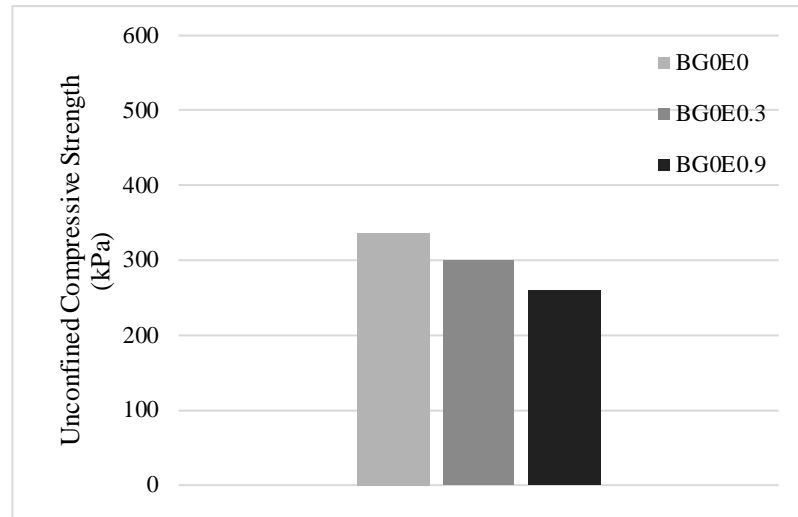
Figure 5.9 (a) Unconfined Compressive Strength for Only Glass Additives (0 day of curing) (b) Unconfined Compressive Strength for Only Glass Additives (7 days of curing)

As described in the studies in the literature [13, 14], the unconfined compressive strength of the studied specimens decreases with increasing EPS content. In this study, the adverse effect of EPS beads on compressive strength is also observed for both 0-day and 7-days curing times, as seen in Table 5.4 and Figure 5.11. The maximum reductions in unconfined compressive strength of 0-day and 7-days cured soil samples observed at the maximum EPS content (0.9%) are 23% and 21%, respectively. Since

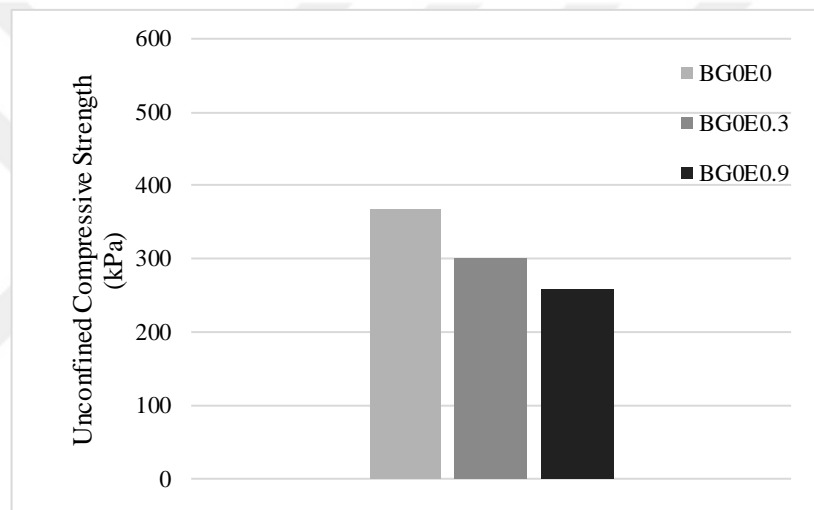
the cohesion between clay-clay particles is greater than the cohesion between clay-EPS particles, this leads to a decrease in the unconfined compressive strength values [13]. Furthermore, the unconfined compression strength of the mixture prepared by using 2% EPS could not be obtained since the addition of a high percentage of EPS beads cause segregation. The extracted samples from this mixture have also been cracked even before the test, as seen in Figure 5.11. Therefore, the results of the unconfined compressive strength test cannot be provided for sample BG0E2 in Table 5.4.



Figure 5.11 Cracks in Sample BG0E2 Due to Segregation



(a)

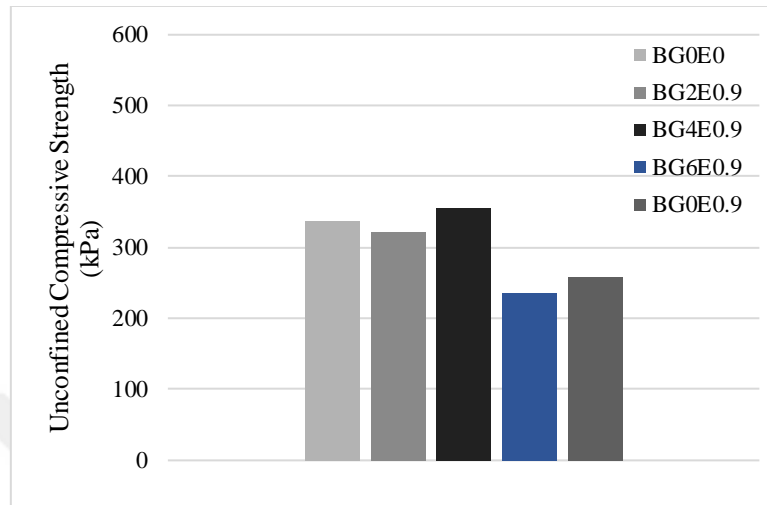


(b)

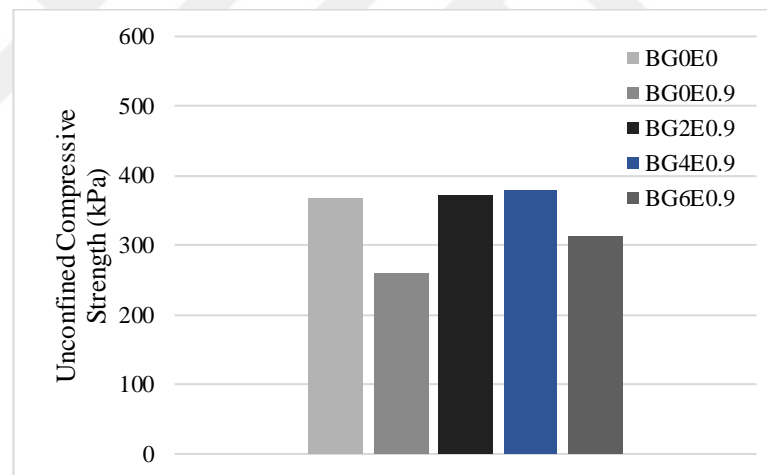
Figure 5.12 (a) Unconfined Compressive Strength for Only EPS Additives (0 day of curing) (b) Unconfined Compressive Strength for Only EPS Additives (7 days of curing)

Finally, considering the effects of the EPS and glass powder admixture by keeping the percentage of EPS beads constant at 0.9% of the dry weight of the soil sample, it is found that the unfavorable effect of the EPS beads on the unconfined compressive strength of the samples is still observed after 0-day and 7-days of curing time. However, the addition of 4% glass powder is sufficient to eliminate the reduction of strength due to EPS inclusion as shown in Table 5.4. With 4% glass powder and 0.9%

EPS addition (BG4E0.9), the unconfined compressive strength values are higher than those with BG0E0 and BG0E0.9 for both of the curing times.



(a)



(b)

Figure 5.13 (a) Unconfined Compressive Strength for Both of the Additives (0 day of curing) (b) Unconfined Compressive Strength for Both of the Additives (7 days of curing)

CHAPTER 6

STATISTICAL ANALYSES AND ANN MODEL

In this study, both regression analysis and artificial neural network model (ANN) are utilized as empirical analysis tools to predict the free swell and unconfined compressive strength of treated clayey soil samples. Detailed information about the methods used and the results obtained are presented below.

6.1. Regression Analyses

Regression analysis can be defined as a practical and effective empirical tool used to establish a relationship between an outcome and a set of variables [45]. In the analyses, the independent variables are called predictors while the dependent variable is named the outcome. In multi regression analysis, several independent variables are used to predict the dependent variable and the analyses can be expressed as follows [46].

$$Y' = A + B_1X_1 + B_2X_2 + \dots + B_nX_n \quad (6.1)$$

Where:

Y' = Outcome variable

A = The intercept

X_s = The predictors

B_s = The coefficients assigned to each predictor

It is critical to obtain an outcome that is very close to the actual value to achieve accurate results. Therefore, the intercept (A) and the predictor coefficients (B_s) should be estimated so that the obtained outcome is very close to the actual one. To validate, if there is a statistically linear relationship between the independent and dependent variables, the Pearson correlation coefficient represented by r should be used [47]. The r range is between -1 to 1, where 0.0 means that there is no certain relationship between the outcome and the predictors. However, there is a dependency between the outcome and the predictors if r is greater than 0.2 [46]. After proving that the outcome and predictors are dependent on each other, another coefficient, denoted by p , can be used to evaluate the level of statistical significance of the variables. If the obtained value for p is less than 0.05, it is assumed that the relationship between the variables is significant. Another point to consider in regression analyses is multicollinearity. Multicollinearity can be evaluated by the variance inflation factor (VIF) which is defined as the state of intercorrelation between predictors. High VIF values ($VIF > 10$) indicate a significant degree of multicollinearity between variables [48].

In this study, the multiple linear regression model is used to define a certain relationship between free swell or unconfined compressive strength of treated samples and the additives, index, and engineering properties of the untreated soil such as liquid limit, plastic limit, plasticity index, free swell, and unconfined compressive strength. To expand the database for the statistical analyses, the results from the previous studies in the literature are included in addition to the results of this study. Two datasets each with 45 and 40 test results are provided for each of the experiments' free swell and unconfined compressive strength, respectively. Out of 45 and 40 data, 80% of the datasets (36 and 32 data) are chosen for training, while the remaining 20% are allocated for testing purposes. Table 6.1 shows the effect of the predictors for each data set.

Table 6.1 Effect of Predictors on Free Swell and Unconfined Compressive Strength

| Dependent Variable | Data Set | Predictor | \bar{X} | S | N | r |
|---------------------------------|----------|--------------------------|-----------|---------|-----|--------|
| Free Swell | 1 | EPS | 0.375 | 0.505 | 36 | 0.081 |
| | | WGP | 3.444 | 7.101 | 36 | -0.361 |
| | | PL | 30.807 | 9.891 | 36 | 0.039 |
| | | LL | 103.176 | 76.55 | 36 | 0.052 |
| | | PI | 72.369 | 70.879 | 36 | 0.050 |
| | 2 | FS _{untreated} | 23.829 | 19.784 | 36 | 0.953 |
| Unconfined Compressive Strength | 1 | EPS | 0.096 | 0.268 | 32 | 0.017 |
| | | WGP | 9.063 | 10.398 | 32 | 0.427 |
| | | PL | 29.463 | 12.256 | 32 | 0.149 |
| | | LL | 85.513 | 65.055 | 32 | 0.142 |
| | | PI | 56.051 | 54.473 | 32 | 0.136 |
| | 2 | UCS _{untreated} | 232.119 | 115.822 | 32 | 0.841 |

Where:

Dependent: Free Swell (FS), Unconfined Compressive Strength (UCS)

\bar{X} = Mean

S = Standard deviation

N = Number of samples

R = Pearson correlation coefficient

EPS = EPS bead content (%)

WGP = Glass powder content (%)

PL = Plastic limit (%)

LL = Liquid limit (%)

PI = Plasticity index (%)

FS_{untreated} = Free swell of untreated soil sample (%)

UCS_{untreated} = Unconfined compressive strength of untreated soil sample (kPa)

As shown in Table 6.1, the free swell and unconfined compressive strength tests performed on the untreated soil samples are crucially important for predicting the free swell or strength of the treated samples with high r values. Then, by performing multiple linear regression analysis, the variance inflation factor (VIF) for the predictors of each data set is evaluated and presented in Table 6.2.

Table 6.2 The Model Summary of Free Swell and Unconfined Compressive Strength

| Dependent Variable | Predictor | R^2 | p | VIF |
|--------------------------------------|----------------------------------|-------|-------|-------|
| Free Swell | EPS, WGP, $FS_{untreated}$ | 0.936 | 0.000 | 1.133 |
| | | | | 1.220 |
| | | | | 1.167 |
| | EPS, WGP, $FS_{untreated}$, PL | 0.942 | 0.000 | 1.192 |
| | | | | 1.228 |
| | | | | 1.176 |
| EPS, WGP, $FS_{untreated}$, PL, LL | 0.942 | 0.000 | 1.201 | |
| | | | 1.266 | |
| | | | 1.179 | |
| Unconfined Compressive Strength | EPS, WGP, $UCS_{untreated}$ | 0.926 | 0.000 | 1.188 |
| | | | | 1.065 |
| | | | | 1.120 |
| | EPS, WGP, $UCS_{untreated}$, PL | 0.931 | 0.000 | 1.487 |
| | | | | 1.076 |
| | | | | 1.150 |
| EPS, WGP, $UCS_{untreated}$, PL, LL | 0.931 | 0.000 | 1.429 | |
| | | | 1.496 | |
| | | | 1.117 | |
| | | | | 1.160 |
| | | | | 5.852 |
| | | | | 4.995 |

Last but not least, the intercept (A) and the predictor coefficients (B_s) are obtained by performing a multiple linear regression analysis and empirical correlations are developed. The prediction equations are shown in Table 6.3. As a result of linear regression analysis, the predicted values for free swell and unconfined compressive strength of the treated specimens can be obtained with high R^2 values indicating strong predictability.

Table 6.3 Prediction Equation of Free Swell and Unconfined Compressive Strength for the Treated Samples

| Dependent Variable | Prediction Equations | Training R^2 | Testing R^2 |
|---------------------------------|--|----------------|---------------|
| Free Swell | $FS = 1.591 - 5.965EPS - 0.180WGP + 0.844FS_{untreated}$ | 0.936 | 0.981 |
| | $FS = -2.751 - 6.616EPS - 0.163WGP + 0.850FS_{untreated} + 0.142PL$ | 0.942 | 0.981 |
| | $FS = -2.661 - 6.65EPS - 0.158WGP + 0.851FS_{untreated} + 0.128PL + 0.003LL$ | 0.942 | 0.980 |
| Unconfined Compression Strength | $UCS = -58.46 - 102.948EPS + 5.889WGP + 1.203UCS_{untreated}$ | 0.926 | 0.928 |
| | $UCS = -84.175 - 124.618EPS + 5.996WGP + 1.187UCS_{untreated} + 1.035PL$ | 0.931 | 0.931 |
| | $UCS = -82.743 - 124.371EPS + 5.982WGP + 1.188UCS_{untreated} + 0.916PL + 0.024LL$ | 0.931 | 0.930 |

In the regression model, it is expected that R^2 values will increase when the number of predictors increases. For the empirical correlations given above, the R^2 values are slightly higher or can be considered as same when the plastic limit and liquid limit values are used as other predictors in addition to the test result of the untreated specimen. It can be concluded that adding the Atterberg limits as additional variables

does not necessarily improve the model performance. The equations based on waste material percentages and test results of untreated samples can be used as empirical correlations.

6.2. Artificial Neural Network (ANN) Model

An artificial neural network (ANN) can be defined as software that can generate new data from what the brain collects by learning, remembering, and generalizing. In other words, ANN is a computing system that imitates the learning process of the human brain [48]. One of the most widely used versions of the ANN model is Artificial Multilayer Perception (MLP), which consists of inputs, multiple hidden layers, and output [49]. To train the system, input and output data are sufficient since the model learns the path automatically. To determine the output of the neuron and the output signal, an activation signal is needed. In this study, the Sigmoid function is used as the activation function. The general form of the function is given as follows.

$$o_j = f(\text{net}_j) = f(i_1w_{1j} + i_2w_{2j} + \dots + i_nw_{nj} + b_j) \quad (6.2)$$

Where:

o_j = the activation functions

net_j = input of the function

i_n = scalar inputs

w_{nj} = weights of neurons

b_j = total bias

As mentioned earlier, the system consists of input, hidden, and output layers. During the process, the inputs are trained by the hidden and output layers and the output is estimated. The weights of the neurons are adjusted via ANN to achieve the lowest possible prediction error. The error is calculated by formulating the root mean square of the errors, as shown in Eq. 6.3.

$$RMSE = \sqrt{\frac{\sum_d \sum_i |t_{ip} - o_{ip}|}{2}} \quad (6.3)$$

Where:

$RMSE$ = root mean square of errors

t_{ip} = real experimental values

o_{ip} = model estimated output

After determining the error for each weight and the biases using Eq. 6.3, the effects of the error on the variables during training are calculated by feed forward and backward processes. Thus, it can be said that the forward and backward process ensures that the weights and biases are updated so that the difference between the real value and the value estimated by the model is minimal. The process can be run numerous times and bounded by an iteration number. The following equations give the formulations used to update the weights and biases.

$$w_{i,j}^l = w_{i,j}^l - \alpha \frac{\partial E(w,b)}{\partial w_{i,j}^l} \quad (6.4)$$

$$b_i^l = b_i^l - \alpha \frac{\partial E(w,b)}{\partial b_i^l} \quad (6.5)$$

In the above equation:

$w_{i,j}$ = weights

b_i = biases

α = learning rate

$E(w, b)$ = error value

To minimize the errors and provide an improvement in the network accuracy, the Levenberg-Marquardt (LM) learning algorithm is used. Each dataset is normalized by using the maximum and minimum input values as presented in below equation. In Eqn. 6.6, the constants of χ is taken as 0.1, whereas β equals to 0.8.

$$x_{in} = \chi + \beta \frac{x_i - x_{min_i}}{x_{max_i} - x_{min_i}} \quad (6.6)$$

Where:

x_{in} = ith input data

x_{min} = minimum input value

x_{max} = maximum input value

In the last step of the analysis, the coefficient correlations, mean square errors, and mean absolute percentage error is determined. These three counted parameters play an important role in the performance of the model and can be calculated using the following equations (Eqs. 6.7, 6.8, and 6.9).

$$R^2 = 1 - \left(\frac{\sum_i |t_i - o_i|^2}{\sum_i (o_i)^2} \right) \quad (6.7)$$

$$MAPE = \frac{100}{p} \sum_i \left| \frac{t_i - o_i}{o_i} \right| \quad (6.8)$$

$$MSE = \frac{1}{p} \sum |t_i - o_i|^2 \quad (6.9)$$

Here:

R^2 = coefficient of correlation

$MAPE$ = mean absolute percentage error

MSE = mean squared errors

t_i = target experimental value

o_i = model output

p =available data in dataset,

After understanding the basic working principle of the ANN model, the previously developed ANN algorithm [50] is used in the study instead of using a commercial ANN tool. Similar to linear regression analysis, two datasets consist of 45 and 40 data provided from the previous studies as well as the results of this study are used to predict and validate free swell and unconfined compressive strength, respectively. For both data sets, 80% of the data is used for training while the rest is used for testing purposes. Three alternative combinations of inputs are used to estimate the free swell and unconfined compressive strength of the treated soils:

- Three inputs including WGP (waste glass powder percentage), EPS (waste EPS beads percentage), and test results performed on untreated samples ($FS_{untreated}$ or $UCS_{untreated}$).

- Five inputs including *WGP* (waste glass powder percentage), *EPS* (waste EPS bead percentage), *LL* (liquid limit), *PL* (plastic limit), and *PI* (plasticity index) of untreated samples.
- Six inputs including *WGP* (waste glass powder percentage), *EPS* (waste EPS bead percentage), *LL* (liquid limit), *PL* (plastic limit), *PI* (plasticity index), and test results performed on untreated samples ($FS_{untreated}$ or $UCS_{untreated}$):

By performing numerous analyses, the values for the learning rate, hidden layers, and the number of neurons in each layer are determined by considering R^2 , MSE , and $MAPE$ values. Finally, the learning rate and hidden layers are determined as 0.2, and 3, respectively with 5, 10, and 5 neurons. Tables 6.4-6.6 show the results for three, five, and six inputs by comparing them with the training and testing values.

Table 6.4 Statistical Data Obtained Using for Training and Testing Process with Three-Input Parameters

| ANN (Three Inputs) | | | | | | | | |
|---------------------------------|-----------------------------|--------------|----------|------|-------|---------|-------|-------|
| Output | Input | Iteration No | Training | | | Testing | | |
| | | | MSE | MAPE | R^2 | MSE | MAPE | R^2 |
| Free Swell | WGP, EPS, $FS_{untreated}$ | 50000 | 0.29 | 1.77 | 0.984 | 5.49 | 26.90 | 0.998 |
| Unconfined Compressive Strength | WGP, EPS, $UCS_{untreated}$ | 50000 | 985.37 | 9.30 | 0.965 | 1015.5 | 17.61 | 0.930 |

Table 6.5 Statistical Data Obtained Using for Training and Testing Process with Five-Input Parameters

| ANN (Five Inputs) | | | | | | | | |
|---------------------------------|----------------------|--------------|----------|------|-------|---------|-------|-------|
| Output | Input | Iteration No | Training | | | Testing | | |
| | | | MSE | MAPE | R^2 | MSE | MAPE | R^2 |
| Free Swell | WGP, EPS, LL, PL, PI | 50000 | 2.88 | 5.58 | 0.968 | 10.00 | 44.44 | 0.978 |
| Unconfined Compressive Strength | WGP, EPS, LL, PL, PI | 50000 | 496.98 | 6.61 | 0.974 | 686.18 | 8.00 | 0.935 |

Table 6.6 Statistical Data Obtained Using for Training and Testing Process with Six-Input Parameters

| ANN (Six Inputs) | | | | | | | | |
|---------------------------------|--|--------------|----------|------|----------------|---------|-------|----------------|
| Output | Input | Iteration No | Training | | | Testing | | |
| | | | MSE | MAPE | R ² | MSE | MAPE | R ² |
| Free Swell | WGP, EPS, LL, PL, PI, FS _{untreated} | 50000 | 0.88 | 3.08 | 0.995 | 4.54 | 28.05 | 0.995 |
| Unconfined Compressive Strength | WGP, EPS, LL, PL, PI, UCS _{untreated} | 35000 | 453.29 | 6.31 | 0.972 | 711.01 | 13.03 | 0.934 |

Where:

R^2 = Coefficient of correlation

$MAPE$ = Mean absolute percentage error

MSE = Mean squared errors

EPS= EPS beads content (%)

WGP= Glass powder content (%)

PL= Plastic limit (%)

LL= Liquid limit (%)

PI= Plasticity index (%)

FS_{untreated}= Free swell of untreated soil sample (%)

UCS_{untreated}= Unconfined compressive strength of untreated soil sample (kPa)

Figures 6.1-6.12 and Tables 6.4-6.6 show the results of ANN models with three, five, and six input variables for training and testing processes. For the training process, the real and estimated values of FS and UCS are shown in Figures 6.1a-6.6a and 6.1d-

6.6d, respectively. Furthermore, in Figures 6.1b-6.6b the residual plots show that residuals are distributed randomly and unbiasedly. In addition, for all ANN models, a histogram of residuals is generated to assess the variance distribution. Figures 6.1c-6.6c depict these findings. The inaccuracy of estimations for all models is evenly distributed, and all histograms have symmetric bell-shaped patterns, indicating that the normality assumption is correct [51]. When the training R^2 values for models with three, five, and six input variables are evaluated, it is clear that all models ($R^2 \geq 0.965$) are successfully trained (Tables 6.4-6.6). Yet, Figures 6.7-6.12 present a comparison of model estimation for FS and UCS with test results in the form of fitted plot lines for testing procedures. The testing R^2 values for testing models of three, five, and six inputs are 0.998, 0.978, and 0.995 for *FS* and 0.930, 0.935, and 0.934 for *UCS*, respectively. These outcomes are very satisfying.

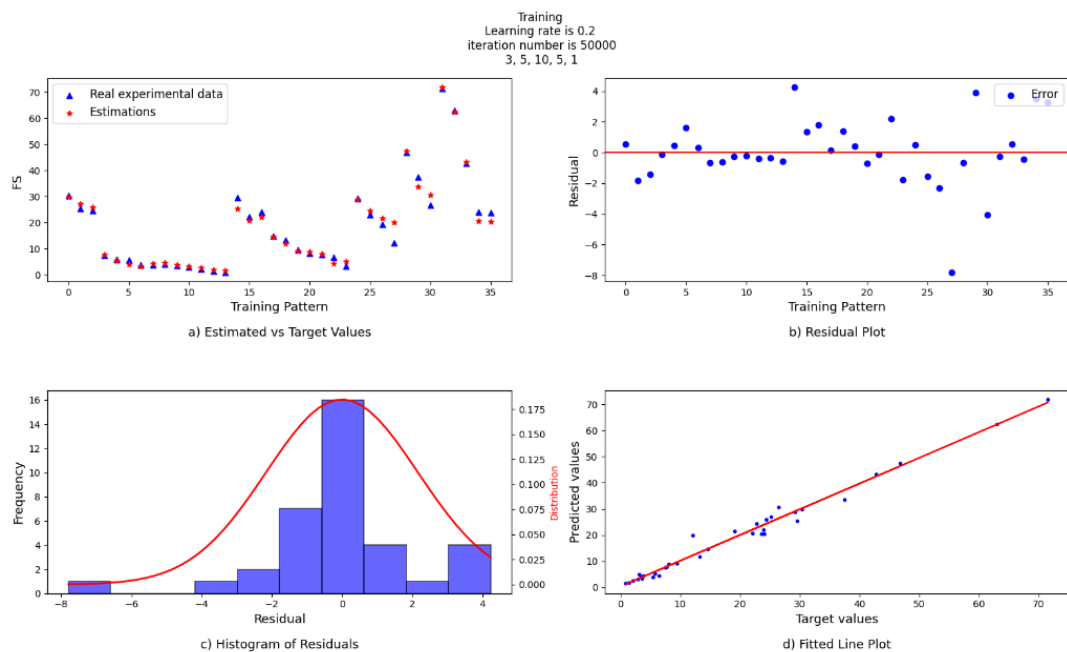


Figure 6.1 Training results for free swell test with three inputs:

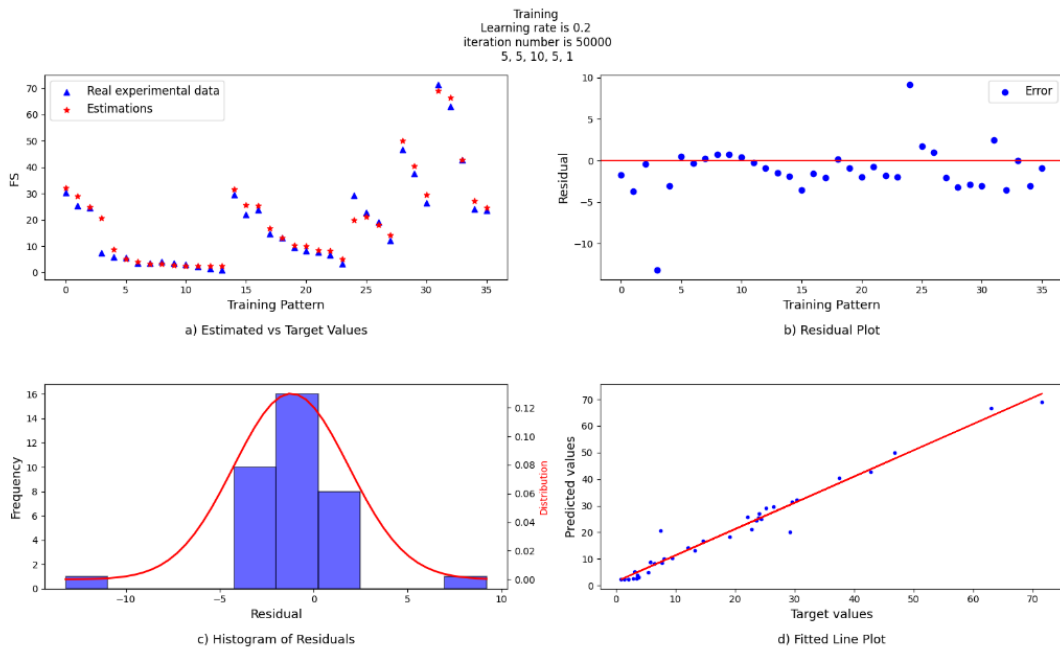


Figure 6.2 Training results for free swell test with five inputs

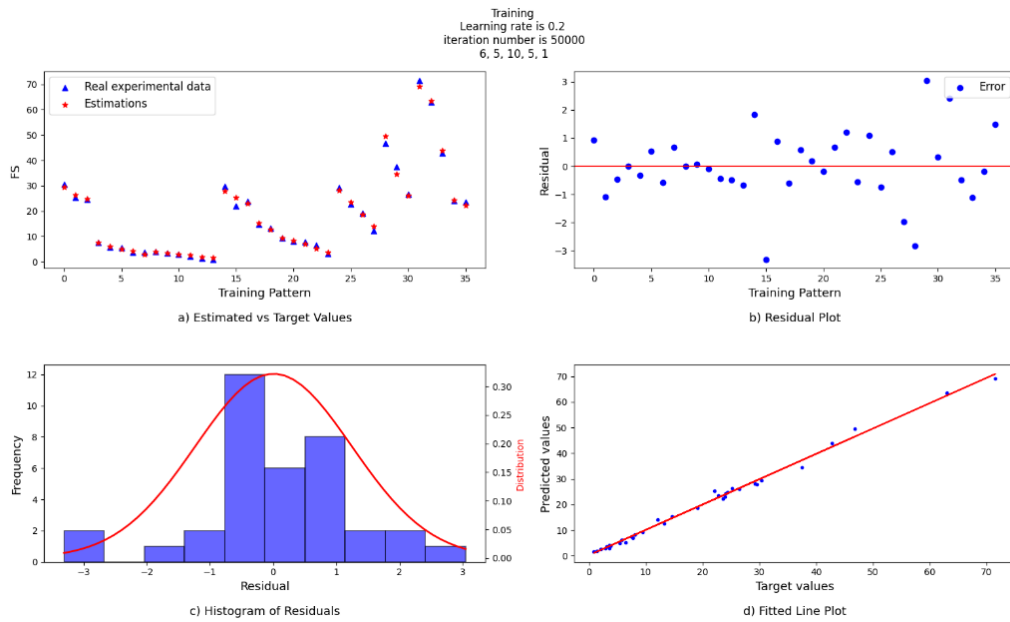


Figure 6.3 Training results for free swell test with six inputs

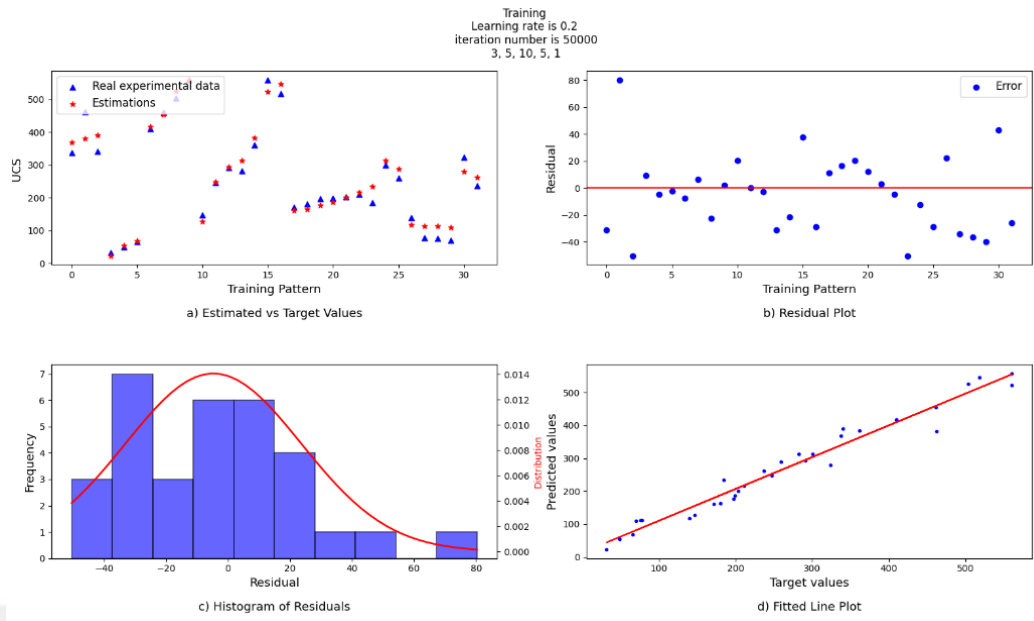


Figure 6.4 Training results for unconfined compressive strength test with three inputs

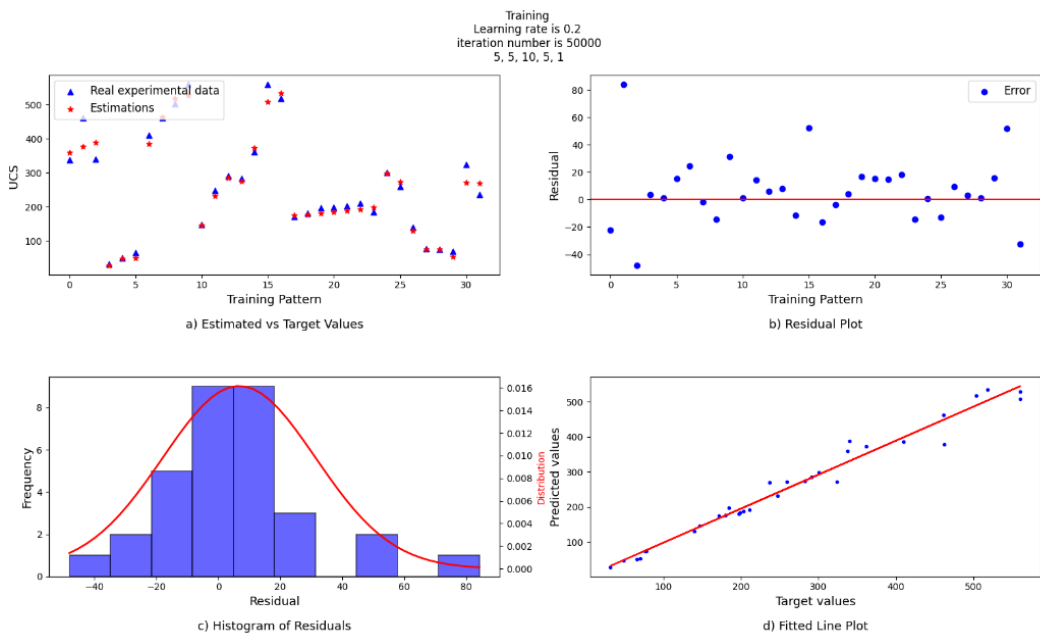


Figure 6.5 Training results for unconfined compressive strength test with five inputs:

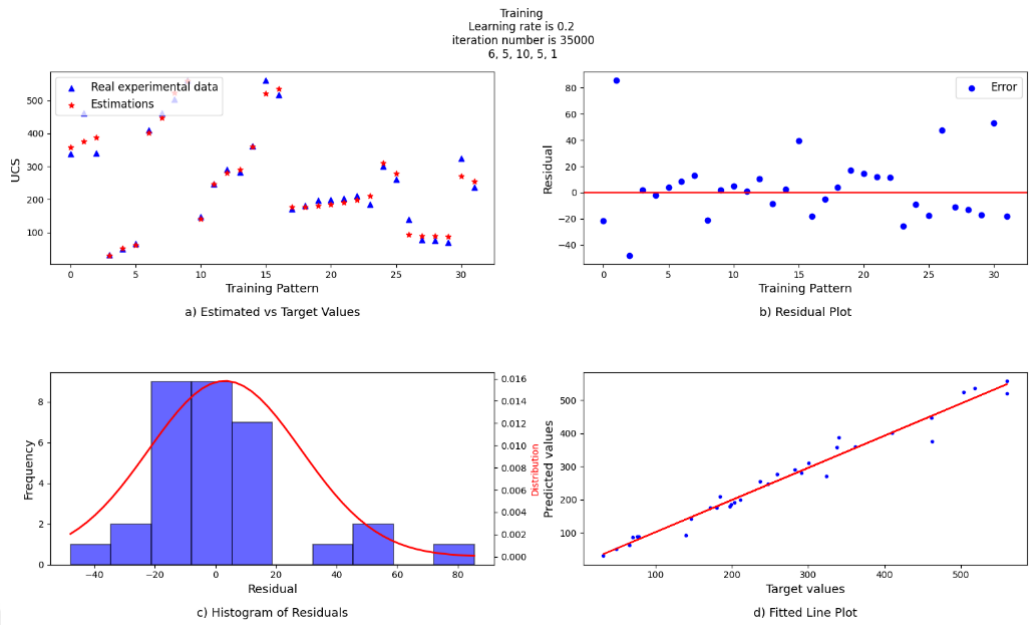


Figure 6.6 Training results for unconfined compressive strength test with six inputs

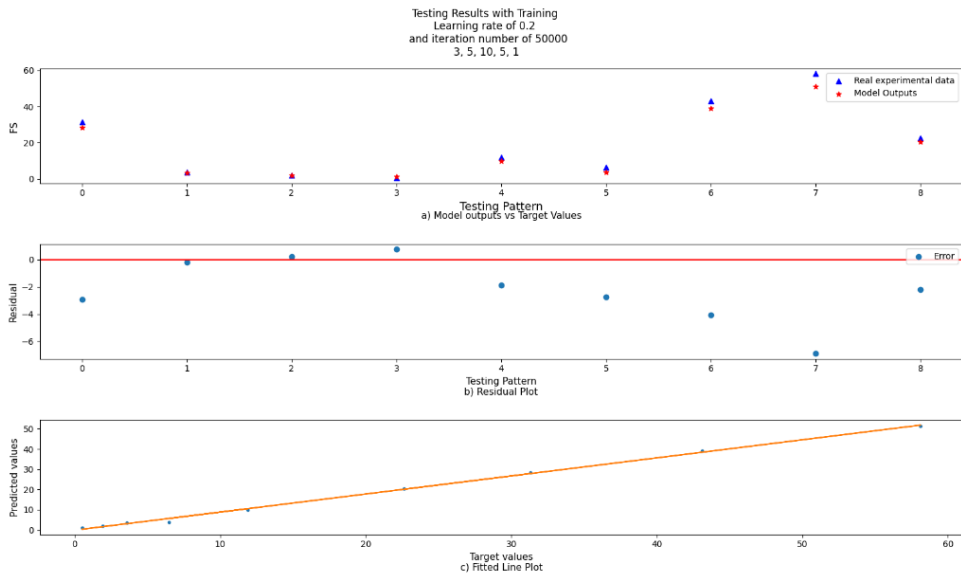


Figure 6.7 Testing results for free swell test with three inputs

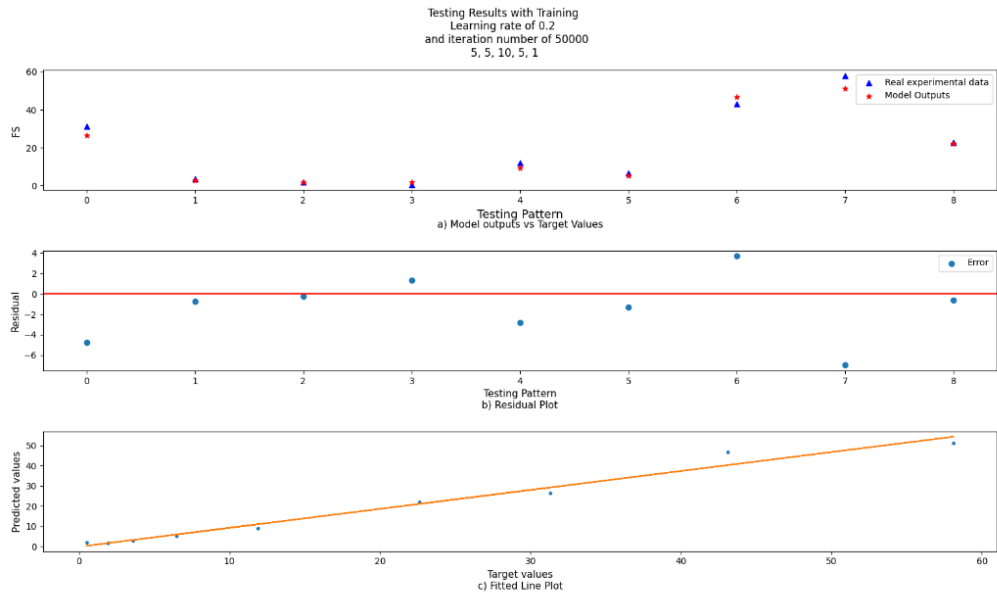


Figure 6.8 Testing results for free swell test with five inputs

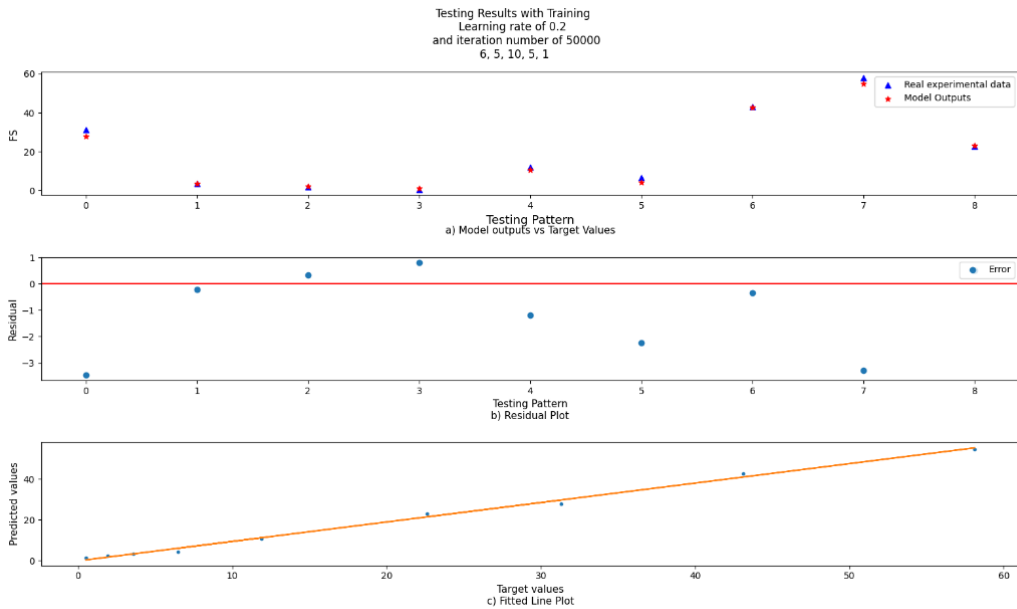


Figure 6.9 Testing results for free swell test with six inputs

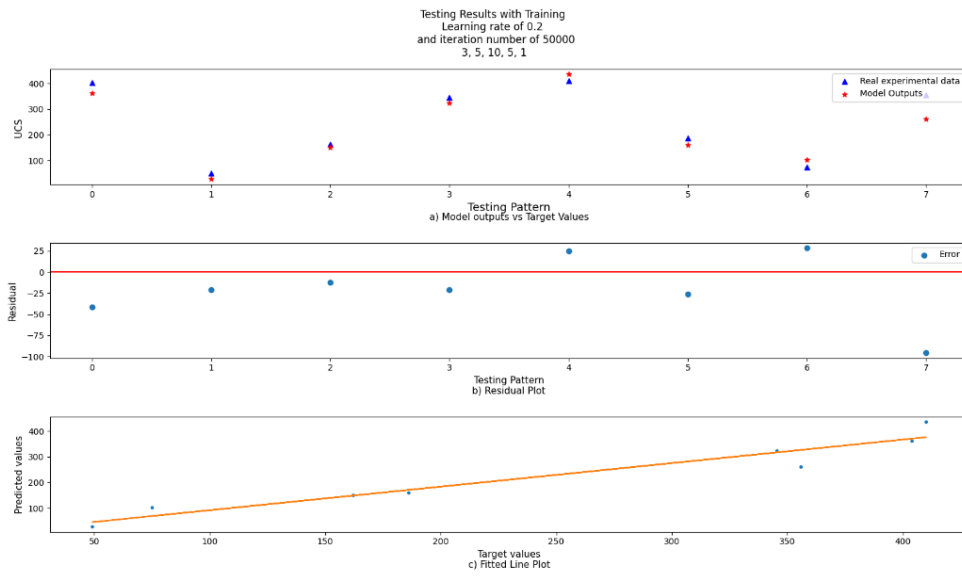


Figure 6.10 Testing results for unconfined compressive strength test with three inputs

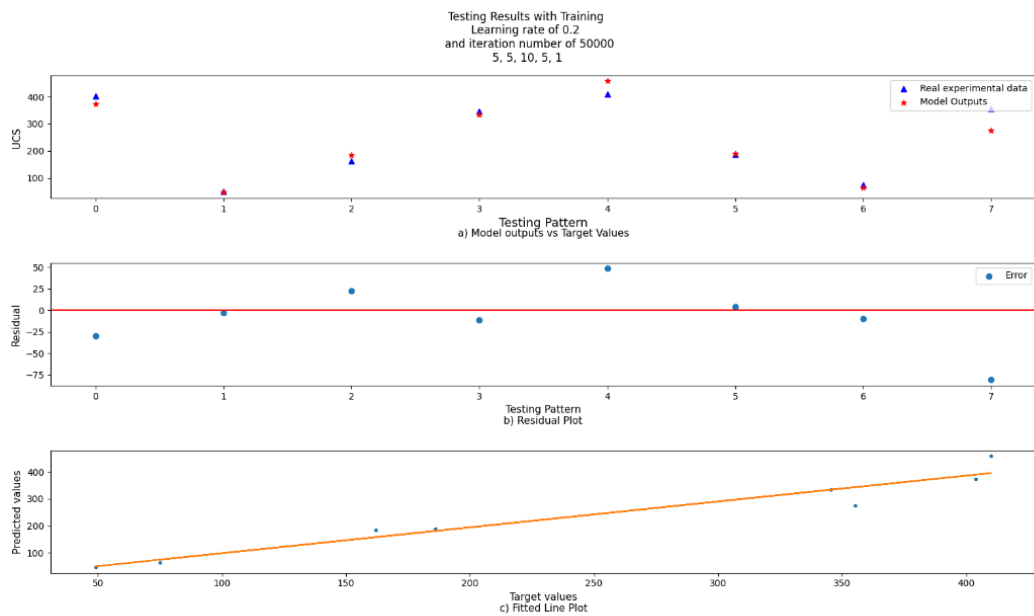


Figure 6.11 Testing results for unconfined compressive strength test with five inputs

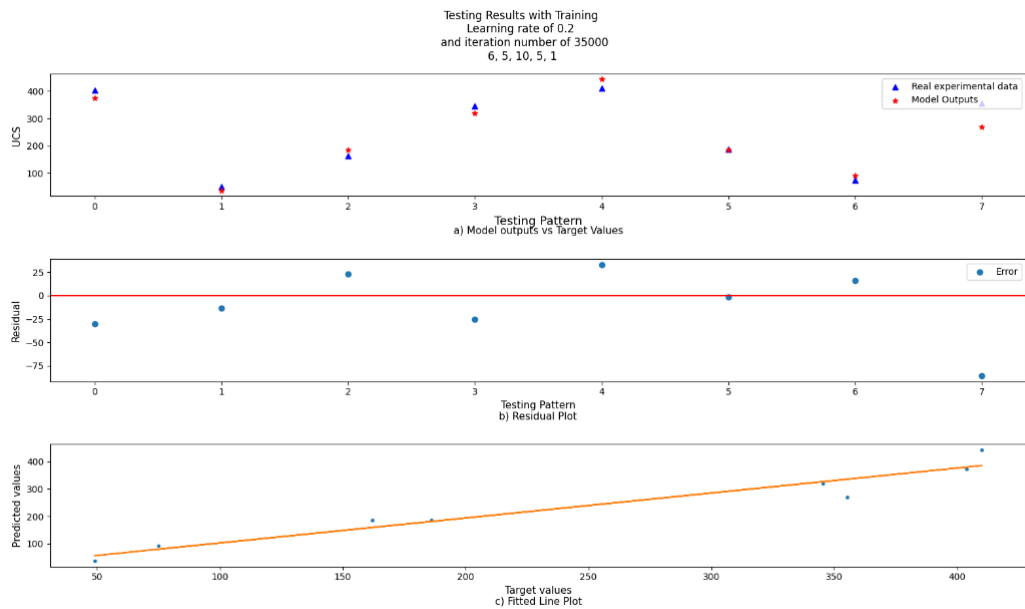


Figure 6.12 Testing results for unconfined compressive strength test with six inputs

6.2.1. K-Fold Cross-Validation

In the ANN model, the analysis results may change depending on variation in the training and testing data sets. For this reason, it is necessary to use the k-fold cross-validation technique to verify that the results of the ANN model are not based on only one particular training and testing data set. In this study, the whole dataset is divided into five equal folds, in other words, a 5-fold cross-validation technique is used. Both training and testing are repeated five times by selecting a different folder from the five folders for testing and using the rest for training [52]. The average results are presented in Table 6.7 for the free swell and unconfined compressive strength values at a learning rate of 0.2, for 3 hidden layers with 5,10, and 5 neurons. In any case, as shown in Table 6.7, the variation of training and testing data does not affect the obtained ANN model results. It can be concluded that the model created is independent of the choice of data sets for training and testing.

Table 6.7 5-Fold Cross-Validation Results

| Output | Input | Iteration No | Training | | | Testing | | |
|---------------------------------|---|--------------|----------|-------|----------------|---------|-------|----------------|
| | | | MSE | MAPE | R ² | MSE | MAPE | R ² |
| Free Swell | WGP, EPS, FS _{untreated} | 50000 | 0.63 | 6.47 | 0.987 | 20.1 | 27.02 | 0.915 |
| | WGP, EPS, LL, PL, PI | 50000 | 1.50 | 7.70 | 0.978 | 20.1 | 31.29 | 0.932 |
| | WGP, EPS, LL, PL, PI, FS _{untreated} | 50000 | 0.72 | 8.65 | 0.996 | 10.6 | 21.45 | 0.957 |
| Unconfined Compressive Strength | WGP, EPS, UCS _{untreated} | 50000 | 172.7 | 4.51 | 0.964 | 2314.2 | 16.70 | 0.941 |
| | WGP, EPS, LL, PL, PI | 50000 | 1550.1 | 11.70 | 0.972 | 2869.2 | 19.28 | 0.901 |
| | GP, EPS, LL, PL, PI, UCS _{untreated} | 50000 | 168.1 | 5.33 | 0.975 | 2999.7 | 18.59 | 0.945 |

Where:

R^2 = Coefficient of correlation

$MAPE$ = Mean absolute percentage error

MSE = Mean squared errors

EPS= EPS beads content (%)

WGP= Glass powder content (%)

PL= Plastic limit (%)

LL= Liquid limit (%)

PI= Plasticity index (%)

FS_{untreated}= Free swell of untreated soil sample (%)

UCS_{untreated}= Unconfined compressive strength of untreated soil sample (kPa)

6.3. Discussion of Results

In this chapter, firstly multilinear regression analyses are carried out and empirical equations based on the test results of this study and those in the literature are established for predicting the free swell and unconfined compressive strength values of treated soil samples. The results show that both the free swell and unconfined compressive strength values can be predicted satisfactorily by using the untreated free swell and unconfined compressive strength values.

The 6.10 and 6.11 can be used to predict the treated values for free swell and unconfined compressive strength.

$$FS = 1.591 - 5.965EPS - 0.180WGP + 0.844FS_{untreated} \quad (6.10)$$

$$UCS = -58.460 - 102.948EPS + 5.889WGP + 1.203UCS_{untreated} \quad (6.11)$$

Where:

FS= Free swell (%)

UCS= Unconfined compressive strength (kPa)

EPS= EPS beads content (%)

WGP= Glass powder content (%)

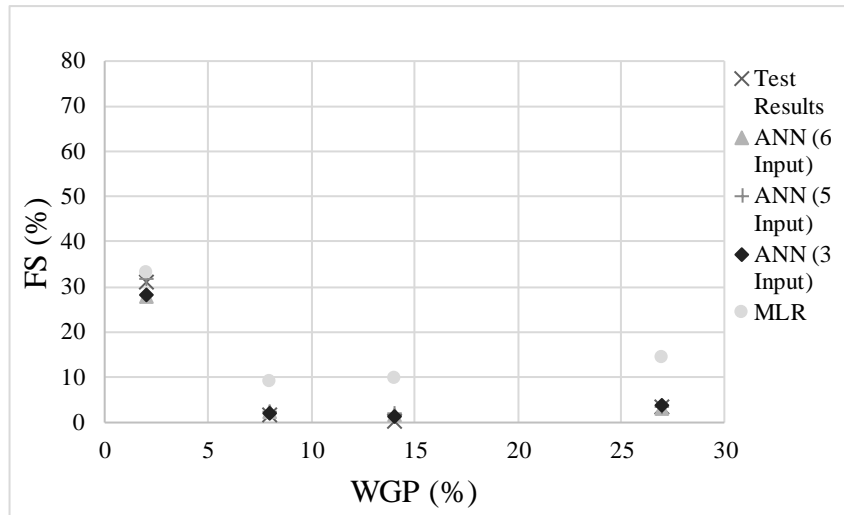
$FS_{untreated}$ = Free swell value of untreated soil (%)

$UCS_{untreated}$ = Unconfined compressive strength of untreated soil (kPa)

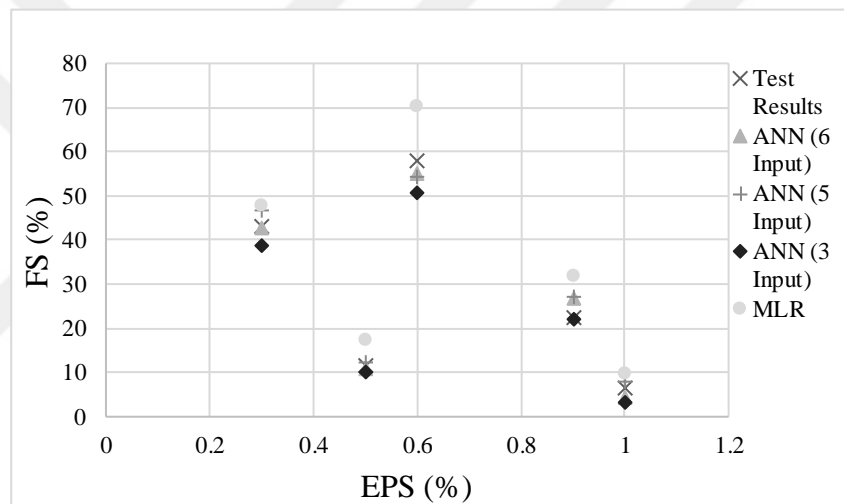
The free swell and unconfined compressive strength of treated soil samples are also estimated using an ANN model that can be employed with or without $FS_{untreated}$ and $UCS_{untreated}$, depending on the data provided. The results reveal that using additives, Atterberg Limits, $FS_{untreated}$, and $UCS_{untreated}$, modestly outperforms linear regression

in the prediction of *FS* and *UCS* with $R^2 = 0.996$ and 0.975 , respectively. Furthermore, utilizing limited input data such as additives and Atterberg Limits, this alternative technique yields similar results in the estimation of *FS* and *UCS* ($R^2 = 0.987$ and 0.964). In order to compare the outcomes generated by MLR and ANN algorithms, similar datasets are used for training and testing processes. The performance of the proposed equations and trained ANN model is evaluated using the testing data. Figures 6.13 and 6.14 demonstrate the outcomes of the prediction equations (Eqs. 6.10 and 6.11) and the ANN approach, as well as empirically obtained *FS* and *UCS* values from this and previous studies.





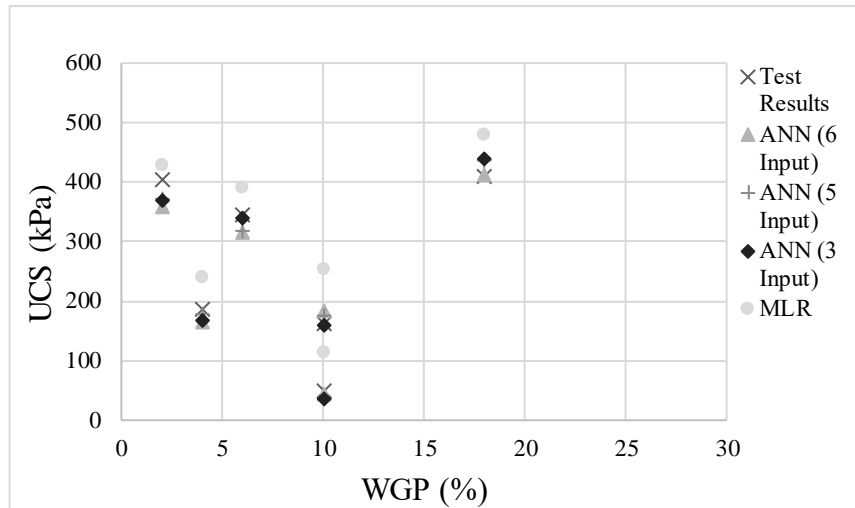
(a)



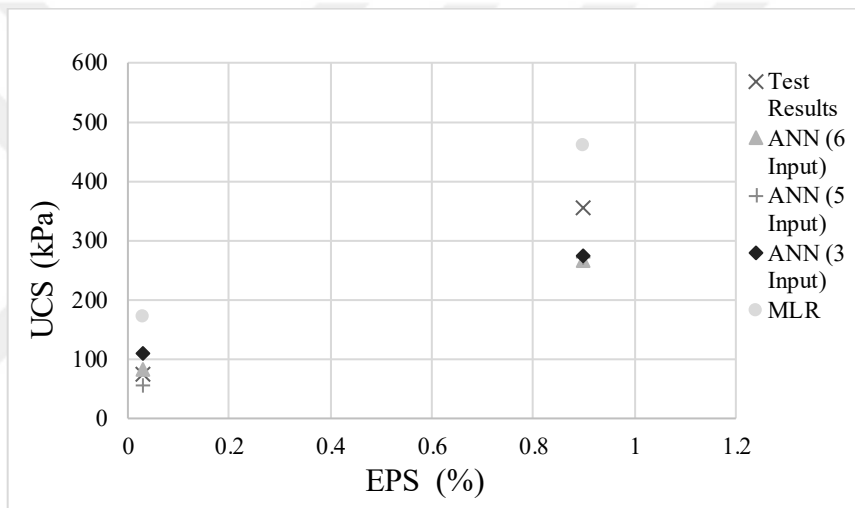
(b)

* For EPS content equals 0.9%, glass powder content equals 4%. Otherwise, there is no glass powder addition.

Figure 6.13. Comparison of the obtained results for free swell (a) for glass powder addition (b) for EPS addition



(a)



(b)

*For EPS content equals 0.9%, glass powder content equals 4%. Otherwise, there is no glass powder addition.

Figure 6.14 Comparison of the obtained results for unconfined compressive strength (a) for glass powder addition (b) for EPS addition

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

In the first part of this study, waste materials: glass powder, and EPS beads are used to improve the engineering properties of highly plastic clay soil. The effect of the additives is studied both together and separately. For WGP and EPS, a typical range of 0 to 6% and 0 to 2% of the dry weight of the soil sample, respectively, is used. The one-dimensional swell test shows that 0.9% EPS additive gives the maximum reduction in free swell values. Therefore, 0.9% EPS is used in combination with various glass powder additives. Atterberg limits, standard proctor tests, one-dimensional swell, and unconfined compressive strength tests are performed and the changes in the index and mechanical properties of the treated soil samples are investigated.

In the second part, to predict free swell and unconfined compressive strength of treated clayey soils, multilinear regression analysis, and artificial neural network model are used as empirical tools. The conclusion of the study can be summarized as follows:

- For glass powder, additive percentages are used as 2%, 4%, and 6% of the dry weight of the soil sample, while EPS inclusions are determined as 0.3%, 0.9%, and 2% of the dry weight of the soil.
- When glass powder is used as an additive, the liquid limit and plastic limit decrease as the glass powder increase, resulting in a lower plasticity index. The liquid limit of the treated soil samples with only EPS first decreases and then increases and remains the same with increasing additive percentage, whereas the plastic limit did not show a trend. If both glass powder and EPS beads are used together, the liquid limit and plastic limit decrease, resulting in a lower plasticity index.

- The OMC of treated samples begins to decrease and then increase and remain constant whereas MDD values increase as the glass powder content increases. Besides, both the optimum moisture content and the maximum dry density decrease with increasing EPS content. When both of the additives are used, similar changes as for the specimens treated with glass addition only are observed.
- The one-dimensional swell test results show that the addition of both glass powder and EPS results in a reduction in free swell values. The test results also show that 4% glass powder addition gives the best improvement in swelling with the combination of 0.9% EPS.
- Unconfined compressive strength tests are performed on 0-day and 7-days cured soil samples. When only EPS is used as an additive, the unconfined compressive strength values also decrease as the EPS content increases. The treatment with only glass powder (up to 4%) causes an increase in unconfined compressive strength values. If the percentage of glass powder increases to 6%, a decrease in strength is observed. When two of the additives are used together, a similar behavior to the addition of solely glass powder is seen. Adding 4% glass powder with 0.9% EPS beads has the highest unconfined compressive strength. Furthermore, higher unconfined compressive strength values are observed for the 7-day cured soil samples.
- MLR and ANN models are used to predict the free swell and unconfined compressive strength of treated soil samples. Two empirical equations based on the percentage of additives and test results of untreated soil samples are predicted by MLR analyses. Additionally, the previously developed ANN model is used to predict the FS and UCS of treated samples by using the same independent variables used in MLR and also with limited input data. If the test results of untreated soil samples are available both of the methods predict the FS and UCS of treated clayey soils satisfactorily. If only limited data (Atterberg limits) of the untreated soil is available, then the ANN model should be used to estimate the FS and UCS of the treated samples.

Recommendations for future studies can be listed as follows:

- By combining the various EPS bead contents with glass powder, the number of data sets can be increased, providing more input for MLR and ANN models.
- The effect of the chemical composition of the glass powder can be studied by using different glass powders, since it affects the pozzolanic behavior.
- The effect of the particle size distribution of the glass powder can be studied by using glass powders with different sizes, since the fineness of the glass affects the pozzolanic behavior of the glass.
- The effect of curing time can also be studied, as it is one of the parameters affecting the unconfined compressive strength, to obtain more data on this subject.

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APPENDICES

APPENDIX A

Datasheets of Some of The Experiments for Basic Properties of Bentonite

Specific Gravity of Bentonite:

| | | | |
|--|------------|--------|--------|
| Sample Name | BG0E0 | | |
| Date | 18.05.2021 | | |
| Sample No. | 1 | 2 | 3 |
| Bottle No. | 4 | 6 | 8 |
| Mass of Bottle (gr.) | 73.43 | 72.01 | 74.44 |
| Mass of Bottle + Dry Soil (gr.) | 103.20 | 102.15 | 104.61 |
| Mass of Bottle + Soil + Water (gr.) | 350.66 | 348.48 | 352.71 |
| Mass of Bottle Filled with Water (gr.) | 332.88 | 330.34 | 334.57 |
| Mass of Water is to Be Used to Fill Empty Volume (gr.) | 247.46 | 246.33 | 248.10 |
| Mass of Dry Soil (gr.) | 29.77 | 30.14 | 30.17 |
| Mass of Water at the Same Volume of Soil Particles (gr.) | 11.99 | 12.00 | 12.03 |
| Specific Gravity | 2.48 | 2.51 | 2.51 |
| Average Specific Gravity | 2.50 | | |

Standard Proctor Test Result of Bentonite:

Dry Density Calculations

| | | | | | |
|--|-------------------|-------------------|-------------------|-------------------|----------------------|
| Sample Name | BG0E0 | | | | |
| Date | 16.11.2021 | | | | |
| Height of Mold (cm) | 11.69 | | | | |
| Diameter of Mold (cm) | 10.12 | | | | |
| Test No | Add % 35 water | Add % 40 water | Add % 50 water | Add % 60 water | Add % 65 water |
| Mass of Mold + Filter Paper (gr.) | 4062.0 | 4062.0 | 4053.0 | 4061.5 | 4053.5 |
| Mass of Mold + Filter Paper + Moist Soil (gr.) | 5202.2 | 5273.5 | 5425.0 | 5524.0 | 5460.0 |
| Volume of Mold (cm ³) | 940.3 | 940.3 | 940.3 | 940.3 | 940.3 |
| Bulk Density (gr/cm ³) | 1.21 | 1.29 | 1.46 | 1.56 | 1.50 |
| Dry Density (gr/cm ³) | 0.896 | 0.927 | 0.990 | 0.984 | 0.929 |

Water Content Calculations

| Sample Name | BG0E0 (continued) | | | | |
|--------------------------------------|---------------------|---------------------|---------------------|---------------------|---------------------|
| Date | 16.11.2021 | | | | |
| Test No | Add %35 water | Add %40 water | Add %50 water | Add %60 water | Add %65 water |
| Container No | 4 | 24 | 28 | 49 | 49 |
| Mass of Container (gr.) | 34.84 | 44.70 | 42.03 | 45.74 | 45.74 |
| Mass of Container + Wet Sample (gr.) | 67.13 | 74.83 | 70.99 | 87.90 | 85.67 |
| Mass of Container + Dry Sample (gr.) | 58.70 | 66.38 | 61.67 | 72.42 | 70.55 |
| Mass of Moisture (gr.) | 8.43 | 8.45 | 9.32 | 15.48 | 15.12 |
| Mass of Dry Soil (gr.) | 23.86 | 21.68 | 19.64 | 26.68 | 24.81 |
| Water Content (%) | 35 | 39 | 47 | 58 | 61 |

Atterberg Limit Test Results of Bentonite:

Liquid Limit:

| Sample Name | BG0E0 | | | | | | | | |
|--------------------------------------|------------|-------|-------|--------|-------|-------|--------|-------|-------|
| Date | 26.05.2021 | | | | | | | | |
| Test Number | Set 1 | | | Set 2 | | | Set 3 | | |
| Penetration (mm) | 28.74 | 29.04 | 27.61 | 24.88 | 25.63 | 24.71 | 20.61 | 20.16 | 20.73 |
| Avg. Penetration (mm) | 28.46 | | | 25.07 | | | 20.50 | | |
| Container No | 65 | | | 28 | | | 30.00 | | |
| Mass of Container (gr.) | 44.45 | | | 42.02 | | | 44.43 | | |
| Mass of Container + Wet Sample (gr.) | 74.12 | | | 74.19 | | | 73.94 | | |
| Mass of Container + Dry Sample (gr.) | 55.10 | | | 53.77 | | | 55.56 | | |
| Mass of Moisture (gr.) | 19.02 | | | 20.42 | | | 18.38 | | |
| Mass of Dry Soil (gr.) | 10.65 | | | 11.75 | | | 11.13 | | |
| Water Content (%) | 178.59 | | | 173.79 | | | 165.14 | | |

| Sample Name | BG0E0 (continued) | | | | | |
|--------------------------------------|-------------------|-------|-------|--------|-------|-------|
| Date | 26.05.2021 | | | | | |
| Test Number | Set 5 | | | Set 6 | | |
| Penetration (mm) | 17.11 | 17.63 | 16.89 | 15.91 | 15.53 | 14.91 |
| Avg. Penetration (mm) | 17.21 | | | 15.45 | | |
| Container No | 24 | | | 73 | | |
| Mass of Container (gr.) | 44.70 | | | 45.86 | | |
| Mass of Container + Wet Sample (gr.) | 76.44 | | | 74.07 | | |
| Mass of Container + Dry Sample (gr.) | 57.11 | | | 57.14 | | |
| Mass of Moisture (gr.) | 19.33 | | | 16.93 | | |
| Mass of Dry Soil (gr.) | 12.41 | | | 11.28 | | |
| Water Content (%) | 155.76 | | | 150.09 | | |



APPENDIX B

Datasheets of Some of The Experiments for Basic Properties of Waste Glass

Specific Gravity of Glass Additive:

| Sample Name | Glass Additive | |
|--|----------------|--------|
| Date | 31.03.2021 | |
| Sample No. | 2 | 3 |
| Bottle No. | 6 | 8 |
| Mass of Bottle (gr.) | 71.62 | 73.43 |
| Mass of Bottle + Dry Soil (gr.) | 107.02 | 109.41 |
| Mass of Bottle + Soil + Water (gr.) | 351.31 | 355.92 |
| Mass of Bottle Filled with Water (gr.) | 330.41 | 334.49 |
| Mass of Water is to Be Used to Fill Empty Volume (gr.) | 244.29 | 246.51 |
| Mass of Dry Soil (gr.) | 24.40 | 34.98 |
| Mass of Water at the Same Volume of Soil Particles (gr.) | 13.50 | 13.55 |
| Specific Gravity | 2.55 | 2.58 |
| Average Specific Gravity | 2.56 | |

Sieve Analysis Test Results of Glass Additive:

| Sample Name | | | Glass Additive | | |
|-------------|-----------------------|---|--|---|----------------------|
| Date | | | 18.05.2021 | | |
| Sieve No. | Sieve Opening (mm) | Mass Retained on Each Sieve (gr.) | Cumulative Retained on Each Sieve (gr.) | Percentage Retained on Each Sieve (%) | Percent Finer (%) |
| 2" | 50.800 | 0.00 | 0.00 | 0.00 | 100.00 |
| 1" | 25.400 | 0.00 | 0.00 | 0.00 | 100.00 |
| ¾" | 19.050 | 0.00 | 0.00 | 0.00 | 100.00 |
| 3/8" | 9.525 | 0.00 | 0.00 | 0.00 | 100.00 |
| No: 4 | 4.750 | 0.00 | 0.00 | 0.00 | 100.00 |
| No: 10 | 2.000 | 0.00 | 0.00 | 0.00 | 100.00 |
| No: 30 | 0.595 | 0.62 | 0.62 | 0.31 | 99.69 |
| No: 40 | 0.420 | 2.06 | 2.68 | 1.35 | 98.65 |
| No: 100 | 0.150 | 52.87 | 55.55 | 28.00 | 73.00 |
| No: 200 | 0.075 | 89.31 | 144.86 | 73.01 | 27.00 |
| pan | - | 53.36 | 198.42 | 100.00 | 0.00 |

APPENDIX C

Liquid Limit Datasheets

Liquid limit datasheets of randomly selected samples are presented as an example.

For BG6E0:

| | | | | | | | | | |
|--------------------------------------|--------|-------|-------|------------|-------|-------|--------|-------|-------|
| Sample Name | | | | BG6E0 | | | | | |
| Date | | | | 09.06.2021 | | | | | |
| Test Number | Set 1 | | | Set 2 | | | Set 3 | | |
| Penetration (mm) | 25.58 | 25.53 | 24.97 | 20.62 | 20.50 | 20.42 | 16.94 | 16.59 | 15.99 |
| Avg. Penetration (mm) | 25.36 | | | 20.51 | | | 16.51 | | |
| Container No | 74 | | | 86 | | | 67 | | |
| Mass of Container (gr.) | 42.87 | | | 45.44 | | | 43.34 | | |
| Mass of Container + Wet Sample (gr.) | 67.26 | | | 67.97 | | | 67.26 | | |
| Mass of Container + Dry Sample (gr.) | 52.25 | | | 54.39 | | | 53.30 | | |
| Mass of Moisture (gr.) | 15.01 | | | 13.58 | | | 13.96 | | |
| Mass of Dry Soil (gr.) | 9.28 | | | 8.95 | | | 9.96 | | |
| Water Content (%) | 160.02 | | | 151.73 | | | 140.16 | | |

| | | | | | | | | | |
|--------------------------------------|--------|--|--|-------------------|--|--|-------|--|--|
| Sample Name | | | | BG6E0 (continued) | | | | | |
| Date | | | | 09.06.2021 | | | | | |
| Test Number | Set 5 | | | | | | | | |
| Penetration (mm) | 13.41 | | | 13.21 | | | 12.92 | | |
| Avg. Penetration (mm) | 13.18 | | | | | | | | |
| Container No | 73 | | | | | | | | |
| Mass of Container (gr.) | 45.85 | | | | | | | | |
| Mass of Container + Wet Sample (gr.) | 71.74 | | | | | | | | |
| Mass of Container + Dry Sample (gr.) | 57.17 | | | | | | | | |
| Mass of Moisture (gr.) | 14.57 | | | | | | | | |
| Mass of Dry Soil (gr.) | 11.32 | | | | | | | | |
| Water Content (%) | 128.71 | | | | | | | | |

For BGE0.3:

| Sample Name | | | | BG0E0.3 | | | | | |
|--------------------------------------|--------|-------|---|------------|-------|-------|--------|-------|---|
| Date | | | | 22.09.2021 | | | | | |
| Test Number | Set 1 | | | Set 2 | | | Set 3 | | |
| Penetration (mm) | 16.55 | 16.20 | - | 19.51 | 18.92 | 19.00 | 24.25 | 24.07 | - |
| Avg. Penetration (mm) | 16.38 | | | 19.14 | | | 24.16 | | |
| Container No | 20 | | | 73 | | | 43 | | |
| Mass of Container (gr.) | 42.67 | | | 45.85 | | | 43.34 | | |
| Mass of Container + Wet Sample (gr.) | 75.42 | | | 76.16 | | | 80.62 | | |
| Mass of Container + Dry Sample (gr.) | 55.99 | | | 57.80 | | | 57.54 | | |
| Mass of Moisture (gr.) | 19.43 | | | 18.36 | | | 23.08 | | |
| Mass of Dry Soil (gr.) | 13.32 | | | 11.95 | | | 14.20 | | |
| Water Content (%) | 145.87 | | | 153.64 | | | 162.54 | | |

For BG2E0.9:

| Sample Name | | | | BG2E0.9 | | | | | |
|--------------------------------------|--------|-------|-------|------------|-------|-------|--------|-------|-------|
| Date | | | | 06.12.2021 | | | | | |
| Test Number | Set 1 | | | Set 2 | | | Set 3 | | |
| Penetration (mm) | 13.12 | 13.80 | 13.59 | 16.08 | 15.92 | 15.73 | 20.93 | 21.29 | 21.84 |
| Avg. Penetration (mm) | 13.50 | | | 15.91 | | | 21.35 | | |
| Container No | 67 | | | 43 | | | 20 | | |
| Mass of Container (gr.) | 43.34 | | | 43.36 | | | 42.69 | | |
| Mass of Container + Wet Sample (gr.) | 69.79 | | | 78.22 | | | 75.85 | | |
| Mass of Container + Dry Sample (gr.) | 54.51 | | | 57.41 | | | 55.42 | | |
| Mass of Moisture (gr.) | 15.28 | | | 20.81 | | | 20.43 | | |
| Mass of Dry Soil (gr.) | 11.17 | | | 14.05 | | | 12.73 | | |
| Water Content (%) | 136.79 | | | 148.11 | | | 160.49 | | |

| | | | |
|--------------------------------------|------------|-------|-------|
| Sample Name | BG2E0.9 | | |
| Date | 06.12.2021 | | |
| Test Number | Set 5 | | |
| Penetration (mm) | 25.71 | 24.85 | 25.43 |
| Avg. Penetration (mm) | 25.33 | | |
| Container No | 60 | | |
| Mass of Container (gr.) | 46.82 | | |
| Mass of Container + Wet Sample (gr.) | 78.86 | | |
| Mass of Container + Dry Sample (gr.) | 58.63 | | |
| Mass of Moisture (gr.) | 20.23 | | |
| Mass of Dry Soil (gr.) | 11.81 | | |
| Water Content (%) | 171.30 | | |



APPENDIX D

Plastic Limit Datasheets

Plastic limit datasheets of randomly selected samples are presented as an example.

BG4E0:

| | | |
|--------------------------------------|------------|-------|
| Sample Name | BG4E0 | |
| Date | 11.12.2021 | |
| Container No | 48 | 57 |
| Mass of Container (gr.) | 45.73 | 43.30 |
| Mass of Container + Wet Sample (gr.) | 59.7 | 56.12 |
| Mass of Container + Dry Sample (gr.) | 55.43 | 52.15 |
| Mass of Moisture (gr.) | 4.27 | 3.97 |
| Mass of Dry Soil (gr.) | 9.70 | 8.85 |
| Water Content (%) | 44.02 | 44.86 |
| Plastic Limit (%) | 44.44 | |

BG0E0.3:

| | | |
|--------------------------------------|------------|-------|
| Sample Name | BG0E2 | |
| Date | 11.12.2021 | |
| Container No | 24 | 30 |
| Mass of Container (gr.) | 44.7 | 44.44 |
| Mass of Container + Wet Sample (gr.) | 56.66 | 56.82 |
| Mass of Container + Dry Sample (gr.) | 53.00 | 52.99 |
| Mass of Moisture (gr.) | 3.66 | 3.83 |
| Mass of Dry Soil (gr.) | 8.30 | 8.55 |
| Water Content (%) | 44.10 | 44.80 |
| Plastic Limit (%) | 44.45 | |

BG6E0.9:

| | | |
|--------------------------------------|------------|-------|
| Sample Name | BG6E0.9 | |
| Date | 11.08.2021 | |
| Container No | 57 | 95 |
| Mass of Container (gr.) | 43.31 | 43.08 |
| Mass of Container + Wet Sample (gr.) | 54.85 | 55.46 |
| Mass of Container + Dry Sample (gr.) | 51.34 | 51.72 |
| Mass of Moisture (gr.) | 3.51 | 3.74 |
| Mass of Dry Soil (gr.) | 8.03 | 8.64 |
| Water Content (%) | 43.71 | 43.29 |
| Plastic Limit (%) | 43.50 | |



APPENDIX E

Standard Proctor Test Datasheets

Standard proctor test datasheets of randomly selected samples are presented as an example.

BG0E0.9:

Dry Density Calculations

| | | | | | |
|--|------------------|------------------|------------------|------------------|------------------|
| Sample Name | BG0E0.9 | | | | |
| Date | 16.11.2021 | | | | |
| Height of Mold (cm) | 11.663 | 11.660 | 11.663 | 11.660 | 11.663 |
| Diameter of Mold (cm) | 10.13 | 10.16 | 10.13 | 10.16 | 10.13 |
| Test No | Add 45% water | Add 50% water | Add 55% water | Add 60% water | Add 65% water |
| Mass of Mold + Filter Paper (gr.) | 4063.5 | 4053.5 | 4062.5 | 4053.0 | 4063.5 |
| Mass of Mold + Filter Paper + Moist Soil (gr.) | 5186.0 | 5262.5 | 5301.0 | 5292.5 | 5274.0 |
| Volume of Mold (cm ³) | 1122.5 | 1209.0 | 1238.5 | 1239.5 | 1210.5 |
| Bulk Density (gr/cm ³) | 940.0 | 945.3 | 940.0 | 945.3 | 940.0 |
| Dry Density (gr/cm ³) | 1.19 | 1.28 | 1.32 | 1.31 | 1.29 |

Water Content Calculations

| | | | | | |
|--------------------------------------|---------------------|---------------------|---------------------|---------------------|---------------------|
| Sample Name | BG0E0.9 (continued) | | | | |
| Date | 29.09.2021 | | | | |
| Test No | Add 45% water | Add 50% water | Add 55% water | Add 60% water | Add 65% water |
| Container No | 24 | 43 | 20 | 30 | 57 |
| Mass of Container (gr.) | 44.70 | 43.34 | 42.66 | 44.43 | 43.30 |
| Mass of Container + Wet Sample (gr.) | 82.69 | 72.12 | 82.84 | 84.46 | 82.13 |
| Mass of Container + Dry Sample (gr.) | 71.03 | 62.47 | 69.01 | 69.69 | 67.15 |
| Mass of Moisture (gr.) | 11.66 | 9.65 | 13.83 | 14.77 | 14.98 |
| Mass of Dry Soil (gr.) | 26.33 | 19.13 | 26.35 | 25.26 | 23.85 |
| Water Content (%) | 44 | 50 | 52 | 58 | 63 |

BG4E0.9:

Dry Density Calculations

| | | | | |
|--|------------------|------------------|------------------|------------------|
| Sample Name | BG4E0.9 | | | |
| Date | 20.10.2021 | | | |
| Height of Mold (cm) | 11.663 | 11.660 | 11.660 | 11.663 |
| Diameter of Mold (cm) | 10.13 | 10.13 | 10.16 | 10.16 |
| Test No | Add 45% water | Add 50% water | Add 55% water | Add 65% water |
| Mass of Mold + Filter Paper (gr.) | 4062.1 | 4062.1 | 4062.1 | 4062.1 |
| Mass of Mold + Filter Paper + Moist Soil (gr.) | 5206.5 | 5248.5 | 5295.0 | 5299.0 |
| Volume of Mold (cm ³) | 1144.4 | 1186.4 | 1232.9 | 1236.9 |
| Bulk Density (gr/cm ³) | 940.0 | 939.7 | 940.0 | 939.7 |
| Dry Density (gr/cm ³) | 1.22 | 1.26 | 1.31 | 1.32 |

Water Content Calculations

| | | | | |
|--------------------------------------|---------------------|------------------|------------------|------------------|
| Sample Name | BG4E0.9 (continued) | | | |
| Date | 20.10.2021 | | | |
| Test No | Add 45% water | Add 50% water | Add 55% water | Add 65% water |
| Container No | 43 | 44 | 20 | 74 |
| Mass of Container (gr.) | 43.36 | 34.84 | 42.69 | 42.86 |
| Mass of Container + Wet Sample (gr.) | 80.07 | 71.43 | 78.37 | 83.74 |
| Mass of Container + Dry Sample (gr.) | 68.70 | 59.36 | 65.96 | 68.26 |
| Mass of Moisture (gr.) | 11.37 | 12.07 | 12.41 | 15.48 |
| Mass of Dry Soil (gr.) | 25.34 | 24.52 | 23.27 | 25.40 |
| Water Content (%) | 45 | 49 | 53 | 61 |

APPENDIX F

One-Dimensional Swell Test Datasheets

One dimensional free swell test datasheets of randomly selected samples are presented as an example.

| | Dial Readings (mm) | | | |
|--------------------|--------------------|-------|-------|---------|
| Sample Height (mm) | 19.07 | 19.13 | 19.13 | 19.13 |
| Elapsed Time | BG0E0 | BG4E0 | BG0E2 | BG2E0.9 |
| 0.5 min | 0.01 | 0.01 | 0.01 | 0.01 |
| 1 min. | 0.02 | 0.01 | 0.02 | 0.02 |
| 2 min. | 0.03 | 0.02 | 0.04 | 0.05 |
| 4 min. | 0.05 | 0.04 | 0.06 | 0.06 |
| 8 min. | 0.09 | 0.07 | 0.10 | 0.08 |
| 15 min. | 0.17 | 0.12 | 0.16 | 0.18 |
| 30 min. | 0.32 | 0.20 | 0.25 | 0.32 |
| 1 hr. | 0.73 | 0.36 | 0.40 | 0.56 |
| 2 hr. | 1.31 | 0.64 | 0.65 | 0.98 |
| 4 hr. | 1.78 | 1.06 | 1.06 | 1.46 |
| 8 hr. | 2.48 | 1.60 | 1.71 | 3.32 |
| 24 hr. | 4.31 | 3.18 | 3.64 | 3.80 |
| 48 hr. | 4.80 | 4.08 | 4.19 | 4.23 |
| 72 hr. | 5.13 | 4.43 | 4.31 | 4.29 |
| 96 hr. | 5.41 | 4.56 | 4.37 | 4.33 |
| 120 hr. | 5.50 | 4.63 | 4.41 | 4.33 |
| 144 hr. | 5.55 | 4.68 | 4.44 | 4.42 |
| 168 hr. | 5.58 | 4.72 | 4.46 | 4.46 |
| 192 hr. | 5.61 | 4.74 | 4.50 | 4.47 |
| 216 hr. | 5.63 | 4.76 | 4.52 | 4.48 |
| 240 hr. | 5.67 | 4.78 | 4.53 | 4.52 |
| 264 hr. | 5.69 | 4.80 | 4.55 | 4.52 |
| 288 hr. | 5.69 | 4.81 | 4.57 | 4.52 |
| 312 hr. | 5.69 | 4.82 | 4.57 | 4.52 |
| 336 hr. | 5.69 | 4.83 | 4.57 | 4.52 |

APPENDIX G

Unconfined Compressive Strength Test Results

Results of unconfined compressive strength test of randomly selected sample (BG6E0) is presented as an example for both 0 day and 7 days of curing conditions.

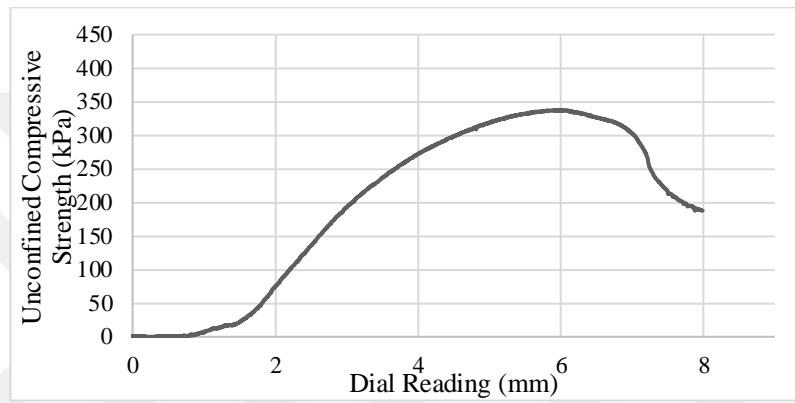


Figure. Unconfined compressive strength test results for BG6E0 (Second sample and 0 day of curing condition)

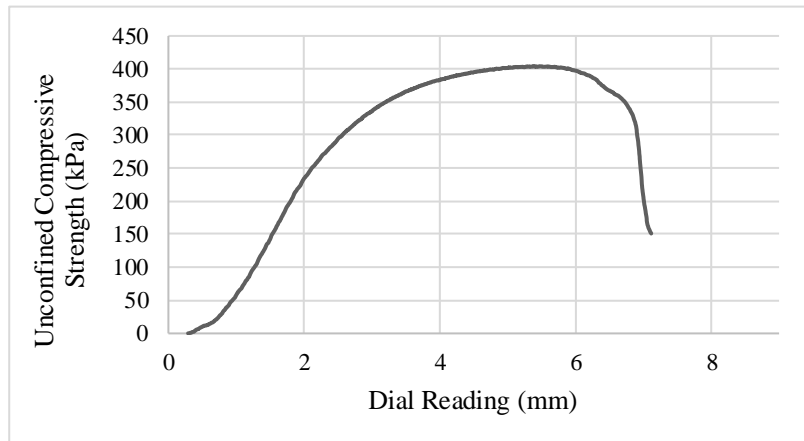


Figure. Unconfined compressive strength test results for BG6E0 (First sample and 7 days of curing condition)