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IZMIR KATIP CELEBI UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

STABILIZATION OF ALLUVIAL HIGHWAY FILLINGS WITH LIME AND THERMAL POWER PLANT FLY ASH

M.Sc. THESIS

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Department of Civil Engineering

Thesis Advisor: Assoc. Prof. Dr. Hasan Fırat PULAT

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To my late mother

FOREWORD

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ABBREVIATIONS

OMC	: Optimum Moisture Content
MDD	: Maximum Dry Density
F.G	: Fine Grain
M.G	: Medium Grain
C.G	: Coarse Grain
HR	: Hour
FA	: Fly Ash
L	: Lime
CBR	: California Bearing Ratio
LL	: Liquid Limit
PL	: Plastic Limit
PI	: Plasticity Index
G_S	: Specific Gravity
VS	: Versus
V	: Volume
W	: Weight
UK	: United Kingdom
USA	: United States of America

STABILIZATION OF ALLUVIAL HIGHWAY FILLINGS WITH LIME AND THERMAL POWER PLANT FLY ASH

ABSTRACT

Alluvial deposits contain a very broad soil range, both in terms of soil type and condition. Alluvial deposits are loose soils that do not fully complete the development of their geology. They, therefore, have a high void ratio, a low bearing capacity, and a high content of organic matter, so, they are considered as problematic soils in terms of civil engineering. The creation of an adequate road network, especially in rural areas, is vital for the socio-economic development of each country. However, the construction, through conventional means and techniques, of a broad network of roads requires heavy financial investment. To meet construction requirements, conventional highway filling design and construction activities require soils with high quality and high bearing capacity. As environmentally friendly and cost-effective process, chamical soil stabilization is used in the construction of soil structures such as roads, canals, dams, and river levees. Chemical soil stabilization is performed by applying by-products or binder to the soil, such as lime, fly ash, for altering the geotechnical efficiency of the soil. Different studies have been performed on soil properties, including compaction, compressibility, bearing capacity, index properties, and strength characteristics.The diversity of outcomes, however, is substantial. Research on the efficacy of fly ash and lime in terms of how quality and quantity can enhance soil characteristics is also limited. Consequently, the objective of this thesis is to perform an analysis, evaluation, and assessment of the role of lime and fly ash in the stabilization of alluvial highway filling. To systematically achieve this goal, firstly, by preparing fine, medium, and coarse grain samples, the grain size effect on engineering properties of alluvial deposits were analyzed. To determine geotechnical index properties; liquid limit, plastic limit, specific gravity, wet sieve analysis, and standard compaction tests were conducted. Compaction properties of alluvial deposits and the effect of fly ash on compaction behavior of alluvial soils were examined. The effect of 2%, 4%, 6%, and 10% lime, and 10%, 15%, and 20% fly ash by weight of dry soil on the bearing capacity of prepared alluvial soil samples were investigated. To examine the curing time effect on CBR of alluvial soils, different curing time as 24, 96, and 168 hours were considered.

It was concluded that the addition of lime and fly ash significantly affects the California Bearing Ratio (CBR) value of alluvial soils. Treating fine and medium grain alluvial soil with 6% lime, and treating coarse grain soil with 4% lime increases the CBR value of these soils. 15% of fly ash increased the CBR value of medium and coarse grain soils significantly. Both particle size and curing time affected the CBR value of alluvial deposits.

Keywords: Alluvial soil, soil stabilization, lime, fly ash, index properties, compaction properties, CBR.

ALÜVYONEL KARAYOLU DOLGULARININ KİREÇ VE TERMİK SANTRAL UÇUCU KÜL İLE İYİLEŞTIRMESİ

ÖZET

Alüvyonal zeminler hem zeminin türü hem de durumu açısından geniş bir zemin yelpazesi oluşturur. Alüvyal zeminler, jeolojik oluşumlarını tam olarak tamamlamayan gevşek topraklardır. Dolayısıyla yüksek boşluk oranına, düşük taşıma kapasitesine ve yüksek organik madde içeriğine sahiptirler. Bu nedenle inşaat mühendisliği açısından sorunlu zeminler olarak kabul edilmektedirler. Yeterli bir yol şebeke oluşturulması, her ülkenin sosyo-ekonomik gelişimi için hayati önem taşımaktadır. Bununla birlikte, geniş bir yol ağının geleneksel yollarla ve tekniklerle inşası ağır mali yatırım gerektirir. Konvansiyonel yol dolgu tasarımı ve yapım faaliyetleri, inşaat gereksinimlerini karşılamak için kaliteli ve yüksek taşıma kapasiteli zeminler gerektirir. Uygun maliyetli ve çevre dostu bir metod olarak yol, baraj, kanal ve nehir yatağı gibi zemin yapılarının inşasında zemin stabilizasyonu kullanılmaktadır. Kimyasal zemin stabilizasyonu, zemini, kireç, uçucu kül gibi bağlayıcı veya yan ürünler uygulanarak yapılır ve böylece zeminin geoteknik özellikleri değiştirilir. Indeks özellikleri, sıkıştırma, sıkıştırılabilirlik, taşıma kapasitesi ve mukavemet özellikleri gibi zemin özellikleri ile ilgili çeşitli çalışmalar yapılmıştır. Sonuçların çeşitliliği de dikkat çekicidir. Uçucu kül ve kirecin, kalite ve miktarın zeminin özelliklerini nasıl artırabileceği etkileri üzerine araştırmalar da sınırlıdır.Bu nedenle, bu araştırmanın amacı, alüvyonlu yol dolgusunun stabilizasyonunda kireç ve uçucu külün rolüne ilişkin bir analiz ve değerlendirme yapmaktır. Bu amaca sistematik olarak ulaşmak için öncelikle ince, orta ve iri daneli numuneler hazırlanarak alüvyon zemin mühendislik özellikleri üzerindeki dane boyutunun etkisi incelenmiştir. Geoteknik indeks özelliklerini belirlemek için; yaş elek analizi, plastik limit, likit limit, özgül ağırlık, standart sıkıştırma testleri yapılmıştır. Alüvyonal zeminin sıkıştırma özellikleri ve uçucu külün alüvyonal zeminlerin sıkıştırma davranışına etkisi incelenmiştir. hazırlanan alüvyonal zemin örneklerinin taşıma kapasitesine % 2, % 4, %6, %10 kireç ve %10, %15, %20 uçucu külün etkisi araştırılmıştır. Kürlenme süresinin alüvyonal zeminlerin CBR üzerindeki etkisini incelemek için 24, 96 ve 168 saat olarak farklı kürlenme süreleri uygulanmıştır. Kireç ve uçucu kül ilavesinin alüvyonal zeminlerin CBR değerini önemli ölçüde etkilediği sonucuna varılmıştır. İnce ve orta daneli alüvyonal zeminlerin % 6 kireç ile işlenmesi ve iri taneli zeminin % 4 kireç ile işlenmesi bu zeminlerin CBR değerini arttırır. % 15 uçucu kül, orta ve iri taneli zeminlerin CBR değerini önemli ölçüde artırmıştır. Alüvyonal zeminlerin CBR değerini hem dane boyutu hem de kürlenme süresi etkilediği ortaya çımıştır.

Anahtar Kelimeler: Alüvyonal zeminler, zemin stabilizasyonu, kireç, uçucu kül, indeks özellikleri, sıkıştırma özellikleri, CBR.

1. INTRODUCTION

Alluvial soils contain a wide soil range, both in terms of soil type and condition. This comes from its variety of the facies at the cross-section and especially along the river's course in the river valley system. With the growing distance from the head of the river, fine material is increasingly transported and deposited by rivers, often containing substantial organic fraction admixtures [1]. Alluvial deposits are loose soils that do not fully complete the development of their geology. They therefore have a high void ratio, a low bearing capacity and a high content of organic matter [2].

The creation of an adequate road network, especially in rural areas, is vital for the socio-economic development of ech country. However, the construction, through conventional means and techniques, of a broad network of roads requires heavy financial investment. Engineers are continually faced with pavement infrastructure maintenance and growth with insufficient financial resources. To meet construction requirements, great quality materials are required for conventional construction activities and pavement design. Quality products are scarce or in short supply in many parts of the world. Because of these limitations, often, engineers are required to use under-standard materials, industrial construction aids, and creative design methods to pursue alternative designs. On poor soil, concrete or asphalt pavement should not be installed, since the pavement can be easily cracked in this situation. Since sub-grade soil is used to move applied loads from the pavement to the layer below, it should have adequate capacity to carry loads. There are many areas around the world where we can find clayey soil. It is very difficult and troublesome for geotechnical engineers to design and create pavement over this porous and expandable form of soil [3].

To enhance the mechanical and chemical characteristics of its engineering efficiency, techniques for soil stabilization have been applied, such as incorporating binders. Furthermore, the use of stabilizing agents in roadwork and sub-grades with weak soil conditions enhance other characteristics, such as cohesion, thus contributing to the stabilization of the structure or embankment. This may potentially contribute to a major decrease in the cost of road construction. For this reason, various additives such as fly ash, lime, or other minerals such as cement, have been utilized. Stabilizing soils with local natural and industrial products is also well known to have a major impact on improving soil properties. In particular, lime and fly ash have been used in several geotechnical structures such as highways, foundation bases, and embankments as an effective additive in soil stabilization. This process is mostly used in the broad spectrum of civil engineering infrastructure such as road woks and pavement woks. In understanding the problems of alluvial soils in highway and road projects, the analysis of the geotechnical behavior of alluvial deposits is important. This research considered the index properties, compaction behavior, and bearing capacity of three classes of alluvial deposits.

1.1 Aims and Scopes of the Thesis

The purpose of this study is to conduct an objective examination into the impact of lime and fly ash on alluvial deposits' bearing capacity and compaction behavior. The mechanical characterization of composites was analyzed from the geotechnical viewpoint. The work has been divided into five main aspects to systematically accomplish this aim. These, to be short, are:

1. To analyze the effects of lime on the bearing capacity of alluvial soil.

   The impact of different lime proportions upon bearing capacity of alluvial deposits was investigated through 45 CBR tests.

2. To analyze the impact of fly ash upon bearing capacity of alluvial soil.

   The effect of different amounts of fly ash on the bearing capacity of alluvial soil was analyzed by means of 18 CBR tests.

3. To investigate the effect of curing age on the load carrying capacity of alluvial deposits.

   The effect of 24, 96, and 168 hours curing age on the load carrying capacity of alluvial soils were investigated through 63 CBR tests.

4. To analyze the particle size effect on the engineering properties of alluvial deposits.Three groups of samples were prepared as coarse, medium, and fine grain.

5. To analyze the index properties of alluvial soil in Çiğli Balatçık region.

For determining geotechnical index properties; plastic limit, liquid limit, specific gravity, wet sieve analysis, and standard compaction tests were conducted.

1.2 Highlight of the study

The study was conducted in 5 different chapters as follows:

1. A basic overview of the research consist of research aim and scope.
2. A description about the backround of soil stabilization such as alluvial soil's problems and characteristics, soil stabilization history, lime and fly ash stabilization.
3. A complete review of research methods and selected materials.
4. A data presentation of geotechnical tests, interpretation and discussion of results.
5. A overall conclusion and suggestion for further investigations.

2. BACKGROUND OF THE STUDY

In engineering projects, the undesirable properties of soil are presently of major concern. Enhancing the characteristics of undesirable soils by stabilization is, in some cases, a fundamental step before construction. To enhance its engineering efficiency, soil stabilization is done by applying a binder to the soil.

Studies have shown that after it has been stabilized, additives contribute to improvements in the effectiveness and mechanical characteristics of soil. Further more, for chemical stabilization, lime or fly ash were used as regional industrial and natural tools in many investigation. The mechanical properties of soil such as strength, compaction, plasticity index, swelling, and compressibility are enhanced by these additives. Lime stabilization is one of the most frequent techniques of chemical stabilization used in order to further enhance the characteristics of the stabilized soil, fly ash as an agent may also be applied. The lime interacts with the water found within the soil in lime stabilization and draws the soil particles to each other.

2.1 Outline of background study

In view of the fact that some fundamental principles must be assumed with respect to the quantity of lime/fly ash, its methods, implementation and appropriate mixtures as an admixture for soil stabilization and some concerns with regard to economic and environmental issues, the aims of this part are:

1. To review alluvial soils and some encountered problems of highway and road construction projects because of insufficient soils.
2. To look at the history and implementation of soil stabilization.
3. To monitor some backstory and major researches on the stabilization of lime.
4. To monitor some backstory and major research on the stabilization of fly ash.

2.2 Introduction

2.2.1 Alluvial deposits

Alluvial deposits form a wide variety of soils, both in terms of type and condition of the soil. Alluvial deposits are loose soils that do not completely complete the development of their geology. They, therefore, have a high void ratio, a low bearing capacity, and a high content of organic matter. The existence of a substantial mixture of organic particles is another typical characteristic of the alluvial soils studied, also to make it possible identify alluvial soils as organic silt as well. Two parameters expressed in total stress are defined as the shear strength of alluvial deposits categorized as silts: angle of internal friction and cohesion. All such parameters are known as random variables and are common random variables, and both moisture content and unit weight affect their variance [1].

In general alluvial deposits are considered as problematic soils in terms of civil engineering. A big complaint for construction works in several countries, such as Australia, the United States, India and also some European countries is the building of construction and roadwork on soft and inadequate soil [4], [5], [14], [6]–[13]. This topic presents significant challenges in finding ways to boost the problematic and insufficient soil used for building projects. For construction works such as railways, airports, runways, highways, bridge foundations, some road works, and the foundation of high-rise buildings, necessary land treatment is carried out [15], [16]. The type and prioritization of soil treatment depend on the building conditions, such as the geotechnical characteristics of soil resources, the accessibility of materials, the economic and environmental aspects, and the risks are foreseen. Various technics have been applied to change soil output to the optimal level for building projects [17], [18]. Several chemical and mechanical approaches have been introduced and performed to improve the properties of certain challenging soil. Some of these technics including of densifying (for example preloading or compacting) treatments, applying a system of pore water pressure (such as electro-osmosis or dewatering), displacement-replacement (such as extracting soft soil and replacing it with solid soil), phase loading and the use of reinforcing elements (such as geotextiles), pile-supported containers, lightweight filling rafts, and deep in situ-chemical stabilization [4], [13], [17], [19]–[21]. Most of the techniques (electro-osmosis or dewatering, displacement-replacement, reinforcing elements) are expensive [10], [13], [15], [16]. However, the most efficient and cost-effective method is soil stabilization. Some scholars say that the soil stabilization results differ. Despite this, the section below will discuss the different studies aimed at evaluating the impact of chemical stabilization on lime and fly ash soil properties.

2.3 Soil stabilization

The shortage of land and economic resources are challenging to be tackled when projects use local lands. The assessment of soil quality in many areas indicates poor soil properties with unwanted engineering characteristics for instance low load capacity, high shrinkage and swell potentials, and high sensitivity to moisture.[19], [20], [22].

A good example is the use of soft soil in a sub-grade layer of pavement, which demonstrates the effects of insufficient land in a building. The pavement is quickly weakened, resulting in premature pavement failure [22]. However, depending on the financial condition of their country, the construction professionals accountable for the paving have to deal with the minimal available funding and the limited availability of sufficient resources. A balance must therefore often be made in terms of consistency and durability. Meanwhile, cost-effective approaches are being developed and stabilization strategies improved. This is why soil stabilization is used for several years as a cost effective form of soil treatment that is friendly to the environment and efficiency. It is a process that can be tailored to meet the needs of particular engineering characteristics [4], [14], [17]–[20], [23].

2.3.1 Mechanical stabilization

Soil stabilization reduces earthworks costs to a minimum and provides a soil management process to maintain, alter, or improve soil efficiency [21]. Two methods of implementing the soil stabilization method are: the mechanical (i.e. by adjusting the gradation) and the chemical (i.e. by changing the chemical combination of the soil) [18]. To achieve high soil quality and gradience relative to their separate components, mechanical stabilization is carried out by combining two or more forms of natural field. By adding or removing each soil element to develop the required mixture, the percentage of fine and coarse mixture particles is altered and establish the appropriate requirements. The right content combination is finally correctly positioned and compacted. The freshly prepared mix of soil should boost the soil strength by regulating internal friction and cohesion, and increase the soil's loading ability as it is becoming a more stable combination [4].

2.3.2 Chemical stabilization

Moreover, the use of stabilizers develops parameters of soil engineering in weak soils used in structures, roadworks, and subgrades of soft soil. Via cohesion, which contributes to structural stabilization, such as an embankment, the strength properties are strengthened. This method lowers construction costs gradually [21]. Some studies have suggested the successful use of various additives for chemical stabilization of soft soils such as cement, fly ash, lime, rice husk, and silica fume [17], [18], [21]. It is also common knowledge that stabilizing soils, especially with lime, has a major impact on improving the soil properties of local natural and industrial resources [9], [14], [17], [19], [24], [25], as well as fly ash; [4], [26]–[28]. The selection of stabilizers depends therefore on the condition and economic factors of the region chosen for construction. For example, most research in the field of chemical stabilization was conducted in countries such as Nigeria with lime compared to other stabilisers due to financial constraints [14].

Additives are mixed with specific humidity content in soil stabilization with lime and fly ash and then added to enhance the land quality of engineering projects.

2.3.2.1 Lime stabilization

Some studies have found that lime has been used for 2000 years and has been used in ancient Mesopotamia and Egypt, Greece, and Rome as a soil-limestone combination [4], [29]. In recent decades, however, lime stabilization has not been commonly used because of some constraining modern geotechnical applications, namely the lack of adequate understanding of the usefulness and protection aspects and limits to natural resources. Despite this, lime stabilization was used in earth building such as roads, dams, routes, airports, hills, foundation bases, and canal linings, all within certain economic limits. In 1924, Lime was used for the first time on modern roads when hydrated lime was applied for the reinforcement of short routes [4], [12], [19], [29]. The quality, simplification, and related economic factors resulted in the lime being used extensively to change the engineering properties of soft soil as a stabiliser [19], [20], [29].

In addition, the effect on the environment was greatly reduced by lime treatment applications for road work and building. This involves reducing the excavation and compaction necessary and is also associated with undesirable visual and auditory facilities and pollution [25].

2.3.2.2 Fly ash stabilization

Throughout history, various methods of fly ash stabilization have been introduced. One of these preferred strategies is chemical stabilization. The potential for natural resources and mineral products to be used for soil stabilization has also been observed in recent years. There seem to be 3 kinds of ash generated during burning coal in the industrial environment, namely bottom ash, fly ash, and pond ash. [30], [31]. One of most plenteous and useful byproducts of waste is fly ash. This industrial gray powder include incombustible, glass-like particles, and from thermal power plants creates residues from the combustion of powdered coal. Flue gasses in furnaces are processed and then gathered by mechanical or electrostatic precipitators or cyclone dividers and filter pouches [8], [13], [28], [31], [32], [33]. The bottom ash is prepared from the ashes contained at the bottom of the furnace. More than two other ashes, the amount of pond ash output that is produced from a mixture of fly ash and bottom ash deposited in an ash pond [31]. The shortage of conventional construction is a crucial problem all over the world. At the same time, major environmental concerns and environmental imbalances are exacerbated by vast quantities of industrial waste material. Furthermore, some unforeseeable ash pond failures may affect farmland and pollute rivers up to 100 kms away, risking human life. [31], [34]. For the application of fly ash, the same environmental issue was noted. Nevertheless, a decade ago, the interpretation of fly ash as a "Polluting Industrial Waste" was altered and revisited as a resource material in building projects [33]. Despite the emergence of such environmental threats such as leaching, dusting and damage to fertile land by fly ash, it has been found to be helpful in engineering buildings [31], [33], [34].

2.3.2.3 Fly ash production

Different current evaluations have suggested that the use of fly ash is lower than the quantity produced, although potential use will increase [33]. Fly ash of about 500 million tons accounts for approximately 75 percent - 80 percent of the world's total production, based on assessments (i.e., around 600 million tons). The annual production of fly ash in the US, for instance, is 75 million tons; China produces over than 100 million tonnes, 112 million tonnes for India, and 10 million tonnes for Australia Figure 2.1 [8], [33], [35].

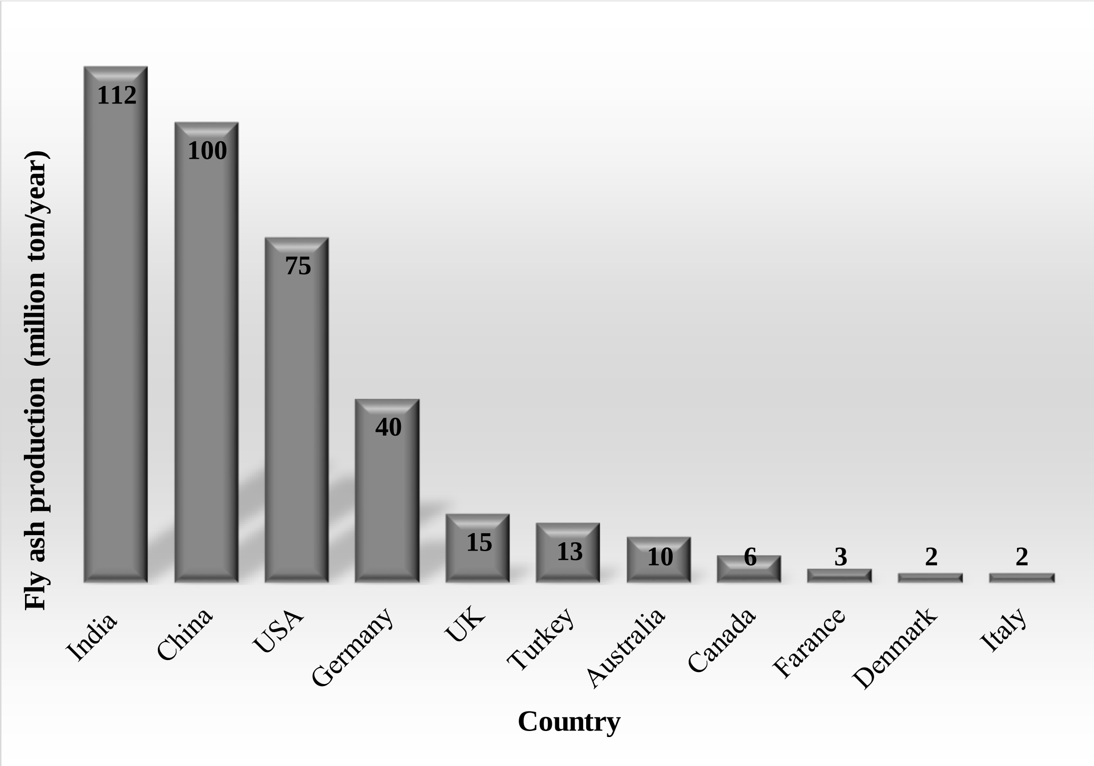

Figure 2. 1 Fly ash production in different countries [35], [36]

2.3.3 Impact of fly ash and lime on various engineering characteristics of alluvial soils

2.3.3.1 Plasticity index

The ability of lime to alter soil plasticity is one of the most visible advantages of lime, which has been stated by several scholars. Some scholars have claimed in this area that the decrease in the plasticity index was caused by an increase in the quantity of lime in the process of chemical stabilization. In certain cases, the findings obtained showed significant decreases in the plasticity index of pure soil after treatment with lime [4], [14], [17]–[20], [25], [29], [37], [38]. If, as a result of lime alteration, plastic limits present a soil's plasticity, there is a width of diffuse double layers of water. They are encircled by clay particles, impacting the soil's plasticity index. Increase in the amount of lime results in reduce in the soil's liquid limit. The plastic limit is then increased by this [29], [39], [40]. Consequently, using the variance among the liquid limit and the plastic limit, the statistical measurement of the index of plasticity of the soil would logically yield a lower result relative to unmodified soil. A reduction in the plasticity index is directly related to the development of a soil that is more friable and workable. Some research into the CH (Fat clay) and CL (Lean clay) groups suggested that the increase in lime that modified both clays led to a decrease in the plasticity index (PI) [19]. In general, there are some clay minerals, including montmorillonite, in high plastic soils that strongly attract water. This activity raises the risk of swelling, thus overlaying the building after it has been destroyed [38], [41].

2.3.3.2 Swelling properties

Furthermore, the soil index of plasticity is directly related to the soil's swelling pressure and swelling potential [42]. Lime can effectively restrict the swelling ability of soils by chemical stabilization. As a direct consequence of a drop in the index of plasticity of the stabilized soil, the swelling pressure will then decrease. Eventually, this feature will greatly reduce the deformation found in construction [12], [14], [19], [20], [24], [25].

In addition to the ability of the lime to alter plasticity and swelling properties, lime alteration can influence the strength of the soil through its cementitious properties [43], [44]. Although study into the lime-stabilized specimens revealed a mode of shear failure such as a brittle material failure [19], [20], [45], further studies [9], [24], [27] confirmed the effective function of lime in the production of soil strength characteristics [29].

2.3.3.3 Compaction characteristics

Many studies tend to show that lime has no enhancing influence on compaction properties. However, lime generates a fast and extensive chemical reaction with soil particles compared to other binders. As a consequence of chemical interaction, the change in soil characteristics leads to a change in soil properties, such as its compaction parameters [14], [19], [20], [24], [25], [38]. Lime stabilization not only induces a substantial enhance in the optimum moisture content relative to unstable soil, but the findings also indicate a decrease in the overall dry density after lime stabilization [14], [19], [20], [25], [29], [46], [47], [48].

Several researchers have performed studies on soil compaction features [8], [13], [26], [49], [50]. They have identified that the addition of fly ash to soils changes the spectrum of soils' void ratio and porosity. Soil particles can absorb higher concentrations of water by soil stabilisation. This relationship directly contributes to a rise in the optimum content of moisture and a decrease in the overall dry density.

2.3.3.4 Consolidation properties

The soil compressibility property is described by the consolidation process as a key element in the construction of highways, airports, structural foundations, and embankments. Consolidation is a process of extruding water particles and voids by the application of loads that are connected to air volume, soil permeability, and pozzolanic action [51], [52]. By filling the soil, atoms are reorganized into a new path to enhance the soil structure's stability [17], [18], [53], [54]. The rate of soil settlement can be restricted by fly ash or lime recovery to mitigate settlement in geotechnical structures. A limited amount of the secondary compression coefficient in the fly ash treated samples was recorded in the results of the experiment. This gain could reduce the likelihood of arbitration due to secondary structural consolidation [27].

2.3.3.5 Strength of soil

To boost soil strength, some scholars post different suggested amounts of lime and this may be linked to the number of clay mineral used in their soil modification tests. The availability of kaolinite, illite and montmorillonite may help to increase the soil bearing value in this context [37]. The researches also found that the mineralogical parameters and the surrounding environment played a key role in soil shrinkage potential and swelling of soils [37], [55].

A subject which has been studied by several researchers is the determination of a proper proportion of lime as an essential factor in soil stabilization. The explanation for the importance of the dosage of the modifier is its capacity to generate the opposite effect of what is desired. Economic aspects are also present. In this area, several various studies have been reported. Dash et al. [29] stated that the use of lime beyond the specific quantity induced a decrease in soil strength, resulting in improper angles of cohesion and friction in the stabilized soil [29]. A variety of lime numbers, between 0.5 percent and 10 percent, were proposed for different studies [37], [42], [55] treatment of soil.

Also, several studies have reported on the effectiveness of fly ash on the strength of soil [7], [19], [28], [56]. The findings obtained suggested that the mixing of fly ash into soil particles contributed to a major increase in the soil's strength characteristics. Therefore, due to enhancement in shear strength and soil cohesion, the bearing ability of soil treated with fly ash can be effectively established.

2.4 An outline of some specified extensive researches

Some of the detailed studies were selected and discussed in this section.

2.4.1 Implementation of Lime-Fly Ash

To further enhance soil quality, more researchers have analyzed the potential modifications that could be made to weak soils by a mixture of lime and fly ash. Negawo et al [57] investigated the lime stabilization of clay soils from Ethiopia. To assess the efficacy of the treatment of lime in order to enhance its mechanical characteristics for road subgrades, highly expansive clay soils from the Highlands of Ethiopia have been studied. Soils treated with quick lime at 5%, 7% and 9% percent dry soil weight and stabilized at a steady temperature of 40 ± 2 C^o for 7 days curing time and geomechanical laboratory tests have been performed, to assess their effects on soil engineering characteristics. Test findings indicate considerable changes in the soil's characteristics following treatment with lime. The addition of lime dramatically decreases the soil's plasticity index and swelling potential. Likewise, the CBR and unconfined compressive strength display remarkable increase, despite the decrease in maximum proctor dry density due to lime stabilization. Based on current analysis, for road subgrade works, expansive soils of the examined region can be effectively stabilized with the addition of 7 percent quick lime by dry soil weight. The significant decrease of the swelling potential is of special interest for the future implementation of a road subgrade for very problematic soil, as the one examined here [57].

Mohammad et al. [58] examined the effect of lime on engineering characteristics of soil including the load bearing capacity of the soil. For 5 soil specimen collected from various regions of Sudan, hydrated lime was used as a stabilizer. There were varying quantities of hydrated lime applied to the specimen. The proportion of hydrated lime was varying between 0.5 percent to 7 percent dry weight of soil sample. The conclusions below were reached from the findings of this study: Increased workability of potentially expansive soils by adding hydrated lime by decreasing their plasticity. Maximum hydrated lime for various tropical soils from Sudan was found to be within 5.5 ~7 % based on test results. The liquid limit and plasticity index drop drastically, while the plastic limit rises as hydrated lime increases. The overall dry density of the soil is known to increase, although with an increase in the hydrated lime content, the optimum water content is reported to reduce. A large rise in the CBR is observed with an increase in the amount of hydrated lime, with a peak increase in the optimum content of hydrated lime. By stabilizing the potentially expansive soils with optimal hydrated lime, quality requirements of common Subgrade and sub-base materials were achieved. [58].

Based on a published study Zha et al.[59], in accordance of lime-fly ash and fly ash stabilization of various soils, the following findings can be identified:

Lime-fly ash stabilization decreases both shrinkage and swelling characteristics of the soil. With rising lime-fly ash amount, the reduction of swelling pressure, free swell, swell potential, and linear shrinkage decreases. In addition, the swelling capacity of stabilized soil and its swelling pressure decrease with rises in the curing time [59].

Investigating the compaction characteristics of soil shows a number of different findings depend on the soil used. Zha et al. examine the influence of lime-fly ash on Hefei expansive soil and found that a rise in the amount of additives contributed to a decrease in both the maximum dry soil density the and optimum moisture content parameters, and this outcome was then verified by other researches [42], [49]. However, more research [60] presented different findings through a systematic analysis of the compaction properties of lateritic soil. The proportion of additives, compaction latency, and the impact of recompaction cycles on soil was studied by Zha et al. [59].

Different dose range of additives (i.e.between 2% to 4% lime dosage) Up to 50% fly ash was used to analyze the impact of lime-fly ash on the soil. Increases in the quantity of additives have resulted in an enhance in the optimal moisture content and a decrease in maximum dry density [61]. The MDD and OMC have a reverse relation in the sense of compaction delay. A higher pause in compaction causes rapid reduction of the MDD, and, on the other hand, rises rapidly. Mainly as a result of the recompaction cycle on lateritic soil treated with lime-fly ash, MDD, and OMC are rising and decreasing, respectively.

2.4.2 Implementation of Fly Ash-Lime

The efficacy of self-cementing fly ashes for stabilisation of soft fine-grained soils was evaluated by Edil et al. [62]. Tests were performed on admixtures for CBR and modulus of resilient (Mr). Various soft fine-grained soils, such as inorganic soils, organic soil, and various fly ashes, have been used. Two of the fly ashes are Class C ashes of good quality and the other ashes are off-specific ashes. Experiments were carried out on soils and mixtures of soil-fly ash prepared at optimum water content in various wetlands. The findings showed that the addition of fly ash improved the CBR and Mr of the inorganic soils dramatically. On the other hand, with increasing fly ash amount, the CBR of soil-fly ash mixtures usually increased and decreased with rising water compaction content. Also, to increase the pavement resistance, fly ash should be stiffened over time. Typically, organic soil had slightly lower CBR and Mr values than inorganic soil. However, there was a further improvement in the resilient module for wetter or more plastic fine-grained soils.

Bose [32], examined the effectiveness of fly ash on expansive soil with different proportions of fly ash varying from 0 percent to 90 percent. To exmine the soil's geotechnical behavior, specific gravity, Atterberg limits, characteristics of compaction, free swell, swelling pressure, swelling potential, percentage of axial shrinkage, and unconfined compressive strength tests were performed. The findings indicate that adding fly ash may decrease the plasticity characteristics of expansive soil. By increasing the fly ash content, the plasticity index and linear shrinkage reduced dramatically and the shrinkage limit was enhanced. The maximum unconfined compressive strength and maximum dry density displayed an incremental trend of up to 20% fly ash, whereas the optimum water content reduced with a rise in the fly ash proportion.

By observing the subgrade characteristics, Prasad and Sharma [63] demonstrated the effect of clay soil blended with sand and fly ash for soil stabilisation. The goal of this research is to determine a solution for properly disposing of fly ash and to provide better subgrade material for the construction of pavements. The findings point out that compaction and CBRs of composites containing clay, sand, and fly ash were substantially improved. After stabilization, the swelling of the clay also decreased. As fly ash was applied, the maximum dry density of the clay-sand-fly ash mixture decreased and the optimum moisture content increased. This stabilized soil can be used in low-traffic areas for the building of flexible pavements.

To increase the bearing capacity of the soil, Prabakar et al. [56] researched the behavior of soils mixed with fly ash. Three different soil types and varying percentages of fly ash were used. This research aimed to exmamine the effectiveness of fly ash-soil admixtures and concentrated on improving the engineering characteristeics of soils with improved load-bearing capacity. This investigation also stated the Cost-efficiency of fly ash for soil improvement and covered the behavior of compaction, settlement, California bearing ratio, parameters of shear strength, and characteristics of swelling. The findings showed that the addition of fly ash decreased the dry density of the soil and unit weight of soil. With enhancing fly ash mount in soils, the void ratios and porosity have changed. By the addition of fly ash, the shear strength of the mixture was increased and the increase was nonlinear. By applying fly ash, the value of cohesion improved and this modification was linear. CBR value of soil has increased by incorporating fly ash. The results show that the angle of internal friction and the shear strength of soil mixed with fly ash caused a good strength. The use of fly ash in the soil also decreased soil swelling. Also, the shear strength, cohesion and bearing capacity were enhanced by the fly ash. This mixture can also be used as the basis material for roads, backfilling, etc.

2.5 Lime/fly ash characterization analysis

2.5.1 Lime properties

Two types of lime are commonly used for soil stabilisation, namely, hydrated and unhydrated [4]. Heating limestone or dolomite (calcium magnesium carbonate) to form calcium oxide (CaO) with varying concentrations of magnesium oxide (MgO) produces unhydrated (unslaked) lime. This can be slaked by steam or water treatment, and calcium hydroxide (Ca(OH)2) or calcium and magnesium hydroxide (Ca(OH)2 + Mg(OH)2) are formed. Normally, calcium oxide hydration is much faster than that of magnesium oxide. Hydrated lime, also referred to as slaked lime, is quicklime to which water has been applied before all the calcium and magnesium oxides have been converted to hydroxides, the thirst of quicklime has slackened by water. Hydrated lime is made from pure calcium oxide and 24 percent chemically combined water. The hydrated lime is powdery and white. Hydrated lime is most often used for soil lime stabilization, it is used for road construction because of its plastic nature and therefore avoids pothole formations.

When lime is mixed with soil, two basic through complex reaction apparently take place, namely:

1. A very quick and often almost immediate improvement that can include the exchange of ions.
2. Pozzolanic reactions occur over a period of time ranging from a few minutes to several months or longer. A chemical reaction between the lime and the soil is present in both cases.

2.5.1.1 Ion Exchange

This pretty fast reaction involves both anions and cations and is followed by flocculation and the creation of agglomerations caused by clay particles having to adhere to one another, This raises the plastic limit and therefore the Plasticity Index (PI) is decreased, while the liquid limit can remain unchanged, decrease or increase. But the material becomes more satisfactory and typically increases in strength. To prevent salt damage, lime is often applied to acidic, sulfate-contaminated crushed rock. While clay minerals are not involved, this can be regarded as a form of ion exchange.

2.5.1.2 Puzzolanic reaction

The PH is increased to around 12.4, which is the PH of saturated lime water at $25C^{o}$ if adequate lime is applied to the soil. Hydrated calcium silicate and aluminate gels similar to those found in hydrated Portland cement are formed in these high PH reactions between lime and clay minerals and other pozzolans, such as amorphous silica to create cementations. The strength produced is primarily responsible for the crystallization and hardening of these gels.

2.5.2 Fly ash properties

Depending on the type of coal that has been burned, the fly ash is split into class-C and class-F [13], [26], [28]. Class C fly ash is usually formed by sub-bituminous or lignite pulverized burning. Although Class-F fly ash is produced from a combustion heater by bituminous or anthracite coals [8], [13], [28].

Overall, fly ash is made up mostly of silicon, aluminum, iron, and calcium oxides. They also detect magnesium, potassium, sodium, titanium, and, rarely, sulfur. In addition to quartz, tricalcium aluminate, and more than 20 percent CaO, Class-C fly ash usually contains calcium alumina sulfate glass. Fly ash is classified on the basis of its CaO content by the American Society for Testing and Materials specification (ASTM-C618).

Table 2. 1 Chemical composition of class F and class C fly ash [4].

Chemical composition	Fly ash class	
	F	C
Silicon dioxide (SiO_2) + aluminium oxide (Al_2O_3) + iron oxide (Fe_2O_3), min, %	70	50
Sulphur trioxide (SO_3), max, %	5	5
Moisture content, max, %	3	3
Loss on ignition*, max, %	6	6

3. MATERIAL AND METHOD

3.1 Material

3.1.1 Alluvial deposits

Alluvial soils that are used in this study were taken from Çiğli – Balatçık region (İzmir, Turkey). The samples were taken from a construction site where is located at İzmir Katip Celebi University Figure 3.1. The total exploration depth was 30 m and for each 1.50 m, alluvial soil samples were collected from boreholes. The samples were immediately coated with nylon to protect their natural properties and they were transferred to the soil mechanics laboratory and kept in airtight boxes.

Figure 3. 1 location map of Izmir Katip Çelebi University [2]

To investigate the engineering properties and the effect of particle size on the bearing capacity of alluvial soils, three groups of samples are prepared. Figure 3.2 shows different grain size alluvial soil samples. Coarse-grained particle size rages between 2.00mm-4.75mm, medium-grained particle size varies between 0.425mm-2.00mm, and fine particle size is below 0.425mm.

Figure 3. 2 Fine, medium, and coarse grain alluvial soil samples

samples are prepared from these three groups of particle size in different combinations. Table 3.1 and Table 3.2 show the particle size ranges and proportions of the prepared samples respectively.

Table 3. 1 Particle size ranges for F.G, M.G, and C.G soils

Group number	Definition	Particle size range
1st group	Coarse grained	2.00 mm < C.G < 4.75 mm
2nd group	Medium grained	0.425 mm < M.G < 2.00 mm
3rd group	Fine grained	F.G < 0.425MM

Table 3. 2 Proportion of F.G, M.G and C.G samples

Coarse sample	Medium sample	Fine sample
60% C.G	35% C.G	10% C.G
30% M.G	30% M.G	30% M.G
10% F.G	35% F.G	60% F.G

3.1.2 Lime

Lime is a binder substance based on an inorganic basis obtained by evaporating the carbon dioxide in it as a result of heating limestone (calcium carbonate) at different degrees (850-1450 ° C). In this research, the slaked lime used was received from İzmir Kâtip Çelebi University, Civil Engineering Department, Construction Materials Laboratory. The maximum grain size of the lime is 0.425 mm. Figure 3.3 shows the used slaked powdered lime. The chemical characteristics of the lime used are shown in Table 3.3.

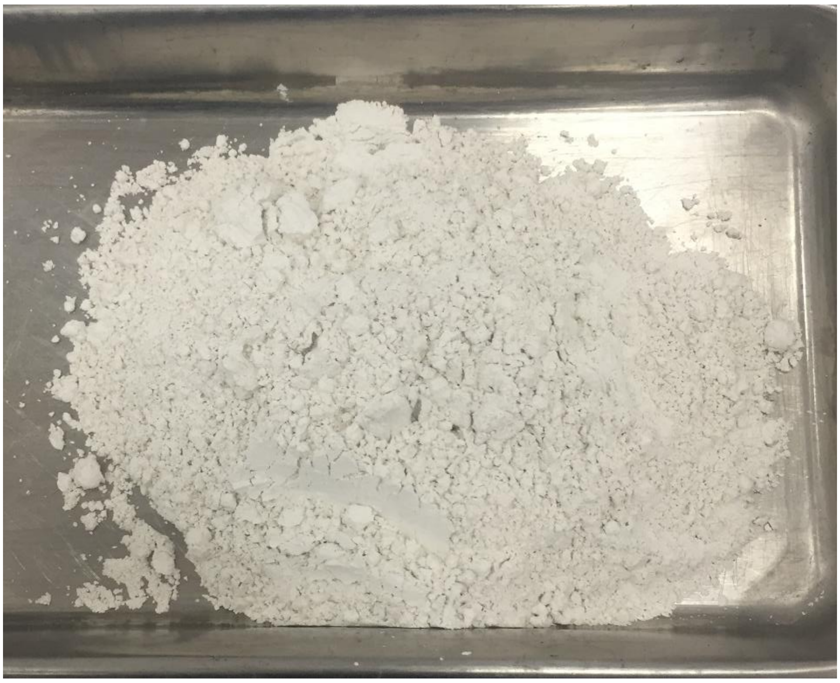

Figure 3. 3 Slaked powdered lime

Table 3. 3 The chemical composition of slaked powdered lime

Main compound	Amount (%)
SiO_2	< 1.3
Al_2O_3	0.4 – 0.8
Fe_2O_3	< 0.3
CaO	70.8
Na_2O	< 0.2
K_2O	< 0.2
MgO	< 0.8

3.1.3 Fly ash

20-50% of lignite coal used in thermal power plants generating electrical energy and 10-15% of hard coal comes out as ash. 75-85% of this ash is removed from the boiler by flue gases and this ash is described as "thermal power plant fly ash". In this research, the thermal power plant fly ash used was received from İzmir Kâtip Çelebi University, Civil Engineering Department, construction Materials Laboratory. The maximum grain size of fly ash is 0.425 mm. The fly ash which is used in this research is shown in Figure 3.4. The chemical compositions of the utilized fly ash are shown in Table 3.4.

Figure 3. 4 Thermal power plant fly ash

Table 3. 4 Chemical composition of thermal power plant fly ash

Main compound	Amount (%)
SiO_2	43.3
Al_2O_3	24.1
P_2O_5	0.2
CaO	14.9
Na_2O	0.3
SO_3	4.1
TiO_2	0.9
Cr_2O_3	0.02
K_2O	2.6
MgO	3.1

3.2 Method

For a complete laboratory testing into the materials identified, a series of experimental tests were specified. To analyze the geotechnical characteristics of alluvial soils, wash sieve analysis, specific gravity test, and Atterberg's limit tests were conducted. To determine the compaction properties and compaction behavior of alluvial soils, a series of standard compaction experiments were conducted. To analyze the bearing capacity of alluvial soils and to investigate the impact of fly ash and lime on the bearing capacity of alluvial soils a series of California Bearing Ratio experiments were done. All experiments were conducted according to the process given in the American Society for Testing and Materials (ASTM) and British Standards (BS).

3.2.1 Wash sieve analysis

To better determine the grain size distribution of all three groups of the coarse, medium, and fine grain samples, a washed sieve analysis test conducted for each soil sample [64].

It is possible to isolate material smaller than the 75-μm (No. 200) sieve from larger particles or to break down soil aggregations much more effectively and completely by wash sieving than by dry sieving. These test methods are therefore used on the test specimen before dry sieving, or as a measurement of the percentage of material that is finer than a 75-µm (No. 200) sieve, when specific determinations of material thinner than a 75-µm (No. 200) sieve are desired. The additional volume of material collected in the dry sieving process that is thinner than a 75-µm (No. 200) sieve is typically a small amount.

3.2.1.1 Sample preparation and procedure

Wash sieve analysis was conducted according to ASTM D1140-17 [64]. After obtaining a 500 gr representative specimen of sufficient size for each group of soil as mentioned in section 3.1.1, soil samples transferred into a pre-weighed container. The entire test specimens dried to a measurable mass at a heat of 110 ± 5°C (230 6 9°F) and determined the mass to four significant digits. Figure 3.5 shows different sizes of (4.74mm, 2mm, 1mm, 0.425mm, and 0.075mm) standard sieves used in sieve analysis.

Figure 3. 5 Different size standard sieves

By adding sodium hexametaphosphate to the water, a dispersing solution was prepared. The solution was added to the soil samples and specimens were fully soaked for at least two hours. After Shaking the contents of the container vigorously soils immediately moved from the container to the 75-µm (No. 200) washing sieve which was guarded with No. 10 sieves. The transfer process is performed in several transfers. It is important to maintain the specimen size to a volume that will not overwhelm the wash sieve and cause overflow. The samples were cleaned using a stream of water from the water distribution system on the sieve(s). The coarse particles of the material have been washed thoroughly. The material was retained on the No. 200 sieve and all the guard sieves transferred back into the specimen container. The soils were then dried at a temperature of 110 ± 5 Co, the retained mass was determined by a balance having an accuracy of ± 0,001. The amount of material passing the No. 200 sieve was then calculated. Dry sieve analyses were conducted for determining the coarser particle size distribution [65]. Figure 3.6 and 3.7 show the remained particles of fine soil sample on No. 200 sieve and dried sample of wash sieve test.

Figure 3. 6 Remained particles of F.G sample on No. 200 sieve

Figure 3. 7 Oven dried samples of C.G, M.G, and F.G samples after wash sieve test

3.2.2 Specific gravity

3.2.2.1 Definition and theory

Specific gravity experiments were conducted according to Standard experiment methods for specific gravity of soil solids by water pycnometer, ASTM 2014 [66]. The specific gravity of a particular substance is calculated as the proportion between the weight of a given material volume and the weight of an equivalent distilled water volume. In civil engineering, a significant factor for the measurement of the weight-volume interaction is the specific gravity of soil solids (frequently referred to as the specific gravity of the soil). The specific gravity of soil solids can be determined by using a 500 ml water pycnometer.

3.2.2.2 Sample preparation

30 gr (w_1) oven-dried soil prepared for coarse, medium, and fine soils. Then, the pycnometer was filled with distilled water and the vacuum was applied for at least 10 minutes. The weight of the pycnometer and water (W_2) was determined. Pycnometer was cleaned and dried, by using a funnel 30 gr soil was added into pycnometer and a slurry prepared by adding water between 1/3 and ½ of the depth of the pycnometer.

Figure 3. 8 Specific gravity test's equipment

The specific gravity of soil was calculated using the following formula:

$$G_S = \frac{w_1}{w_1 + (w_2 - w_3)} \tag{3.1}$$

Where;

G_S: specific gravity

w_1: weight of dry soil sample

w_2: weight of picnometer and water

w_3: weight of picnometer, water and soil sample

For de-airing vacuum was applied for 10 minutes, the pycnometer was filled with water and the mass of the pycnometer, soil and, water (W_3) was determined. Figure 3.8 shows the equipment of the specific gravity test.

3.2.3 Standard Proctor test

The standard compaction experiment (ASTM D698)[64] was performed to evaluate the maximum dry density and optimum water content using the standard compaction process. Using the standard proctor test, soil compaction was calculated, which is typically conducted along with water using mechanical compactors, rammers, and rollers. Soil compaction happens when the void ratio is reduced and the air is expelled through soil particle rearrangement. Improving the load-bearing capacity and consistency of slopes and minimizing unwanted settlement and changes in volume are the main goals of the compaction method. In geotechnical projects such as field dams, landfill liners, highway base courses, subgrades, and embankments, compacted soil is commonly used.

3.2.3.1 Definition and theory

The standard proctor experiment contains a mixture of dry soil with various water percentages. In a cylindrical mold, it is then compacted (i.e., mold volume: 9.44*10-3m). By employing 25 hammer blasts measuring 2.5 kg, the soil is compacted into three equal layers and this is dropped from a height of 305mm.

The transmitted hammer energy is determined by:

$$E_{comp} = m*h*g* Nb* Nl \tag{3.2}$$

m_h: Mass of the hammer

h_d: Height of fall of the hammer

g: Acceleration due to gravity

V: Volume of compacted soil

N_b : Number of blows

N_l : Number of layers

E_{comp}: $2.5 * 9.8 * 25 * 3 * 10^{-3}$: 594 $KJ/m3$

The amount of compaction is determined by the dry unit weight. On the basis of moisture-dry curves of density, maximum dry density, and optimal soil moisture content are achieved. 100% of the maximum dry unit weight for compaction application was performed in practice. The desired compaction curve is shown in Figure 3.9. At two measurement points of water content, i.e. before and after the maximum dry unit weight, this level of compaction could be achieved [65].

The "dry of optimum" that is achieved before the dry maximum unit weight is normaly implemented in cases of small-volume soil changes due to changes in water content, such as granular soils, clay-sand, and sandy clay. On the other side, the "wet optimum" is calculated for soils with significant volume changes due to changes in water content, such as expansive and collapsible soils. However, to avoid sudden failure, which occurs in some cases, the compaction wet of optimum was expected to be set at a level of 5 to 15 percent to avoid a less reasonable range of optimum water content [65].

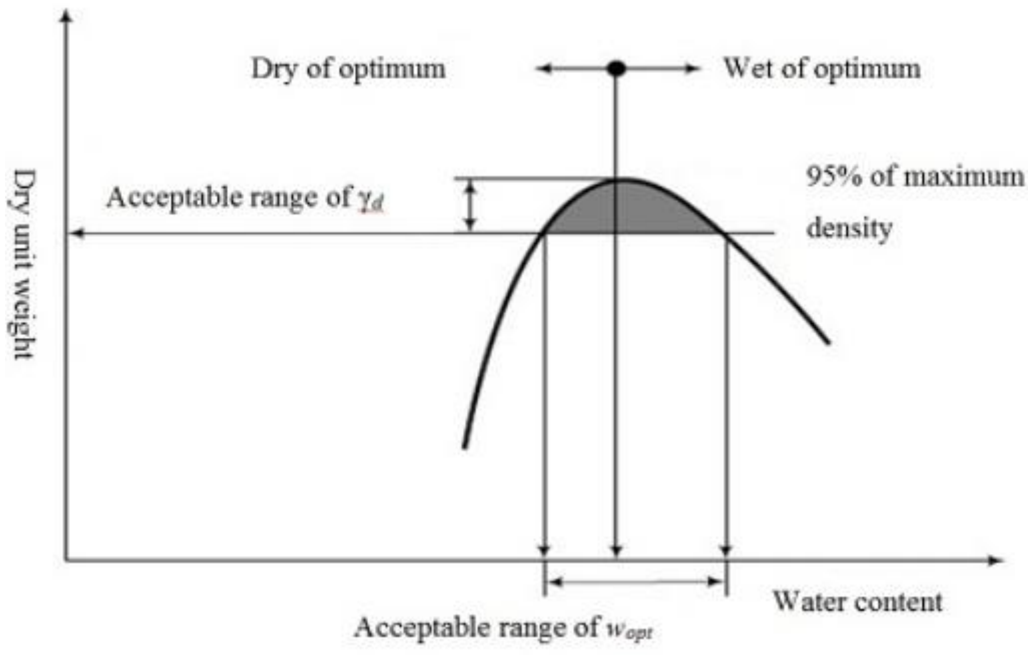

Figure 3. 9 Desired compaction curve [65]

3.2.3.2 Sample preparation

For the sample preparation, approximately 2.5 kg soil for each group of the coarse, medium, and fine soils were dried in the oven at a temperature of 105±5 C^o for 24 hours. The initial set of compaction experiments were applied to determine the compaction properties of untreated coarse, medium, and fine alluvial soils. Based on the dry weight of each soil, other samples were mixed with varying proportions of 10%, 15%, and 20% fly ash.

Samples have been compacted in a 105 mm-diameter mold, applying the standard proctor effort. The dry unit weight and humidity content of each specimen were achieved from the unit weight obtained at the optimum moisture stage, obtained via the intersection of the slopes drawn from the wet side and the dry side of the compaction curve by at least 5 compaction experiment. The mold and rammer used in compaction test is shown in Figure 3.10.

Figure 3. 10 Mold and rammer used in Standard Proctor test

3.2.4 Liquid and plastic limits

The upper and lower limits of the water content range over which plastic behavior is exhibited by the soil are described as liquid (w_l) and plastic limit (w_p), respectively. The soil flows like a liquid (slurry) above the liquid level; below the plastic limit, the soil is brittle and crumbly. The range of water content itself is known as the index of plasticity (I p), and plasticity index can be calculated by the following formula:

$$I_p = w_l - w_p \tag{3.3}$$

Where;

I_p: Plasticity index

w_l: Liquid limit

w_p: Plastic limit

Plasticity is an important feature in the event of fine-grained soil, the capacity of the soil to experience irreparable deformation without cracking or crumbling. In general, soil could be in a liquid, plastic, semi-solid and solid state, based on its moisture content (given as the ratio of the mass of water in the soil to the mass of solid particles). At the limit of shrinkage, given as the water content at which the soil volume approaches its minimum values as the transformation between semi-solid and solid states dries out.

3.2.4.1 Procedure and sample preparation

Liquid and plastic limit experiments are done in accordance with ASTM D4318 (2010)[66]. Using just a mortar and a pestle of rubber, the oven-dried soil sample is allowed to be crumbled and broken up without smashing individual particles, just material passing a 0.425 mm sieve is usually used for the test. The Casagrande apparatus, (Figure 3.11), which is common in the United States and other parts of the globe (ASTM D4318) [66] is used to decide the liquid limit.

Figure 3. 11 Casagrande apparatus for liquid limit test

In a pivoting flat metal cup, a soil paste is inserted and a groove is separated. A mechanism allows the cup to be raised and lowered onto a hard rubber base to a height of 10 mm. When the cup is repeatedly lowered, the two halves of the soil eventually flow together. The moisture content of the sample in the cup is then measured; this is drawn against the number of blows logarithm, and the best straight line is drawn fitting the drawn points. The liquid limit is specified for this test as the moisture content at where 25 blows are needed to close the groove bottom.

3.2.4.2 Plastic limit

The experiment soil is thoroughly mixed with distilled water to assess the plastic limit until it becomes plastic enough to form into a ball. Some of the soil sample (about 2.5gr) is shaped between the first fingers and the thumb of each hand into a thread, almost 6 mm in diameter. On a glass plate, the thread is then placed and rounded with one-handed fingertips until its diameter is limited to 3 mm or so; the rolling pressure during the test must be uniform. Figure 3.12 shows the craked sample of the plastic limit experiment.

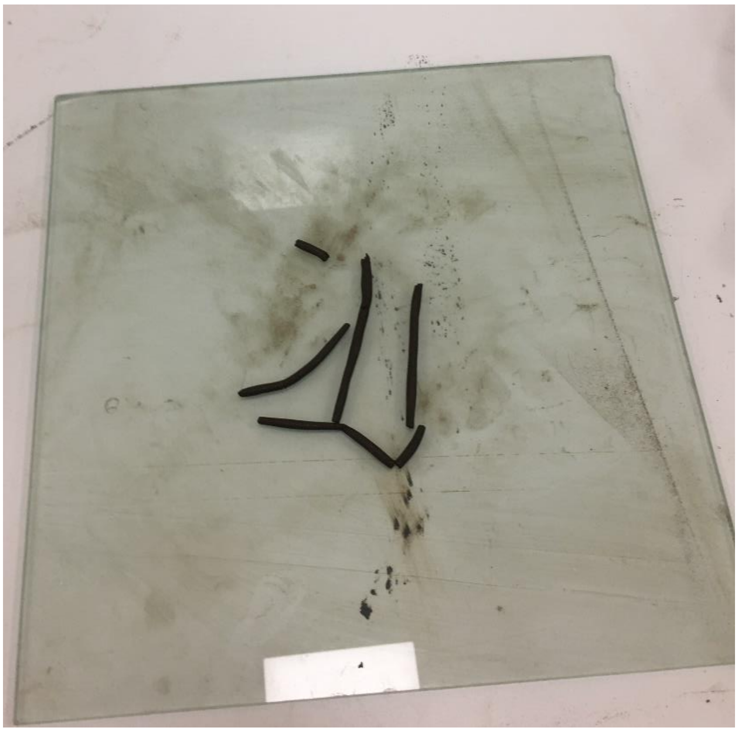

Figure 3. 12 Cracked sample of plastic limit test

The thread is then molded among the fingers (the moisture content is decreased by the humidity of the fingers) and the process is go on until the soil shear thread is spinned to a diameter of 3 mm both longitudinally and transversely. Using three more sections of the sample, the process is repeated and the amount of moisture content of all the crumbled soil is calculated as a whole. This amount of water (to the closest integer) is known as the soil's plastic limit. Using four other sub-samples, the whole test is replicated, and the plastic limit value is the average value taken.

3.2.5 California Bearing Ratio (CBR) test

3.2.5.1 Definition and theory

This testing procedure includes the evaluation of the California Bearing Ratio (CBR) of compacted sample pavement subgrade, subbase, and base course material.

The California Bearing Ratio (CBR) experiment is a load test that is attached to the surface and used to assist pavement model in soil investigations. The testing procedure uses a round axis to enter, at a steady rate of 1.27 (mm/sec) penetrating, material compacted in a mold. The CBR is defined as ratio of the unit load on the piston needed to penetrate 0.1 inches (2.5 mm) and 0.2 inches (5 mm) of the test soil to the unit load needed to penetrate a well-graded crusted stone standard material.

3.2.5.2 Procedure and samples preparation

All CBR experiments were conducted according to ASTM D1883-07 [67]. According to section 3.3.1, and using the combination mentioned in that section, the soil samples for coarse, medium, and fine soils were prepared, (Figure 3.13).

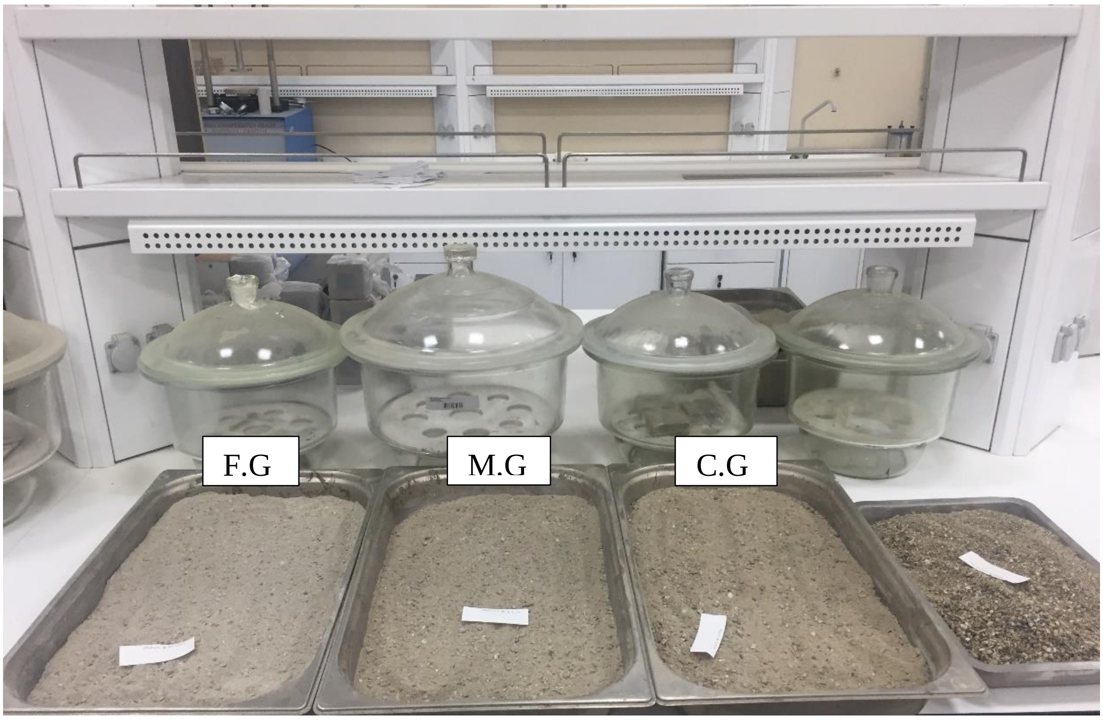

Figure 3. 13 Prepared fine, medium, and coarse grain samples

All CBR experiments were conducted on the soil samples at the maximum dry unit weight and optimum water value for the soil as determined using standard compaction. Firstly, tests were performed for untreated coarse, medium, and fine-grained soil samples. To examine the effect of lime on the bearing capacity of these soils, 2%, 4%, 6%, and 10% lime was thoroughly mixed by dry weight of soil and then experiments were performed. To examine the impact of fly ash on the load-bearing capacity of these soils, 10%, 15%, and 20% fly ash was mixed by dry weight of soil and then tests were performed.

Figure 3. 14 Mold, rammer and prepared soil sample for CBR test

In a CBR mold with a diameter of 150 mm and a height of 175 mm with a removable perforated base plate, the exact quantity of oven-dried (100-105C^{o}) soil was measured and properly mixed with water according to its optimum moisture content (OMC). The soil was then compacted to the maximum weight of the dry unit achieved by the standard laboratory Proctor experiment. Figure 3.14 shows the prepared sample, mold, and rammer used in the CBR test. Molds were soaked in a water sink for 24, 96, and 168 hours for curing. Figure 3. 15, 3.16, and 3.17 show different stages of the CBR test.

Figure 3. 15 Compacted soil sample in CBR mold

Figure 3. 16 Soaking stage of CBR test

Figure 3. 17 CBR test’s loading machine

4. RESULTS AND DISCUSSION

4.1 An overview of this part’s contents

A set of experimental measures were specified for a complete laboratory inquiry into the materials chosen. Wash sieve analysis [68], specific gravity test [69], and Atterberg limit tests [66] have been performed to examine the geotechnical properties of alluvial soils. A series of standard Proctor experiments [64] have been performed to determine the compaction properties and compaction behavior of alluvial soils. California Bearing Ratio experiments [67] were conducted to examine the bearing capacity of alluvial soils and to study the impact of lime and fly ash on the bearing capacity of alluvial soils.

The results are separated into three main parts;

1. Index properties of coarse, medium, and fine grain alluvial soils
2. Compaction behavior and affect of fly ash on compaction beahvior of three groups of alluvial soils.
3. Affect of lime and fly ash on bearing capacity of the coarse, medium, and fine grain alluvial soils.

Then, each part focuses accurately on the specified targets, as mentioned in section 1.1.

The investigation of the bearing capacity of alluvial soil and the effect of lime/fly ash on this property of alluvial soil was separated into two groups.

1. Examining the CBR of the untreated coarse, medium, and fine grain alluvial soil and examining the CBR of these three groups of alluvial soil mixed with 2%, 4%, 6%, and 10% lime.

2. Examining the CBR of the coarse, medium, and fine grain alluvial soil mixed with 10%, 15%, and 20% fly ash.

4.2 Index properties

Laboratory experiments on disturbed alluvial soil samples were performed to define the index properties of the coarse, medium, and fine grain alluvial soils. Three groups of the sample were prepared. The soil mechanics tests were performed at the Soil Mechanics Laboratory of the Department of Civil Engineering at İzmir Katip Celebi University İzmir, Turkey. In the index properties part, the three groups of the coarse, medium, and fine grain alluvial soils were used for grain size distribution, specific gravity, standard compaction test, and Atterberg's limits studies.

4.2.1 Grain size distribution

Wash sieve analysis was performed for each group of alluvial deposits to determine the percentage of different grain sizes contained within each group's samples. Wash sieve analysis was performed according to ASTM D1140-17 [68]. The Figure 4.1 demonstrates the grain size distribution graphs for coarse, medium, and fine grain alluvial soils. Table 4.1 shows the percentages of silt + clay, sand, and gravel for each group of alluvial deposits.

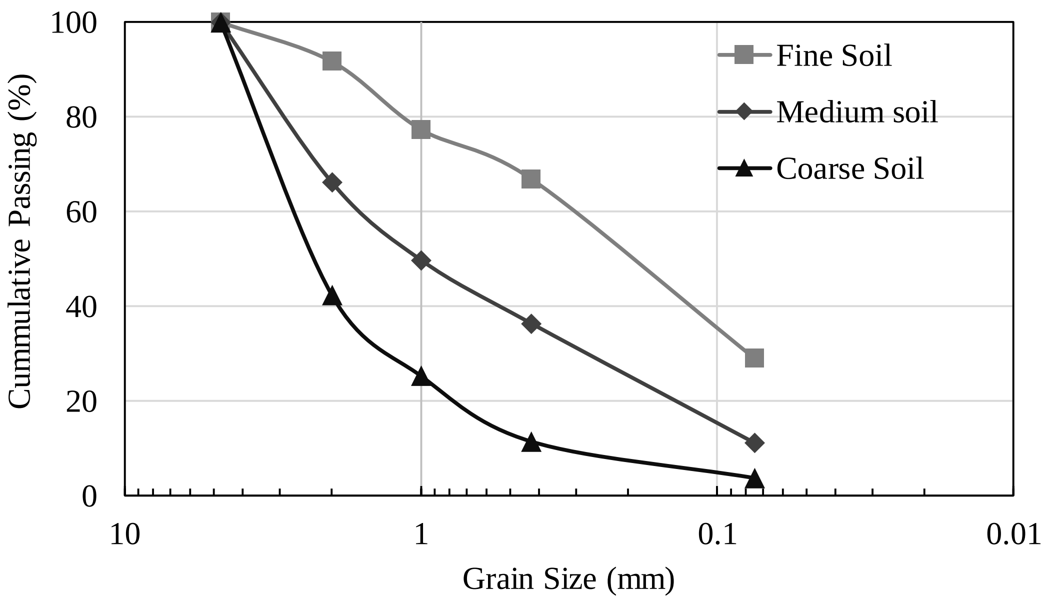

Figure 4. 1 Grain size distribution curves of the fine, medium, and coarse grain samples

Table 4. 1 Percentage of silt/clay, sand, and gravel for each group of alluvial deposits

Group Type	Gravel	Sand	Silt+Clay
Fine Grain	8.3 %	62.64 %	29.04 %
Medium Grain	33.84 %	55.04 %	11.12 %
Coarse Grain	57.6 %	38.6 %	3.8 %

4.2.2 Specific gravity

The specific gravity of coarse, medium and fine grain alluvial deposits are determined. Also, the specific gravity of lime and fly ash is examined as well. Three set of experiment is performed for each material and the average is calculated. Table 4.2 shows the specific gravity of all materials used in this study. All specific gravity experiments were run using standard water pycnometer testing procedures for specific gravity of soil solids, ASTM 2014 [69].

Table 4. 2 Specific gravity of C.G, M.G, F.G soils, lime, and fly ash

Used material	C.G Soil	M.G Soil	F.G Soil	Lime	Fly Ash
1. set	2.67	2.66	2.69	2.65	2.3
2. set	2.65	2.69	2.67	2.54	2.28
3. set	2.68	2.66	2.68	2.65	2.33
Average	2.66	2.67	2.68	2.61	2.3

4.2.3 Liquid and plastic limits tests results

4.2.3.1 Casagrande liquid limit test result

The liquid limit test is performed according to ASTM D4318-2010 [66]. According to standard only material passing 0.425 mm sieve is used in this experiment. For determination of liquid limit the Casagrande apparatus which is famous in the U.S. and other areas of the globe was used.

The upper and lower limits of the water content varies over where plastic behavior is exhibited by the soil are described as liquid (w_l) and plastic limit (w_p), respectively. The soil runs like a fluid (slurry) above the liquid level; below the plastic limit, the soil is brittle and crumbly. Table 4.3 shows the result of the Casagrande test. Figure 4.2 shows the number of blows versus water content.

Table 4. 3 Result of Casagrande test

No.	No. of Blows	Water content (%)
1. set	16	39.98
2. set	22	38.22
3. set	30	37.21
4. set	42	36

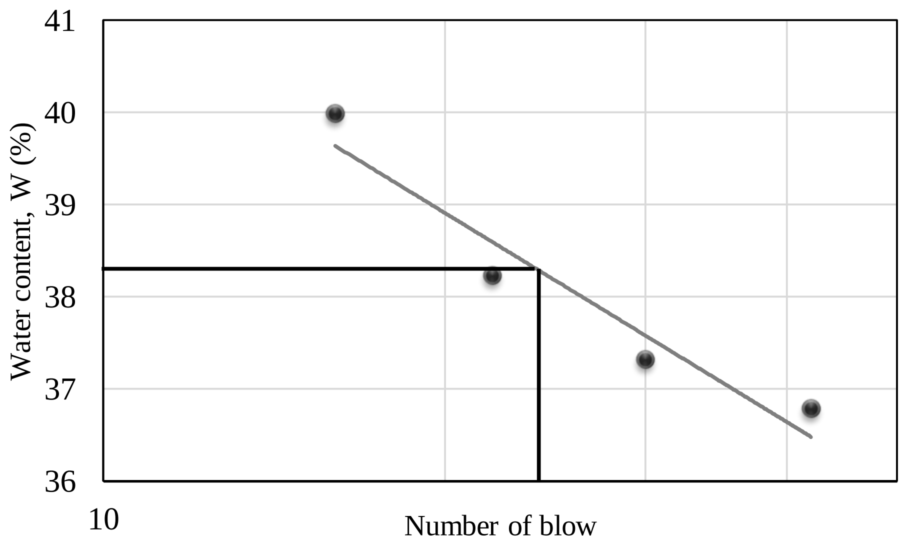

Figure 4. 2 Number of blows versus water content

4.2.4 Plastic limit test

The plastic limit experiment was carried out in accordance with ASTM D4318-2010 [66]. The test soil was mixed with distilled water to assess the plastic limit until it becomes sufficiently plastic to form a ball. Someof the soil specimen (about 2.5gr) was shaped between the first fingers and the thumb of each hand into a thread, almost six millimeter in diameter. The thread was then positioned on a glass plate and rolled with one-handed fingertips until the thread was reduced to approximately 3 mm in diameter. The thread was then molded amonng the fingers (the moisture content was decreased by the humidity of the fingers) and the process was repeated until the soil shear thread was rolled to a diameter of three millimeter both longitudinally and transversely. Using three additional sections of the sample, the process was repeated and the percentage of the moisture value of all the crumbled soil was calculated as a whole. This value of water (to the closest integer) is known as the soil's plastic limit. Using four other sub-samples, the entire procedure was replicated and the average of the plastic limit values is taken. Table 4.4 shows the results of the plastic limit test.

Table 4. 4 Results of plastic limit test

No.	Con. No.	W_1	W_2	W_3	Water Content (%)	Plastic Limit
1	16	15.36	22.63	21.13	25.9965	25.9 (%)
2	17	15.02	22.74	21.14	26.1437	
3	86	11.29	18.75	17.26	24.9581	
4	25	11.6	19.60	17.91	26.78	

The soil runs like a fluid (slurry) above the liquid level; below the plastic limit, the soil is brittle and crumbly. The range of water value itself is known as the index of plasticity (I p), plasticity index is calculated using the 3.3 formula.

The liquid and plastic limits have been compared to a broad range of soil engineering properties, and these Atterberg limits are often used to define a fine-grained soil in accordance to the Unified classification Method of Soil or system of AASHTO. The limits of liquid and plastic are used to define fine soils, using the plasticity graph shown in Figure 4.3.

Table 4. 5 Values of liquid limit, plastic limit, and plasticity index

Liquid limit (w_l)	38.3 %
Plastici limit (w_p)	25.9 %
Plasticity index (I_p)	12.4 %

The plasticity index and liquid limit are the axes of the plasticity chart, so a point on the chart can represent the plasticity characteristics of a particular soil.

According to the plasticity chart, the used fine soil is intermediate plasticity silt.

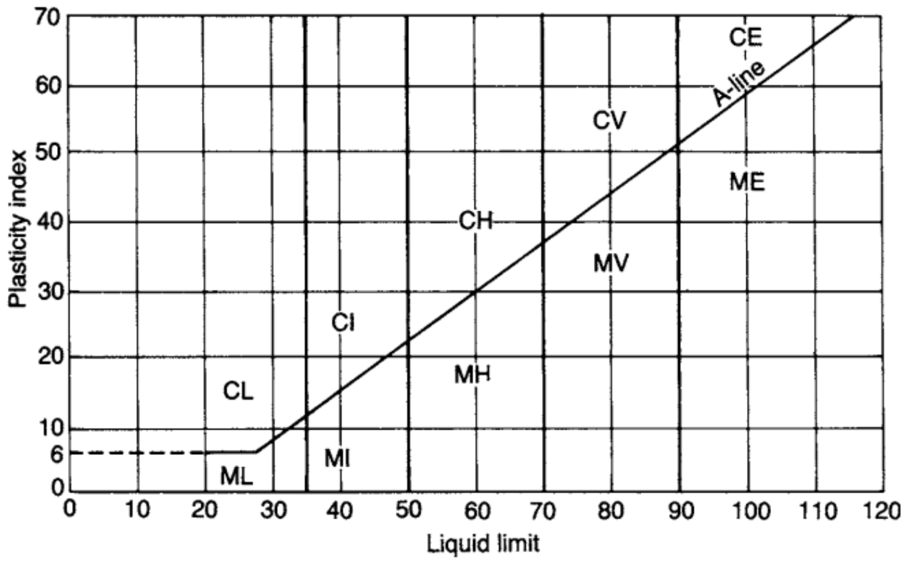

Figure 4. 3 Plasticity chart [65]

4.3 Standard compaction test results

The standard Proctor experiment (ASTM D698) [64] was performed for defining the maximum dry unit weight and optimum water content, using a standard compaction procedure. Three groups of alluvial deposits as coarse, medium, and fine grain soils were prepared and their compaction properties were investigated. Figure 4.4 shows the compaction curves of the coarse, medium, and fine grain untreated alluvial soils respectively. Figures 4.5, 4.6, and 4.7 show the compaction curves of fine, medium, and coarse grain soils treated with 10%, 15%, and 20% FA respectively. The effect of fly ash on the optimum water content and maximum dry unit weight of the fine, medium, and coarse grain soils are shown in Figures 4.8 and 4.9. Table 4.6 shows the changes of optimum water content and maximum dry unit weight by adding fly ash. There is an enhance in maximum dry unit weight and reduction in optimum moisture content for 10% FA treated all soils, then there is a clear pattern that optimum moisture content increases while maximum dry unit weight decreases in 15% and 20% FA treated soils. The reason of the decrease of the optimum moisture content, especially in fine soils with 10% fly ash, can be discussed as follows: the ion exchange among chemicals and fine soil reduces the density of the electrical double layer and enhances flocculation. The flocculation of solid materials means that water-additive-soil mixtures could be compacted with a lower moisture content and an optimum water content can be decreased. The reduction in the optimum moisture content implies that alluvial soil can be stabilized by applying fly ash to soils with a low moisture content. The reduction in the maximum dry weight via an increased percentage of fly ash is due to the decreased specific gravity of the fly ash opposed to alluvial soil and the instant creation of cemented products which decrease the density of the soil treated [49], [60].

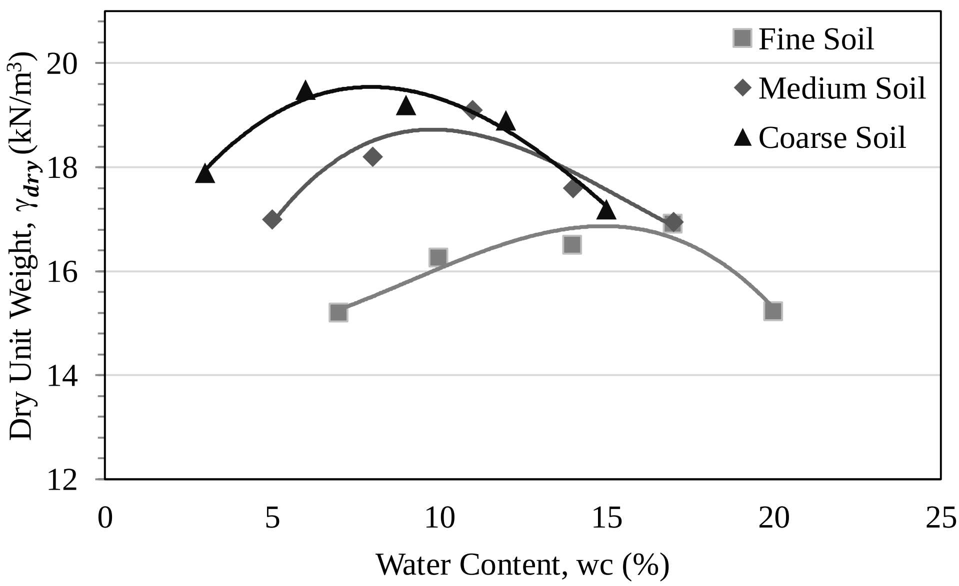

Figure 4. 4 Compaction curves of fine, medium, and coarse soils

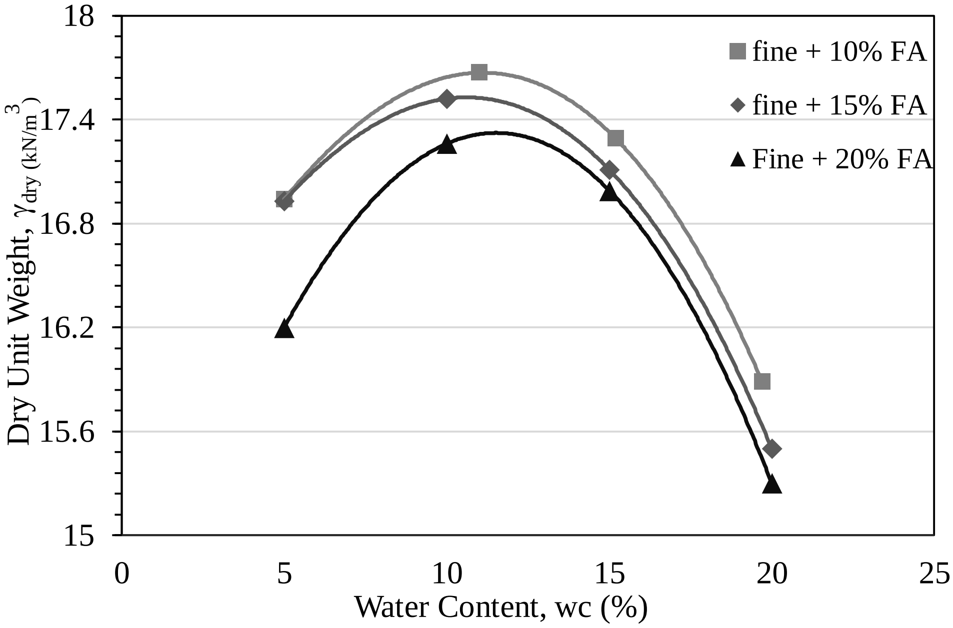

Figure 4. 5 Compaction curves of fine soil treated with 10%, 15%, and 20% FA

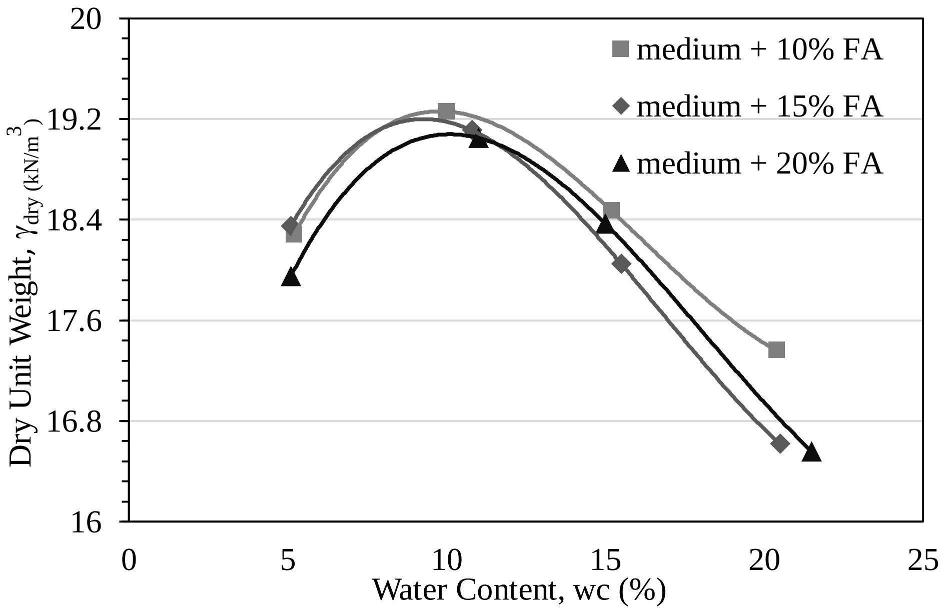

Figure 4. 6 Compaction curves of medium soil treated with 10%, 15%, and 20% FA

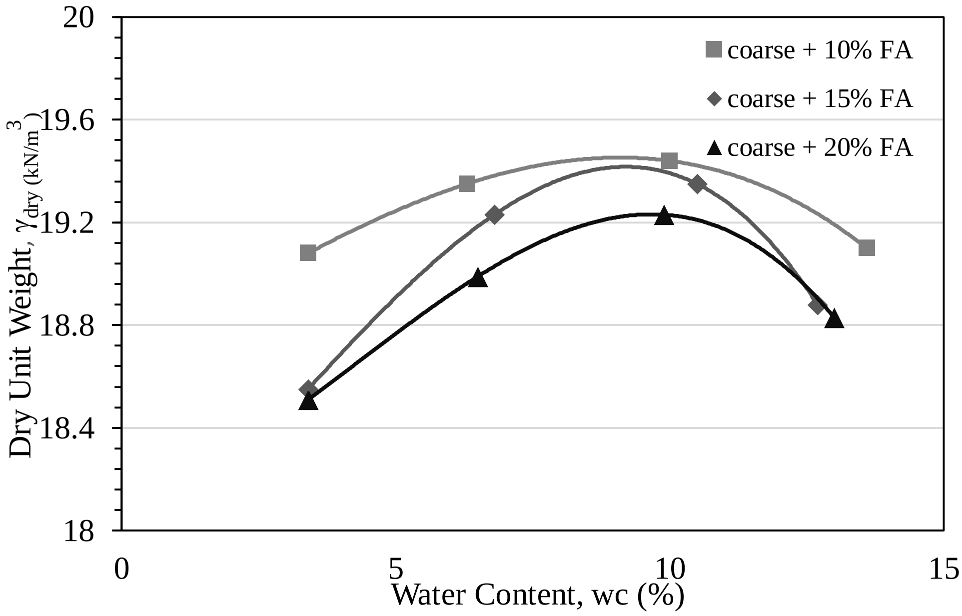

Figure 4. 7 Compaction curves of coarse soil treated with 10%, 15%, and 20% FA

Zha et al. [59] studied the behavior of expansive soil stabilized with fly ash. Standard Proctor experiments were performed on the fly ash treated soils. Their study shows, Via an increase in the content of fly ash, the maximum dry weight and optimum moisture content reduced. By rising the fly ash amount to 15%, optimum water value and maximum dry unit weight decreased 7% and 4.3% respectively.

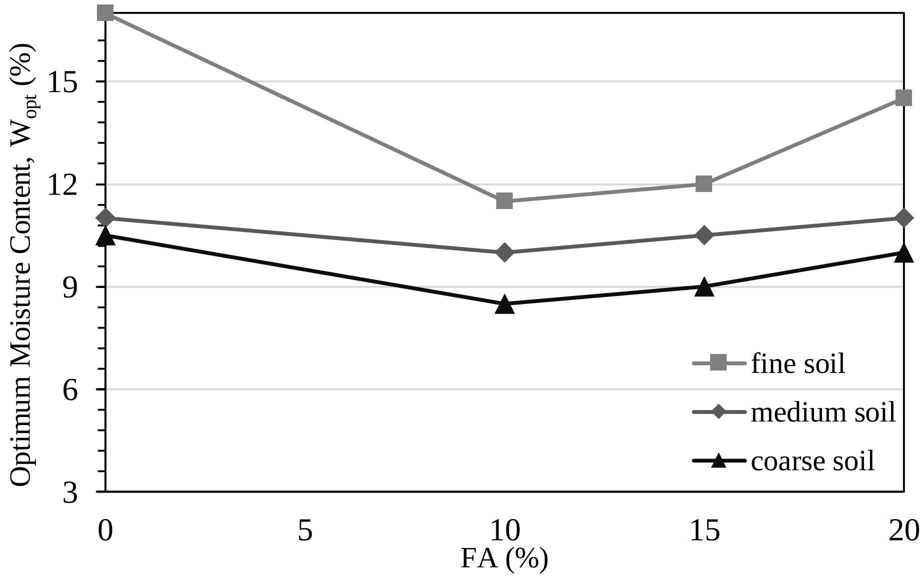

Figure 4. 8 Optimum moisture content versus FA content of fine, medium, and coarse soils treated with 10%, 15%, and 20% FA

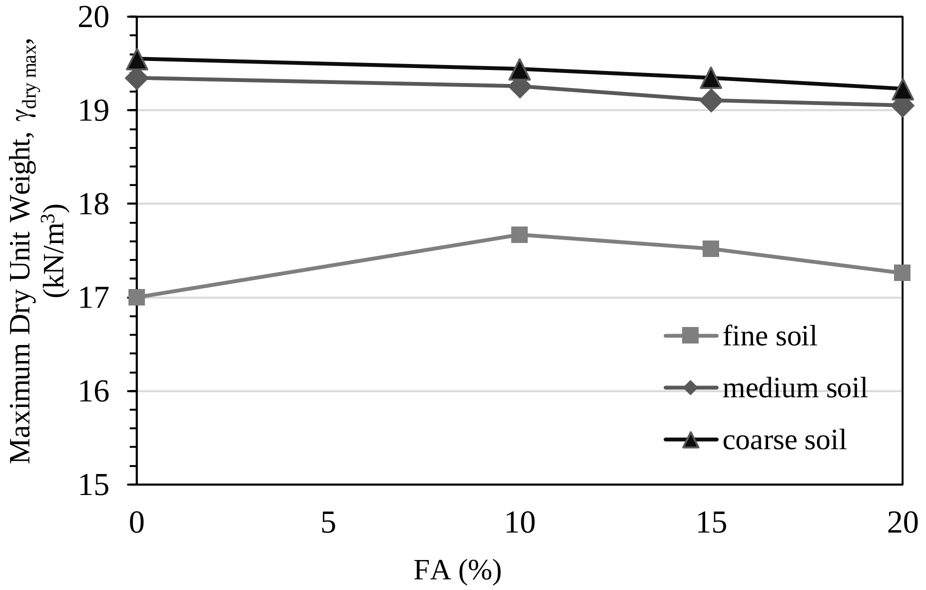

Figure 4. 9 Maximum dry unit weight VS FA content of fine, medium, and coarse soils treated with 10%, 15%, and 20% FA

Table 4. 6 Changes of maximum dry unit weight and optimum water content

FA (%)	Fine		Medium		Coarse	
	$\gamma_{dry\ max}$ (kN/m^3)	W_{opt} (%)	$\gamma_{dry\ max}$ (kN/m^3)	W_{opt} (%)	$\gamma_{dry\ max}$ (kN/m^3)	W_{opt} (%)
0	16.9	17	19.35	11	19.55	10.5
10	17.67	11.5	19.26	10	19.44	8.5
15	17.52	12	19.11	10.5	19.35	9
20	17.26	14.5	19.05	11	19.23	10

4.4 CBR tests results

4.4.1 Untreated alluvial deposits

CBR is the measurement of material tolerance to standard piston penetration under governed density and humidity level. The exact amount of oven-dried soil was considered and properly mixed with water equivalent to its optimum moisture content (OMC) in a CBR mold with a diameter of 150 mm and a height of 175 mm with a removable perforated base plate. The soil was then compacted to its optimum dry weight achieved by the laboratory standard test method.

The CBR experiments were done to examine the bearing capacity of different grain size alluvial deposits. To study the impact of curing time on soaked CBR value, 24, 96, 168 hours were considered as curing time. All CBR tests were conducted according to ASTM 1883-07 [67]. Figure 4.10, 4.11, and 4.12 show the fine, medium, and coarse grain samples CBR curves respectively.

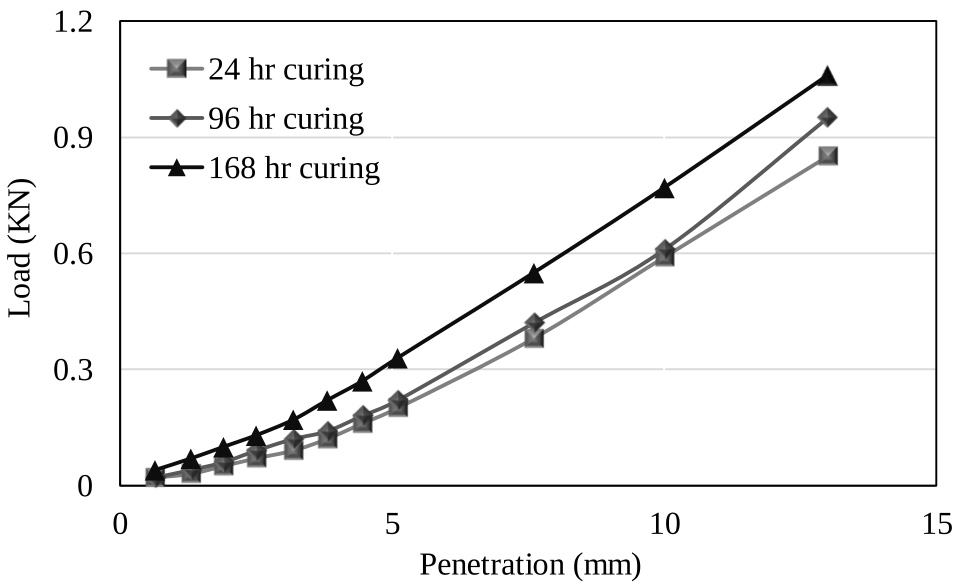

Figure 4. 10 CBR curves of fine soil with 24, 96, and 168 hour curing

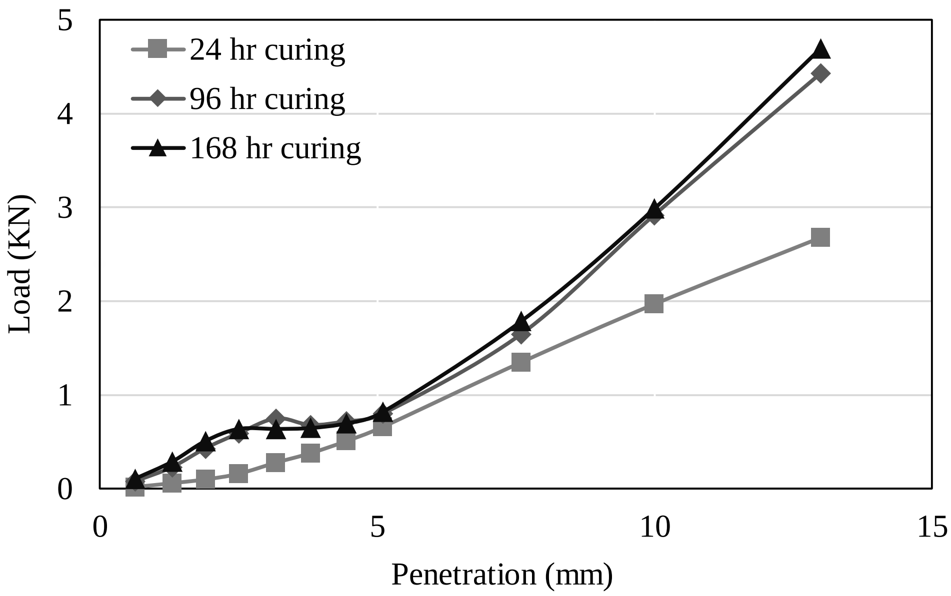

Figure 4. 11 CBR curves of medium soil with 24, 96, and 168 hour curing time

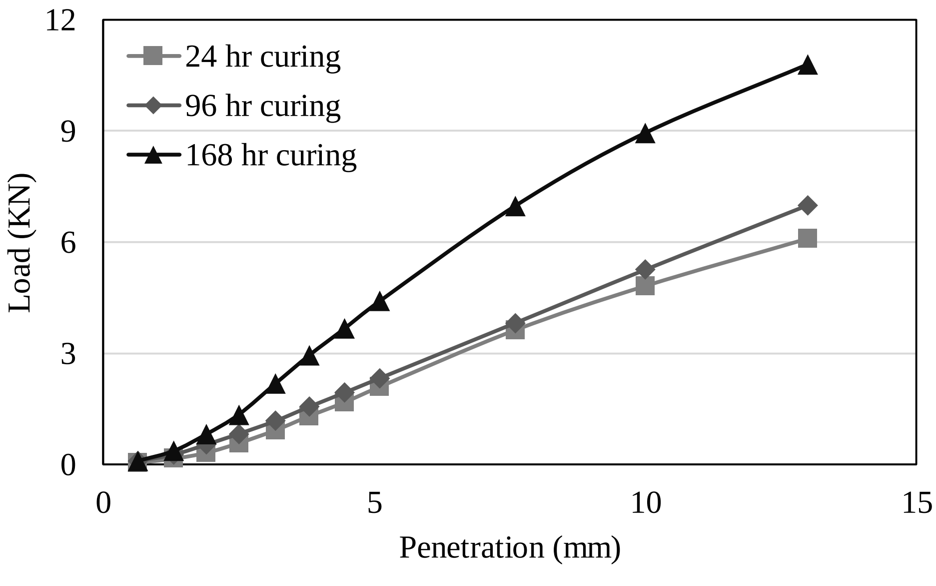

Figure 4. 12 CBR curves of coarse soil with 24, 96, and 168 hour curing time

To better understand the impact of curing time and grain size on load-bearing capacity of alluvial deposits, the compression between F.G, M.G, and C.G samples and 24, 96, and 168 hours of curing time is shown in Figure 4.13.

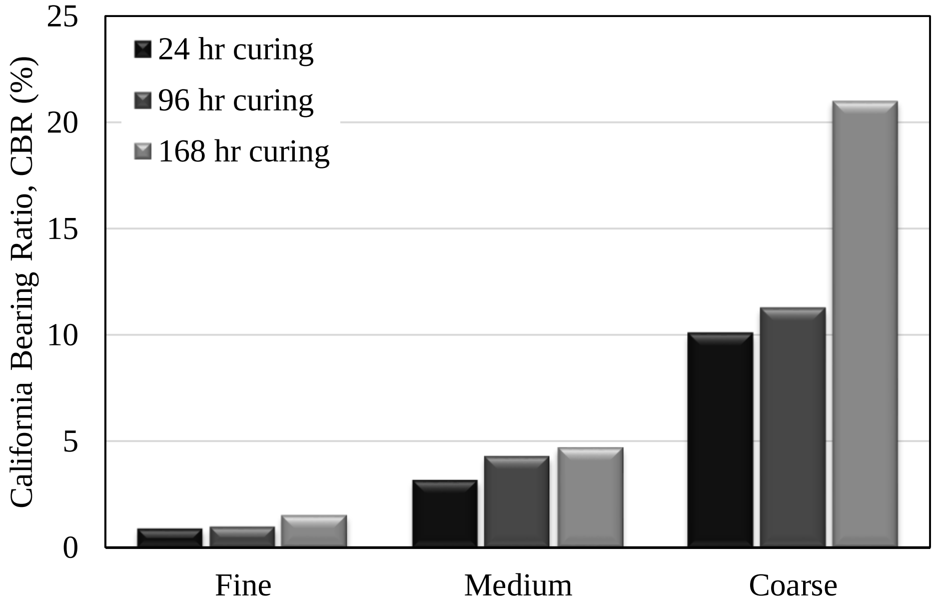

Figure 4. 13 CBR value comparison between fine, medium, and coarse soils

As it can be observe from Figure 4.13, C.G sample with 168 hour curing time has the highest CBR value, while, F.G sample with 24 hour curing time has the lowest CBR value. An abvious effect of grain size and curing time can be observed. Coarse grain samples has higher CBR values. Increasing curing increases the CBR value of soils.

The summary of CBR values for each group of sample in each curring time period is presented in Table 4.7.

Table 4. 7 CBR values in various curing time

	CBR Value (%)		
Curing Time	Fine	Medium	Coarse
24 hr	0.89	3.17	10.07
96 hr	0.98	4.3	11.26
168 hr	1.53	4.7	21

4.4.2 CBR tests results of alluvial deposits treated with lime

To examine the impact of lime on stabilization of alluvial deposits, 2%, 4%, 6% and 10% lime by dry weight of soil was mixed with coarse, medium, and fine grain alluvial soils. For curing time, 24, 96, and 168 hours were considered. Figure 4.14, 4.15, 4.16, and 4.17 show the load and penetration curves of coarse grain alluvial deposits mixed with 2%, 4%, 6%, and 10% lime with different curing times respectively.

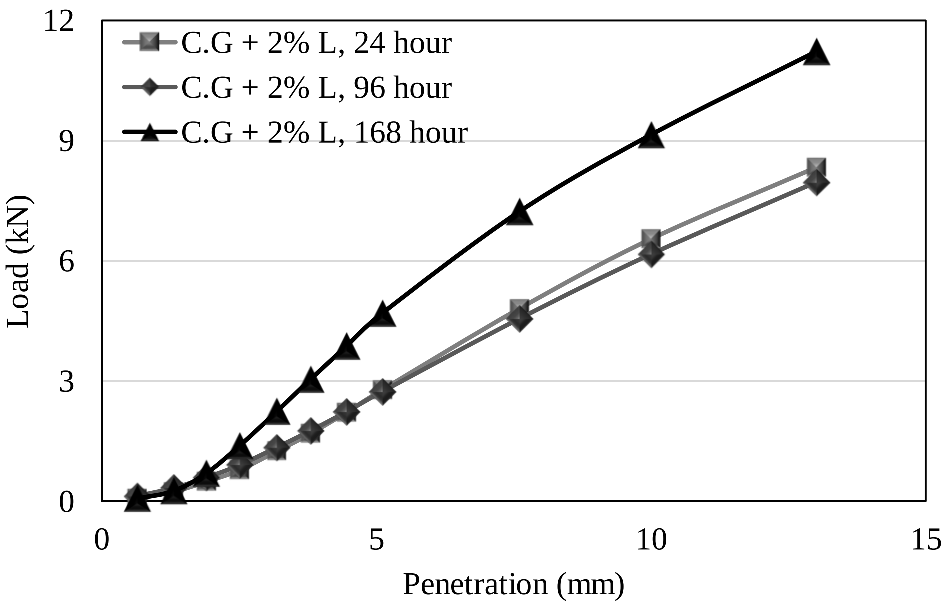

Figure 4. 14 Load penetration curve of C.G sample with 2% lime

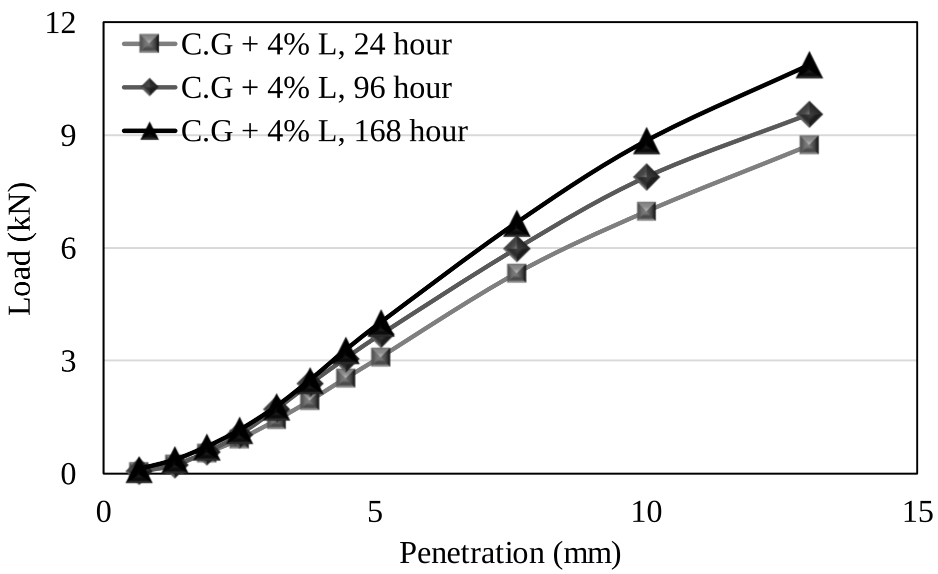

Figure 4. 15 Load penetration curve of C.G sample with 4% lime

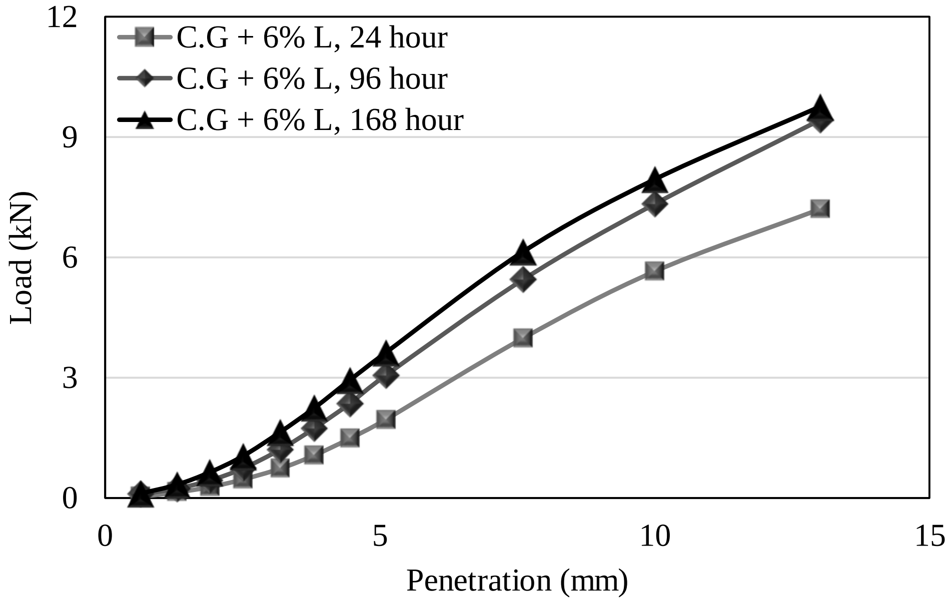

Figure 4. 16 Load penetration curve of C.G sample with 6% lime

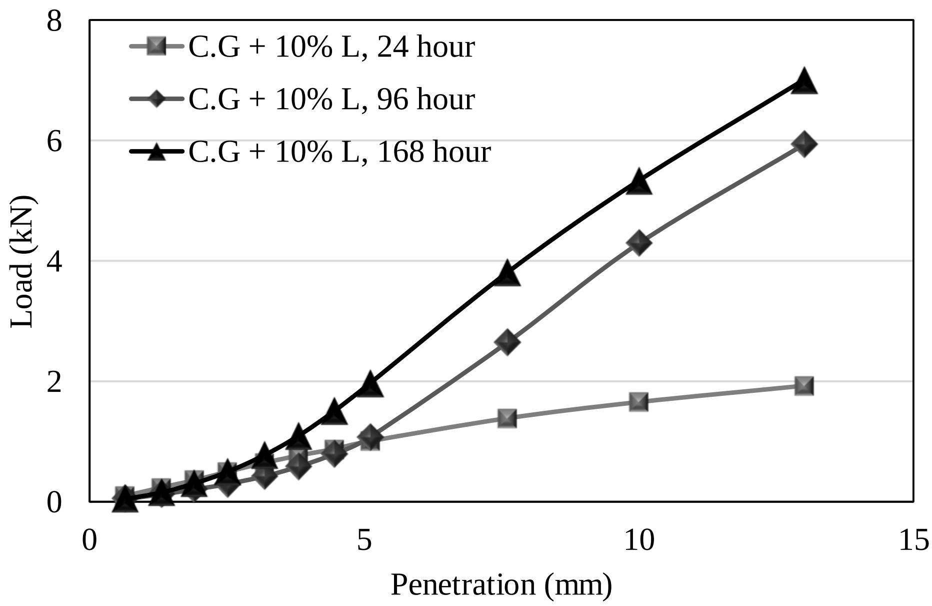

Figure 4. 17 Load penetration curve of C.G sample with 10% lime

Figure 4.18, 4.19, 4.20, and 4.21 show the load and penetration curves of medium grain alluvial deposits mixed with 2%, 4%, 6%, and 10% lime with different curing times respectively.

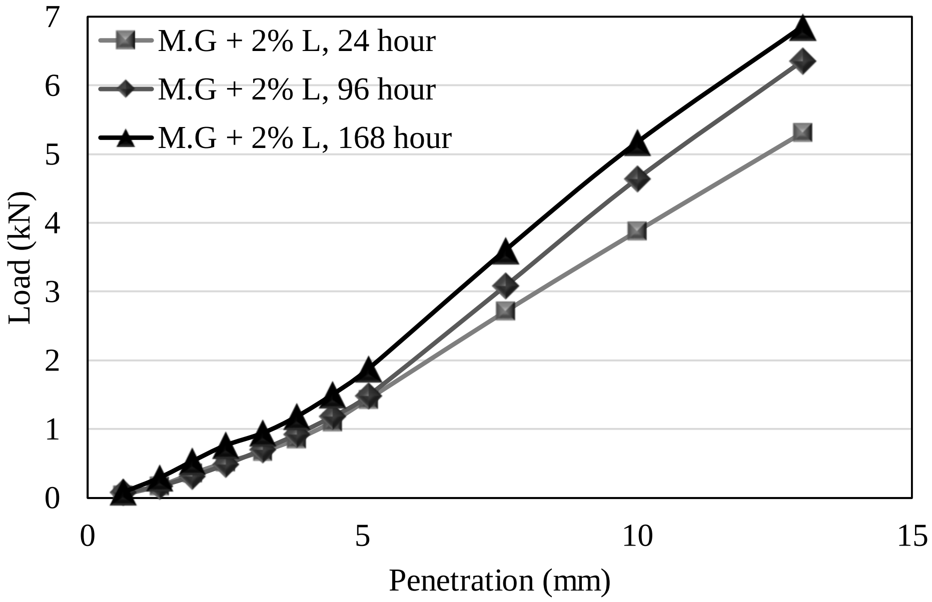

Figure 4. 18 Load penetration curve of M.G sample with 2% lime

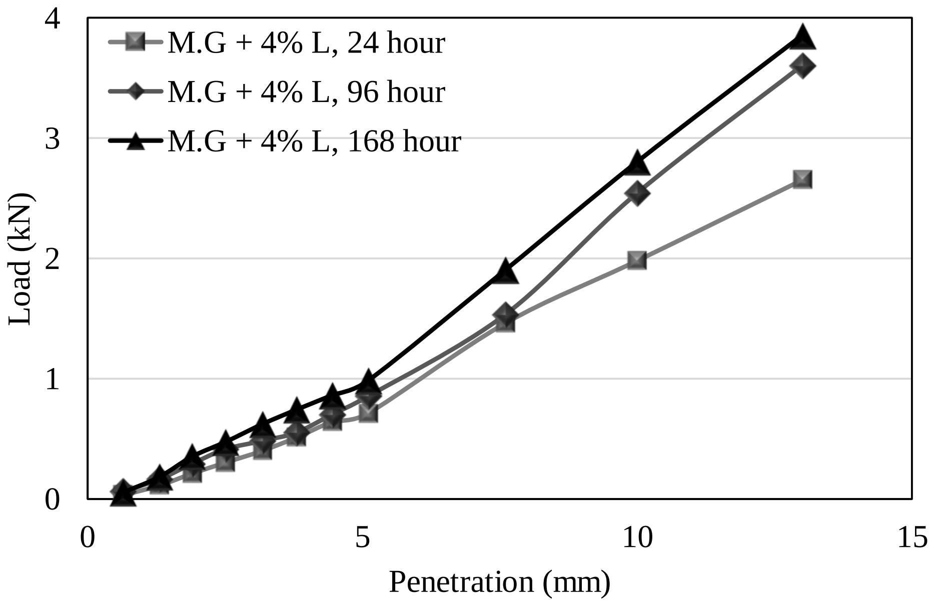

Figure 4. 19 Load penetration curve of M.G sample with 4% lime

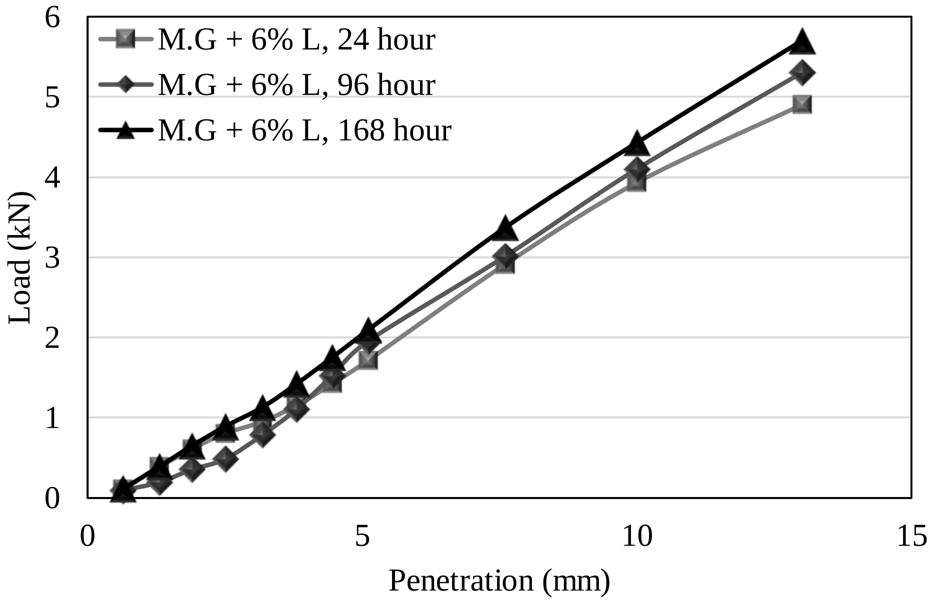

Figure 4. 20 Load penetration curve of M.G sample with 6% lime

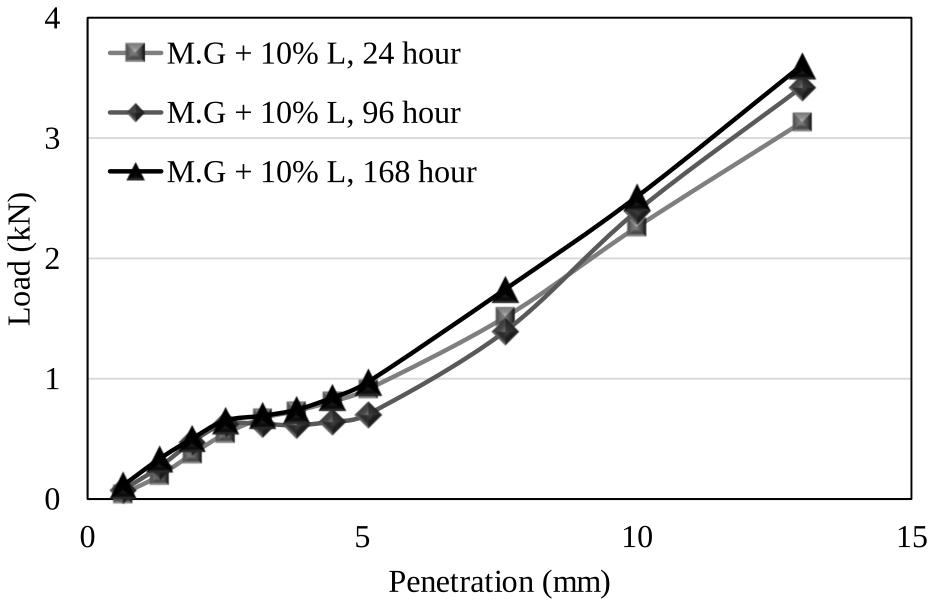

Figure 4. 21 Load penetration curve of M.G sample with 10% lime

Figure 4.22, 4.23, 4.24, and 4.25 show the load and penetration curves of fine grain alluvial deposits mixed with 2%, 4%, 6%, and 10% lime with deffierent curing times respectively.

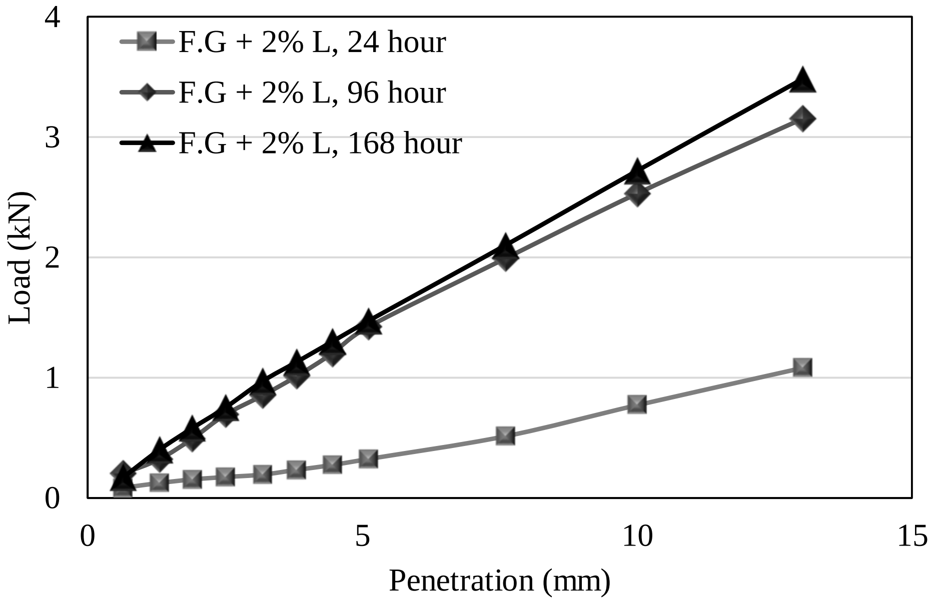

Figure 4. 22 Load penetration curve of F.G sample with 2% lime

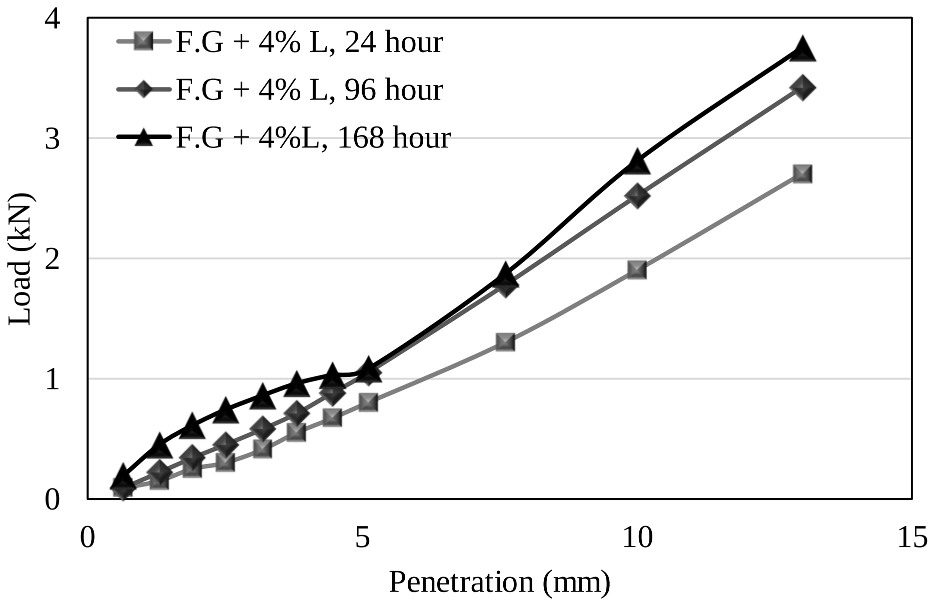

Figure 4. 23 Load penetration curve of F.G sample with 4% lime

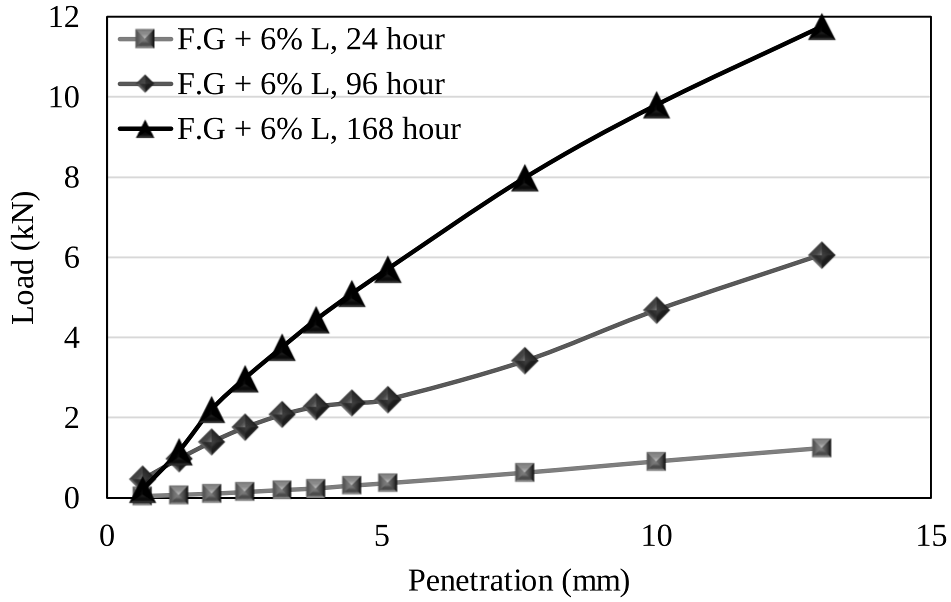

Figure 4. 24 Load penetration curve of F.G sample with 6% lime

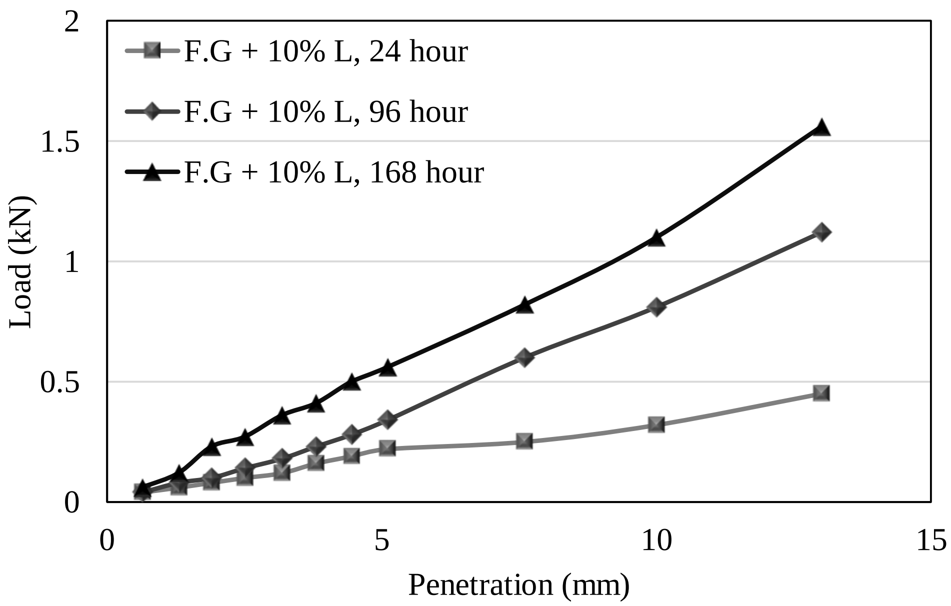

Figure 4. 25 Load penetration curve of F.G sample with 10% lime

After treating F.G, M.G, and C.G alluvial deposits with 2%, 4%, 6%, and 10% lime, the CBR value of samples increased. The maximum CBR value for the F.G sample was obtained in 6% lime mixture at 168 hour curing time as 27.77% which shows a significant increase in CBR value. For M.G samples the maximum CBR value was obtained as 10.16% in 6% lime mixture at 168 hour curing time. So, 6% of lime increased the CBR value of the M.G sample considerably. Since the CBR value of the untreated C.G sample was higher than F.G and M.G samples, the effect of lime on the CBR of C.G samples was not as high as F.G and M.G samples. The maximum value of CBR for C.G samples was 22.66% in 2% mixture of lime.

Figure 4.26, 4.27, and 4.28 show the effect of 2%, 4%, 6%, and 10% lime on CBR of the coarse, medium, and fine grain alluvial deposits in 24, 96, and 168 hours of curing time respectively. Table 4.8 lists the CBR test results of fine, medium, and coarse grain alluvial deposits mixed with 0%, 2%, 4%, 6%, and 10% lime.

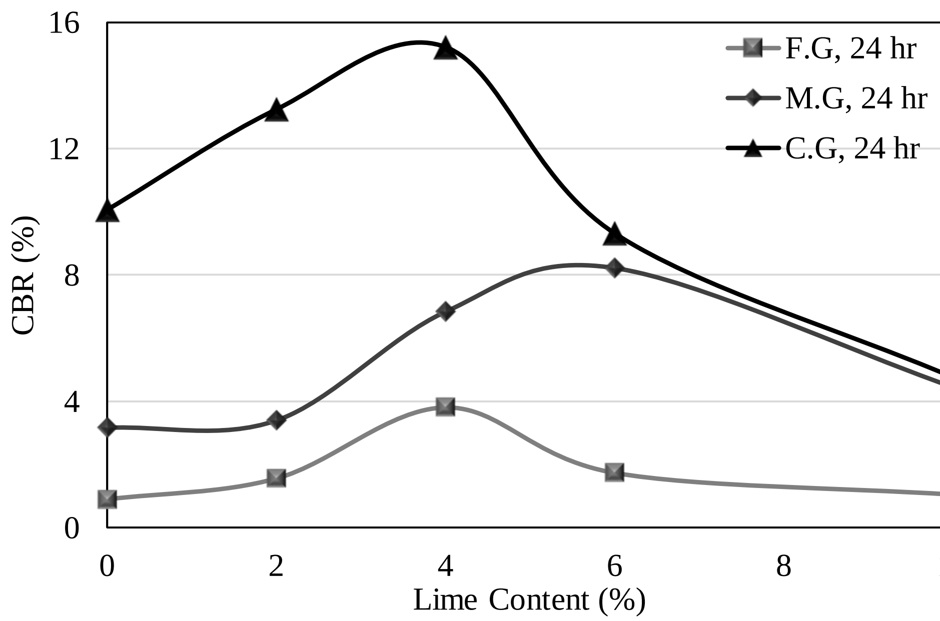

Figure 4. 26 Effect of 2%, 4%, 6%, and 10% lime on CBR of F.G, M.G, and C.G soils in 24 hour curing time

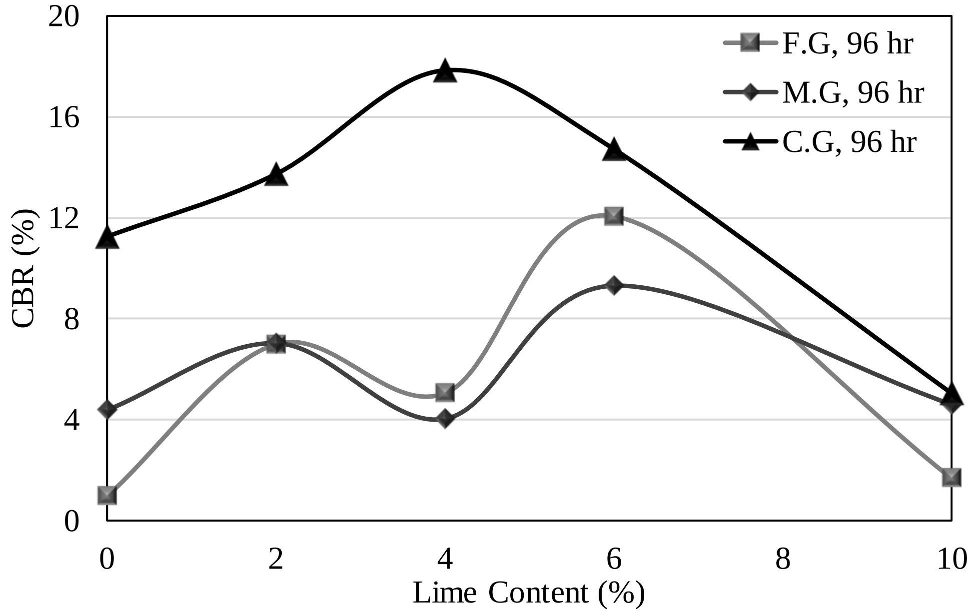

Figure 4. 27 Effect of 2%, 4%, 6%, and 10% lime on CBR of F.G, M.G, and C.G soils in 96 hour curing time

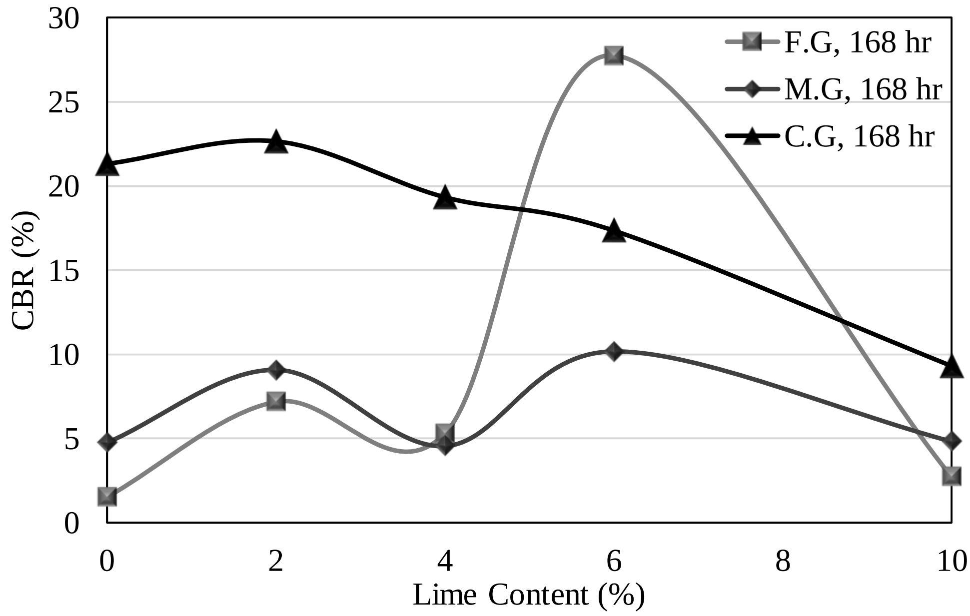

Figure 4. 28 Effect of 2%, 4%, 6%, and 10% lime on CBR of F.G, M.G, and C.G soils in 168 hour curing time

The effect of lime proportions on CBR of the coarse, medium, and fine grain alluvial soils in 24, 96, and 168 hour curing time is also shown in (Figure 4.29, 4.30, and 4.31) respectively.

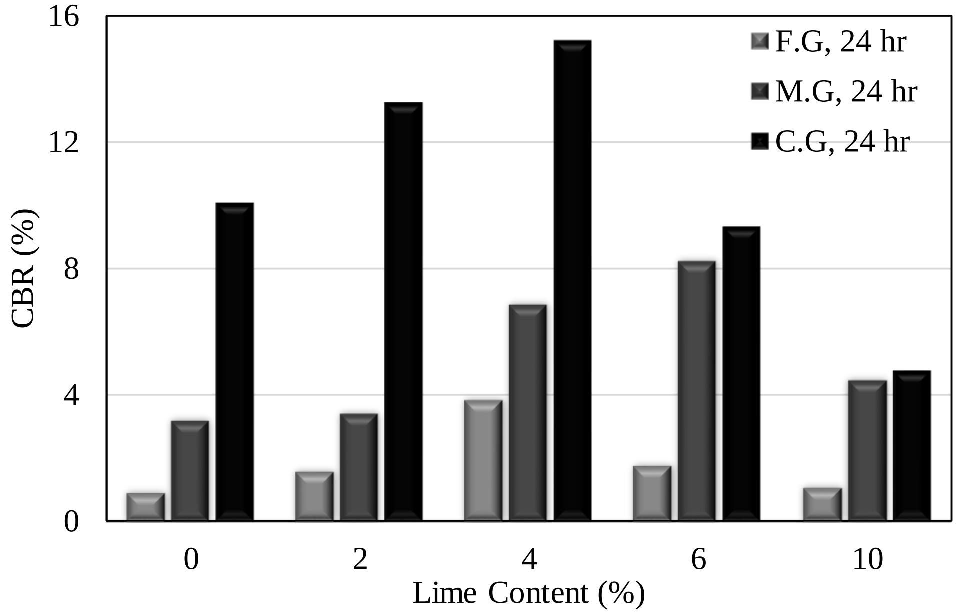

Figure 4. 29 The effect comparison of 2%, 4%, 6%, and 10% lime on CBR of F.G, M.G, and C.G soils in 24 hour curing time

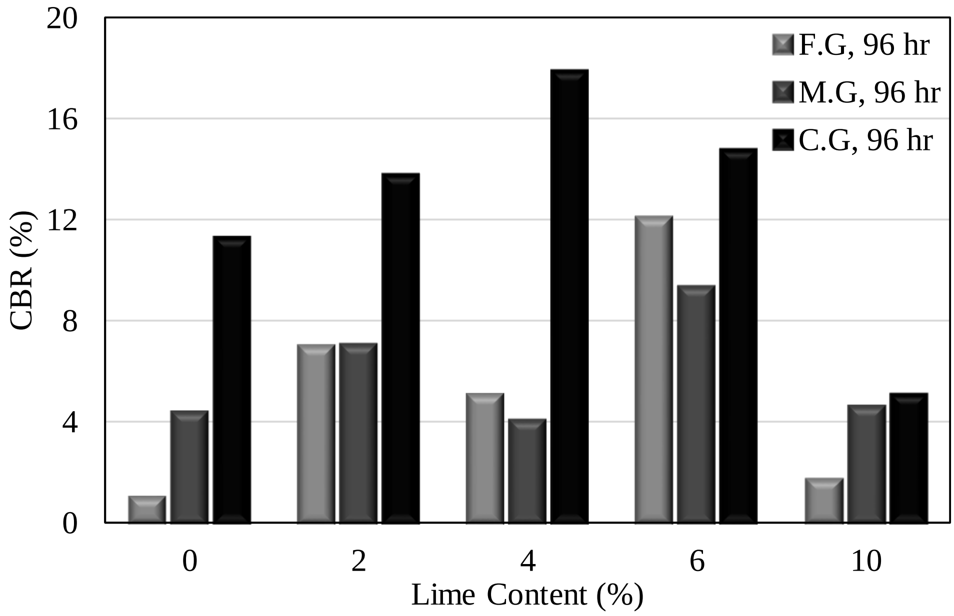

Figure 4. 30 The effect comparison of 2%, 4%, 6%, and 10% lime on CBR of F.G, M.G, and C.G soils in 96 hour curing time

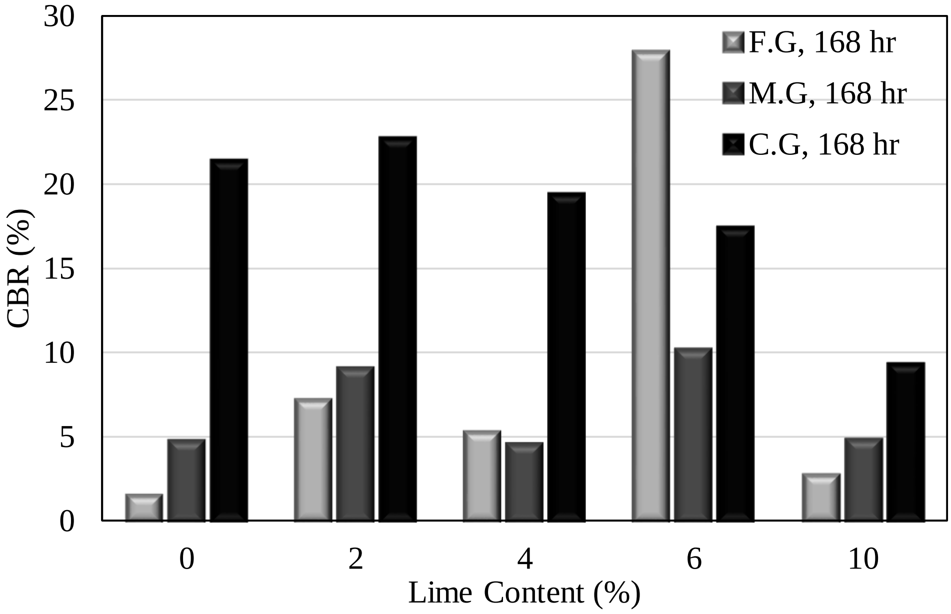

Figure 4. 31 The effect comparison of 2%, 4%, 6%, and 10% lime on CBR of F.G, M.G, and C.G soils in 168 hour curing time

CBR test results for stabilised soil samples indicates that the applying of lime considerably improved the bearing capacity (CBR) of F.G and M.G soils. The results are consistent with those reported by Noor and Uddin [70] and Ahmadi et al. [71]. Noor and Uddin [70], studied the impacts of lime stabilisation on the modification of the mechanical characteristics of the cohesive soil, CBR experiments were carried out on soil samples with a normal composition and also with a varying proportions of lime mixture. This can be stated that the CBR of natural composition has been achieved by 4.5%, CBR values are significantly increased up to 33% after stabilisation with both 2% and 4% lime.

Ahmadi et al. [71], examined the use of quick and hydrated lime in the stabilisation of lateritic soil, laboratory study was conducted to determine and compare the stabilisation efficacy of various proportions of quick and hydrated lime (0, 2.5, 5, 7.5, 10 percent) by applying separately to rigionally available lateritic soil. CBR experiment results indicate that the CBR value of the natural specimen was 8 percent and 2 percent for unsoaked and soaked states respectively. The addition of 6% hydrated lime strongly improved the CBR value of the soaked sample up to 43%.

Table 4.8 CBR test results of fine, medium, and coarse grain alluvial depostits mixed with 0%, 2%, 4%, 6%, and 10% lime

Curing Time	Lime Cont. (%)	CBR of F.G (%)	CBR of M.G (%)	CBR of C.G (%)
24 hour	0	0.89	3.17	10.06
	2	1.55	3.4	13.24
	4	3.8	6.84	15.22
	6	1.73	8.24	9.32
	10	1.04	4.46	4.76
96 hour	0	0.98	4.39	11.26
	2	6.99	7.04	13.74
	4	5.05	4.04	17.85
	6	12.05	9.32	14.73
	10	1.69	4.61	5.05
168 hour	0	1.53	4.76	21.32
	2	7.19	9.07	22.66
	4	5.3	4.56	19.34
	6	27.77	10.16	17.36
	10	2.73	4.83	9.32

4.4.3 CBR tests results of fly ash treated alluvial deposits

To examine the effect of fly ash on stabilization of alluvial deposits, CBR experiments were performed on coarse, medium, and fine grain alluvial deposits mixed with 10%, 15%, and 20% fly ash by dry weight of the soil. 24 and 168 hours were considered as curing time. Figure 4.32, 4.33, and 4.34 show the load-penetration curves of fine-grain alluvial soils treated with 10%, 15%, and 20% fly ash respectively. The addition of fly ash improved the CBR of F.G, M.G, and C.G specimens. The mixture of 15% FA improved significantly the CBR value of M.G and C.G samples.

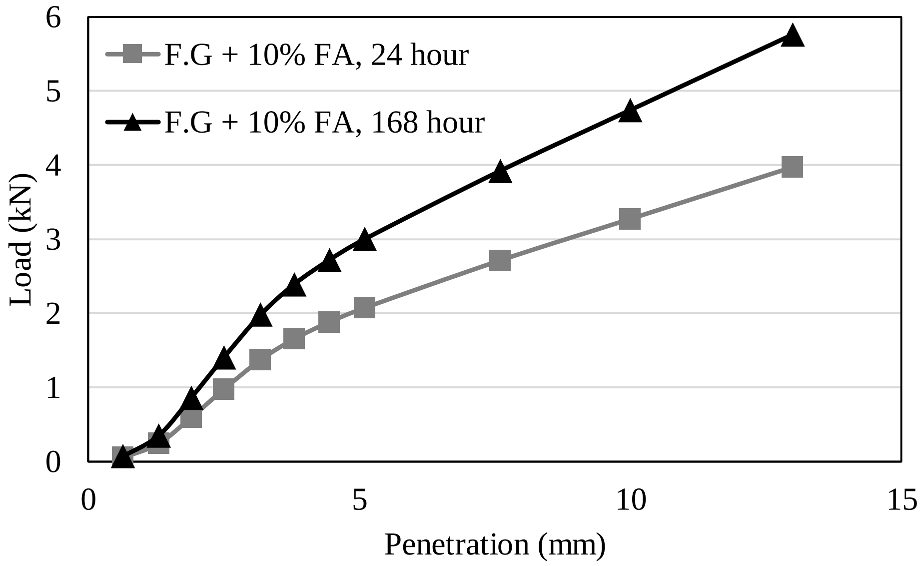

Figure 4. 32 Load penetration curves of F.G soil samples treated with 10% FA

CBR value of 15% mixture of FA of C.G sample was determined as 41.66% which is the maximum CBR value for C.G specimens of this study. The maximum CBR value for the M.G sample is obtained as 38.29% in 15% FA admixture. 25.37% CBR value was obtained in 20% of FA admixture as maximum CBR value for F.G sample.

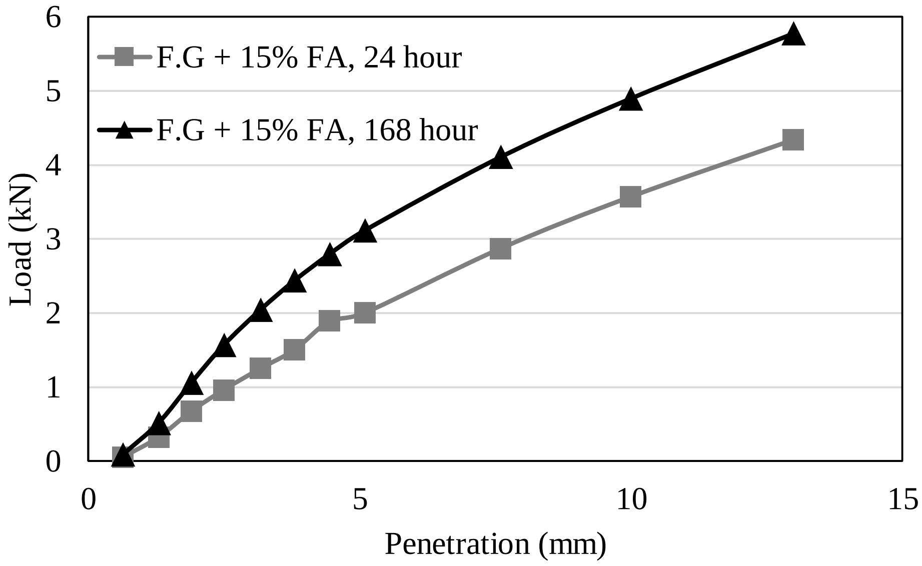

Figure 4. 33 Load penetration curves of F.G soil samples treated with 15% FA

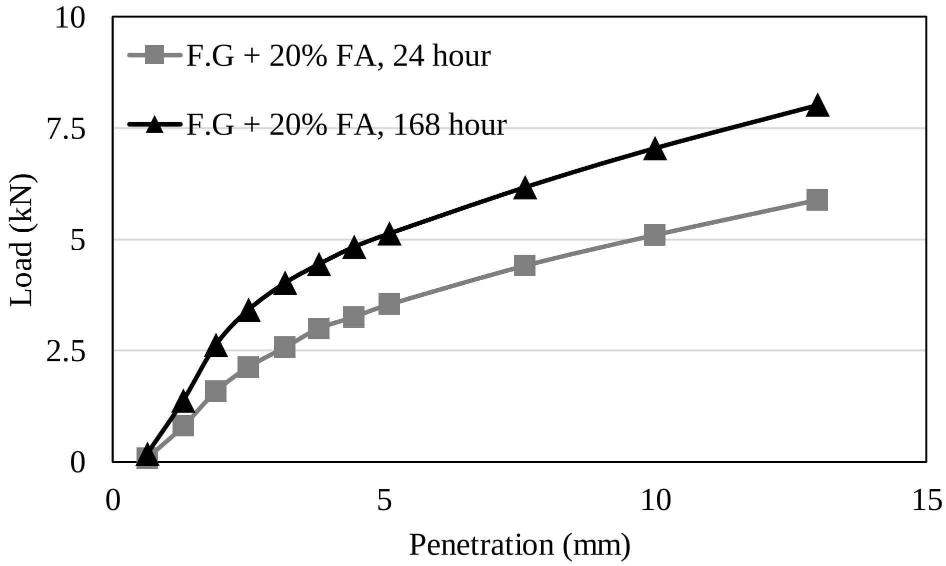

Figure 4. 34 Load penetration curves of F.G soil samples treated with 20% FA

Figure 4.35, 4.36, and 4.37 show the load-penetration curves of medium-grain alluvial soils treated with 10%, 15%, and 20% fly ash respectively.

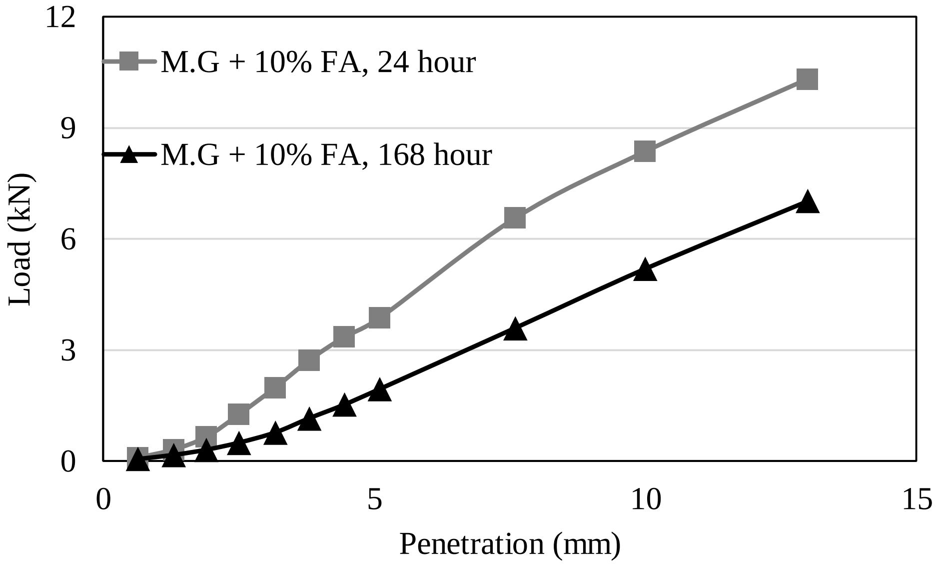

Figure 4. 35 Load penetration curves of M.G soil samples treated with 10% FA

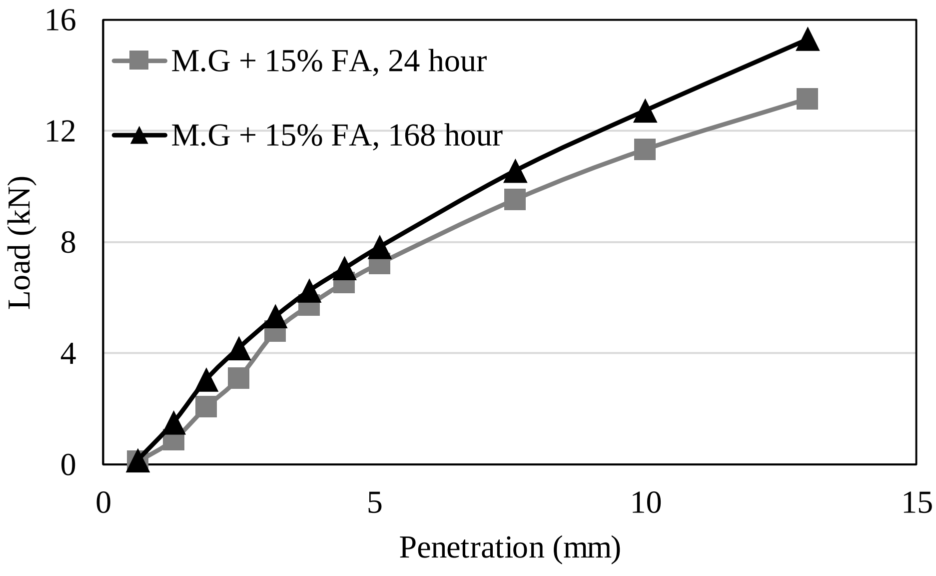

Figure 4. 36 Load penetration curves of M.G soil samples treated with 15% FA

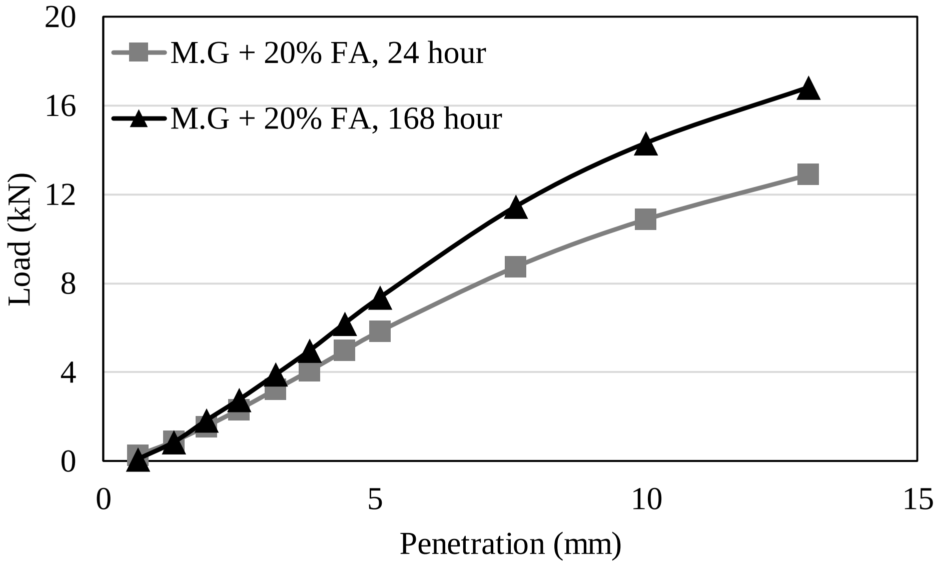

Figure 4. 37 Load penetration curves of M.G soil samples treated with 20% FA

Figure 4.38, 4.39, and 4.40 show the load-penetration curves of coarse-grain alluvial soils treated with 10%, 15%, and 20% fly ash respectively.

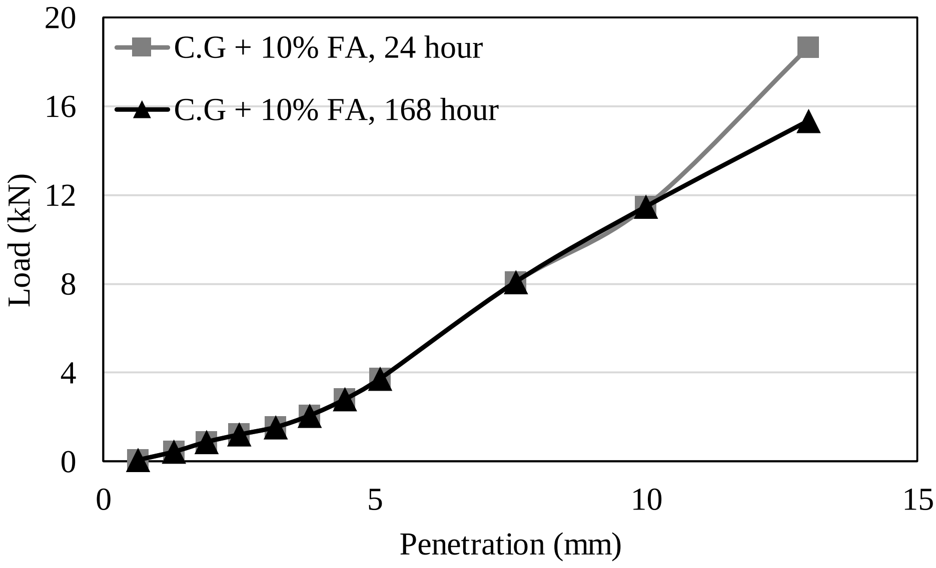

Figure 4. 38 Load penetration curves of C.G soil samples treated with 10% FA

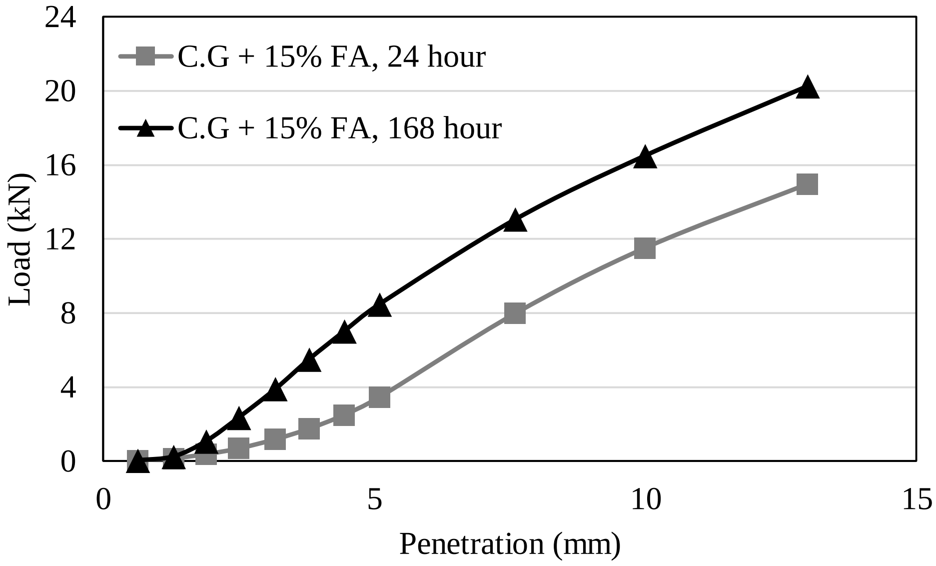

Figure 4. 39 Load penetration curves of C.G soil samples treated with 15% FA

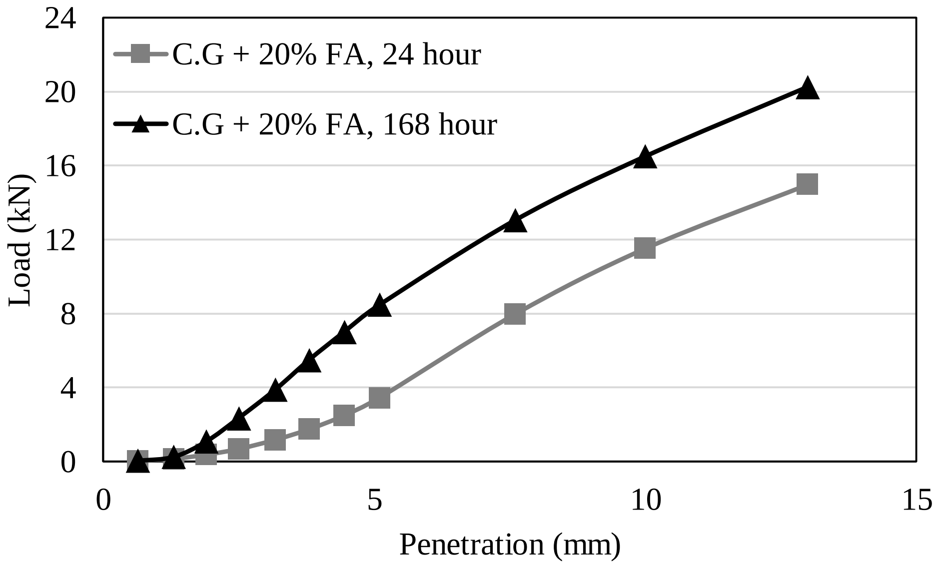

Figure 4. 40 Load penetration curves of C.G soil samples treated with 20% FA

Figure 4.41 and 4.42 show the effect of 10%, 15%, and 20% fly ash on CBR value of fine, medium, and coarse alluvial deposits with 24 and 168 hours of curing time respectively. Figure 4.43 and 4.44 show the comparison between CBR value of fine, medium, and coarse alluvial deposits treated with 10%, 15%, and 20% fly ash in 24 and 168 hours of curing time respectively. All CBR values for fine, medium, and coarse alluvial deposits treated with 10%, 15%, and 20% fly ash are listed in Table 4.9.

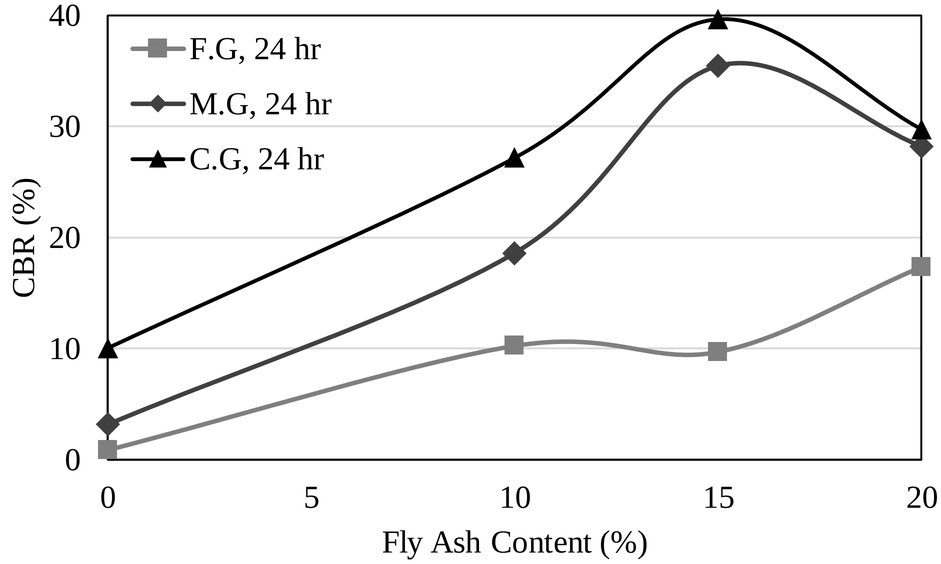

Figure 4. 41 Effect of 10%, 15%, and 20% fly ash on CBR value of F.G, M.G, and C.G alluvial soils in 24 hour curing time

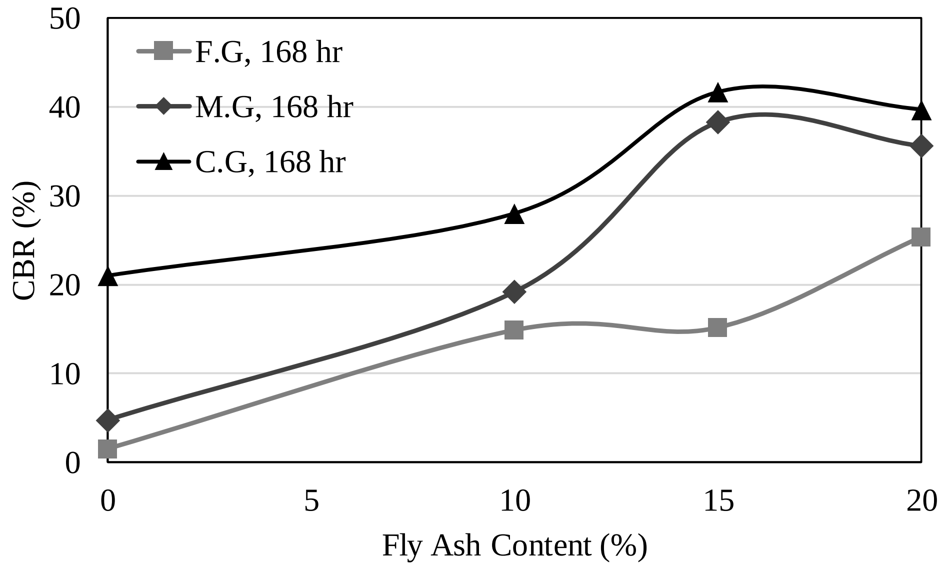

Figure 4. 42 Effect of 10%, 15%, and 20% fly ash on CBR value of F.G, M.G, and C.G alluvial soils in 168 hour curing time

The findings obtained are in excellent accordance with those reported by Edil et al. [62] and Firat et al. [72].

Edil et al. [62] investigated the stabilization of fine-grained soils with fly ash. The focus of this research was to assess the efficacy of self-cementing fly ash extracted from the combustion of sub-bituminous coal for fine-grained soil stabilisation at electric power plants. CBR and resilient modulus (Mr) experiments were performed on mixtures prepared with 7 soft fine-grained soils and four fly ashes. As a result, the presence of fly ash resulted in a significant increase in CBR and Mr. CBRs of the untreated soils varried between 1% and 5%. The implementation of 10% fly ash contributed in CBRs ranging from 8% to 17% and 18% fly ash contributed in CBRs ranging from 15% to 31% .

Firat et al. [72], examined the use of marble dust, fly ash and waste sand (silt quartz) in highway subbase filling materials. 0 percent, 5 percent, 10 percent, 15 percent, 20 percent of fly ash, marble dust, and waste sand is supplemented by two types of natural soils. Experiments were run for normal compaction, permeability, and saturated California Bearing Ratio (CBR). A soaked CBR test was conducted in this study to assess the soil's bearing capacity under severe situations. CBR of untreated soils ranged between 7% and 11%. After increasing fly ash content to 15%, the CBR value considerably improves, and it ranges between 25% and 51%.

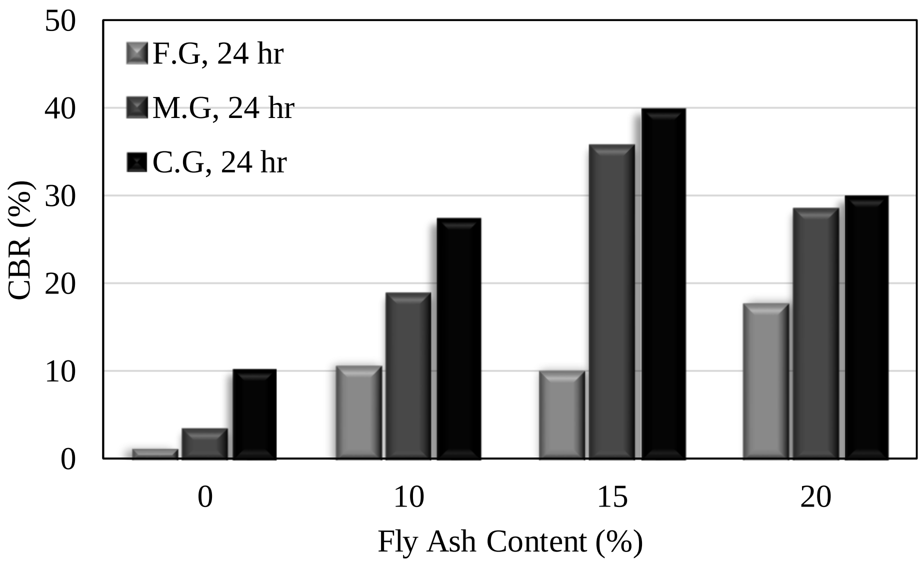

Figure 4. 43 The Effect comparison of 10%, 15%, and 20% FA on CBR of F.G, M.G, and C.G soils in 24 hour curing time

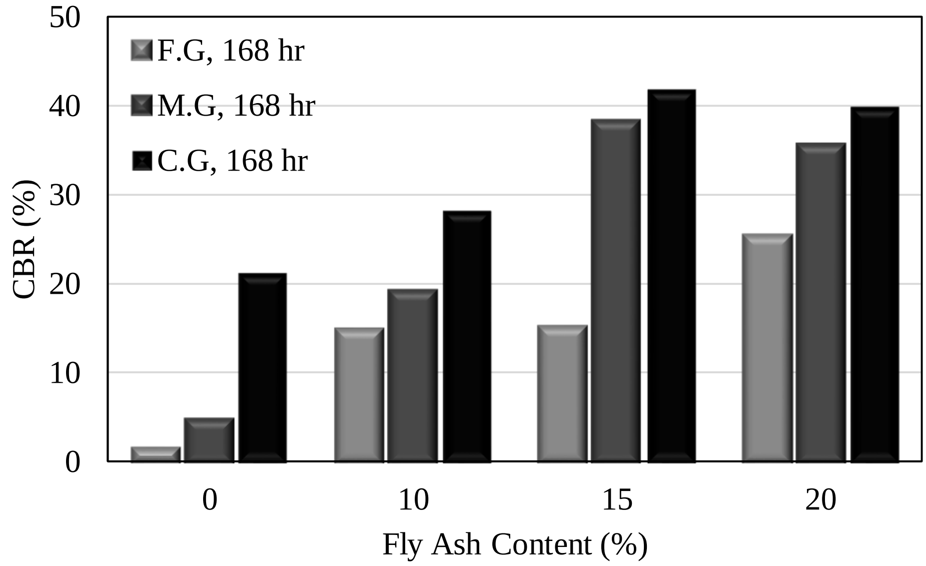

Figure 4. 44 The Effect comparison of 10%, 15%, and 20% FA on CBR of F.G, M.G, and C.G soils in 168 hour curing time

Table 4. 9 CBR values for fine, medium, and coarse alluvial deposits treated with 10%, 15%, and 20% fly ash

Curing Time	Fly Ash cont. (%)	CBR of F.G (%)	CBR of M.G (%)	CBR of C.G (%)
24 hour	0	0.89	3.17	10.06
	10	10.26	18.6	27.2
	15	9.72	35.46	39.68
	20	17.36	28.22	29.76
168 hour	0	1.53	4.76	21.02
	10	14.88	19.19	28.02
	15	15.17	38.29	41.66
	20	25.37	35.61	39.68

5. CONCLUSION AND RECOMMENDATION

The stable and desired conditions for the construction of highway fillings are high bearing capacity, low settlement, low void ratio, and low plasticity. Alluvial deposits are problematic soils because of their low bearing capacity, high organic matter content, high void ratio, so they do not meet the desired condition for the construction of highway fillings. Also, the modification of the engineering properties of alluvial soils is very important for geotechnical engineers in road construction, particularly in urban areas, as borrowing materials are becoming less and less available and very expensive for the foundation soils.

In 1904, as a cost-effective and "environmentally-friendly" method, the process of soil stabilization was introduced in the United State. Chemical soil stabilization is accomplished by applying stabilizers to the soil, mixing soil particles to accomplish the main aims of improving the geotechnical efficiency of the soil [14], [48]. It is possible to improve the undesirable mechanical and chemical characteristics of the soil by adding binders or by-products such as cement, lime, fly ash, and bottom ash to the soil [13], [20]. Mostly, stabilization of lime and fly ash leads to improvements in soil compactibility, compressibility, and bearing capacity through a set of chemical, mineralogical, and microstructural changes in the original soil properties. Cation exchange, flocculation, agglomeration, and pozzolanic interactions are enabled through the use of lime/fly ash, leading to an increase in the size of soil particles [4].

To accurately study the engineering properties, compaction properties, and bearing capacity of composite alluvial soil samples, this research was carried out in two different stages. The first stage was to investigate the engineering index properties and compaction properties of three different grain size alluvial deposits. The second stage was to examine the effect of lime and fly ash on the bearing capacity of fine, medium, and coarse grain alluvial deposits.

To classify the used alluvial deposit, liquid limit, plastic limit, and plasticity index of soil were determined as 38.3%, 25.9%, and 12.4% respectively. According to the plasticity chart, the soil is determined as intermediate plasticity silt (MI).

To determine the specific gravity of alluvial soil, lime, and FA, specific gravity tests were conducted. The specific gravity of fine, medium, and coarse grain alluvial deposite was found as 2.68, 2.67, and 2.66 respectively. The specific gravity of lime and FA were determined as 2.61 and 2.3 respectively.

The results of compaction tests show that the maximum dry unit weight of coarse, medium, and fine grain samples are 19.55 (kN/m^3), 19.35 (kN/m^3), and 16.9 (kN/m^3) respectively.

To check the effect of FA on compaction behavior of alluvial soil, 10%, 15%, and 20% FA by dry weight of soil was mixed and compaction tests were conducted. By increasing the content of FA, maximum dry unit weight and optimum moisture content decreased.

To determine the bearing capacity of untreated alluvial deposits, CBR tests were conducted. Coarse grain soil in 168 hour curing time with 21% CBR value has got the maximum CBR for untreated soil samples. Fine-grain sample in 24 hour curing time with 0.89% CBR was classified as minimum CBR value for untreated soil samples.

To determine the effect of lime on the bearing capacity of alluvial deposits, all samples were treated with 2%, 4%, 6%, and 10% lime. The maximum CBR value for fine-grain soil was obtained at 6% lime in 168 hour curing time as 27.77%. for medium grain soil, the maximum CBR value was 10.16% at 6% lime. The effect of lime on CBR coarse-grain was not as high as fine and medium grain samples. The maximum value of CBR for coarse grain soil was 22.66% at 2% lime mixture.

To study the effect of FA on CBR of different grain size alluvial deposits, 10%, 15%, and 20% FA by dry weight of soil was mixed with soil and CBR tests were performed. FA was more effective on medium and coarse grain alluvial soils than fine-grain soil.

İn the mixture of 15% FA with soil, the CBR of coarse grain sample was obtained as 41.66% which was the maximum CBR value for coarse-grain samples. The maximum value for the medium-grain sample was obtained as 38.29% in 15% FA mixture.

25.37% CBR was obtained in 20% FA mixture as the maximum CBR value for fine-grain alluvial deposit.

The following can be inferred in conclusion:

1. As was noted in several laboratory experiments, fine grain alluvial soil has got the minimum CBR value. Treating fine-grain alluvial soil with 6% lime improves the CBR of this soil significantly.

2. Coarse and medium grain alluvial deposits have acceptable CBR values which make these grain size soils suitable for highway fillings. Treating medium and coarse grain alluvial deposits with 6% and 4% lime improves the bearing capacity of these soils.

3. FA is more effective than lime for improving the bearing capacity of medium and coarse grain alluvial deposits. Medium and coarse grain samples treated with 15% FA were improved significantly. 20% FA maximized the CBR of fine grain alluvial soil.

4. Curing time effects the CBR value of all grain size alluvial soils. Increasing in curing time, increased the CBR value of fine, medium, and coarse grain alluvial soils.

5. Particle size has a significant effect on the bearing capacity of soils. Coarse and medium grain size samples have much higher CBR values than fine grain soils.

The use of chemical stabilization as an economical and environmentally sustainable technique in geotechnical projects will continue. The financial aspects and possible environmental side effects of building projects could be minimized by understanding the key chemical components of stabilizers used to boost the geotechnical efficiency of the soil.

This research study recommends that medium and coarse grain alluvial deposits can be used as highway fillings. Since fine-grain alluvial soils have a low bearing capacity, they should be stabilized either by lime of fly ash. Also, this research study suggests that stabilizing alluvial deposits with fly ash is more sufficient and cost effective, so, using fly ash in highway filling stabilization will be helpful for Turkey’s economy, and will help to have a cleaner environment.

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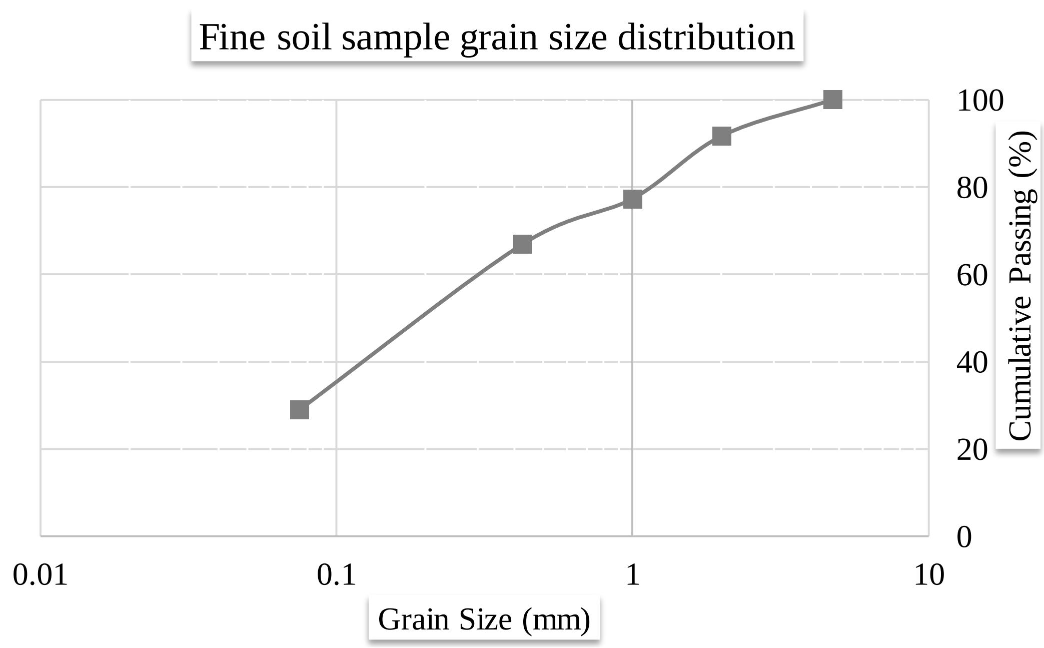
APPENDIX

Appendix 1. Washed sieve analysis results

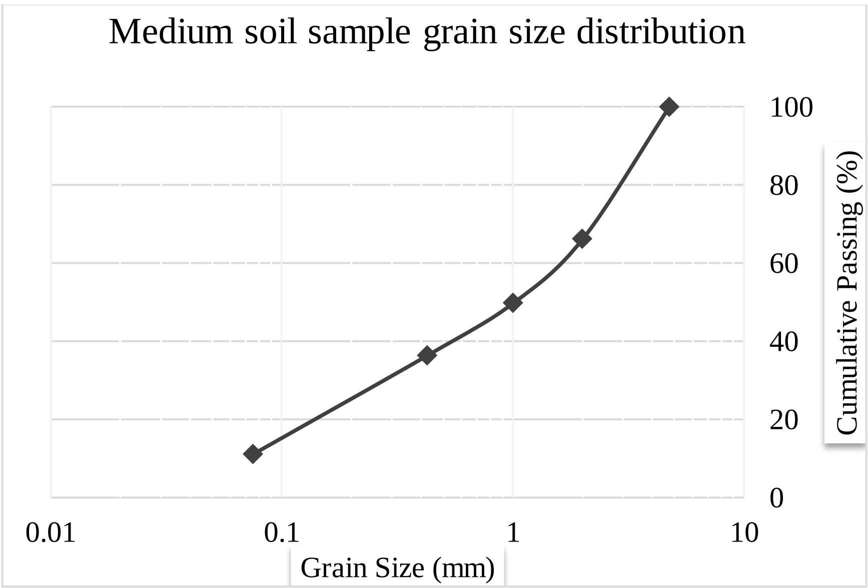
F.G sample grain size distribution

sieve No.	sieve size (mm)	retained weight (gr)	retained cumulative weight (gr)	Retained cumulative (%)	Passing cumulative (%)
No.4	4.75	0	0	0	100
No.10	2	41.55	41.55	8.310166203	91.6898338
No.18	1	71.79	113.34	22.66845337	77.33154663
No.40	0.425	52.05	165.39	33.07866157	66.92133843
No.200	0.075	189.4	354.79	70.95941919	29.04058081
Pan	0	145.2	499.99	100	0
		499.99			



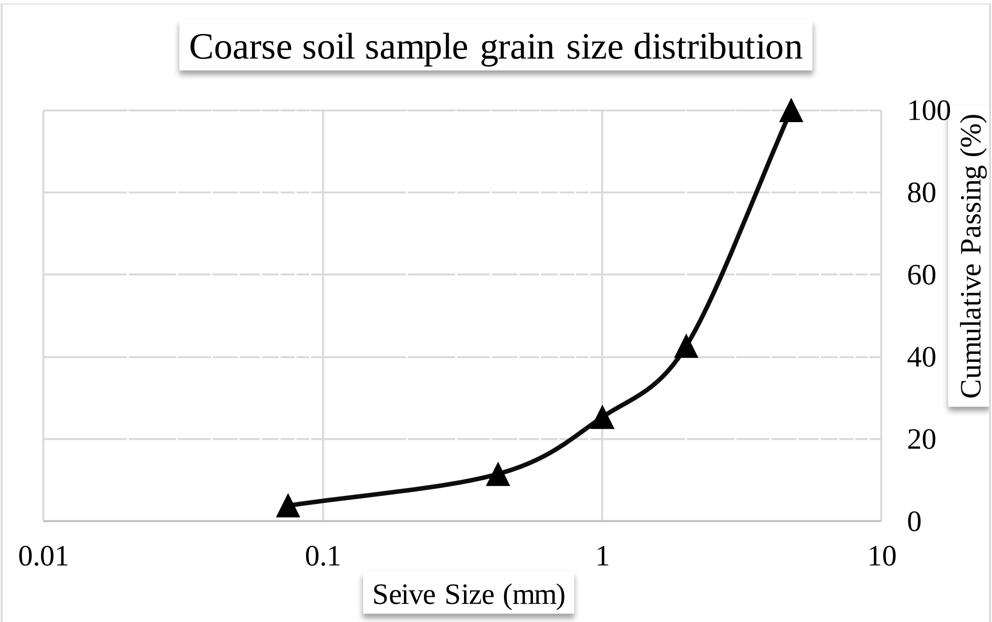
M.G sample grain size distribution

Sieve No.	Sieve size (mm)	Retained weight (gr)	retained cumulative weight (gr)	Retained cumulative (%)	Passing cumulative (%)
No.4	4.75	0	0	0	100
No.10	2	169.2	169.2	33.84	66.16
No.18	1	82.1	251.3	50.26	49.74
No.40	0.425	66.9	318.2	63.64	36.36
No.200	0.075	126.174	444.374	88.8748	11.1252
Pan	0	55.626	500	100	0
		500			



C.G sample grain size distribution

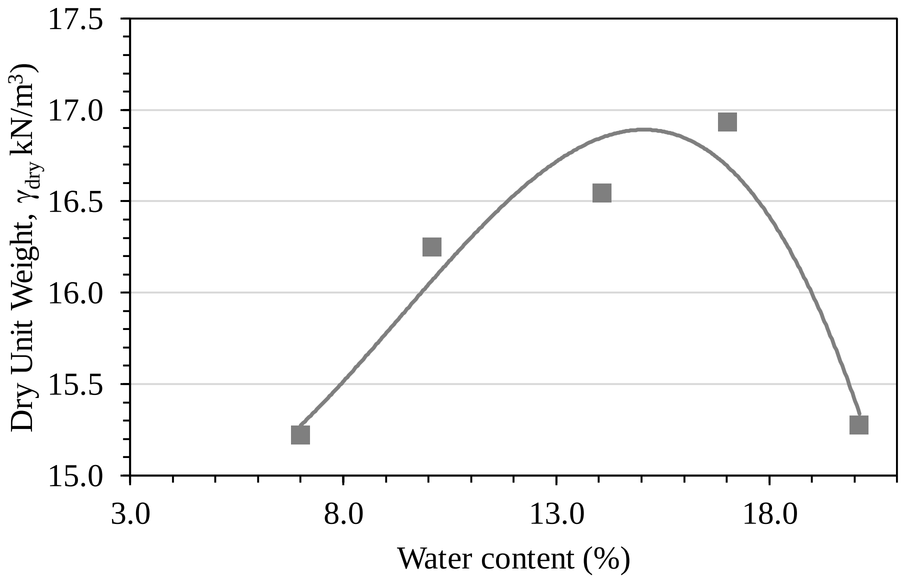
Sieve No.	Sieve size (mm)	Retained wieght (gr)	Retained cumulative wieght (gr)	Retained cumulative (%)	Passing cumulative (%)
No.4	4.75	0	0	0	100
No.10	2	286.93	286.93	57.386	42.614
No.18	1	86.46	373.39	74.678	25.322
No.40	0.425	68.9	442.29	88.458	11.542
No.200	0.075	38.67	480.96	96.192	3.808
Pan	0	19.04	500	100	0
		500			



Appendix 2. Compaction test results

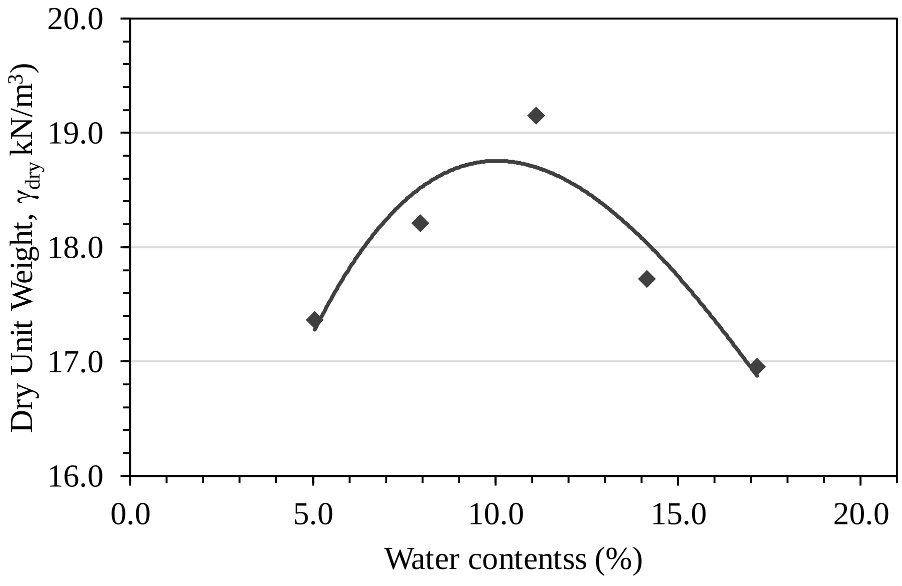
Compaction result of F.G sample

Exp Set	Mold+ soil (gr)	Mold W (gr)	Mold V (cm^3)	Unit Weight (kN/m^3)	Can+Wet (gr)	Can+ Dry (gr)	Water cont. (%)	Dry Unit Weight (kN/m^3)
1	5843	4288	936.75	16.28	69.85	67.28	7.0	15.22
2	5996	4288	936.75	17.89	70.36	65.83	10.1	16.25
3	6090	4288	936.75	18.87	65.10	60.52	14.1	16.54
4	6180	4288	936.75	19.81	78.60	71.67	17.0	16.93
5	6040	4288	936.75	18.35	76.30	68.06	20.1	15.27



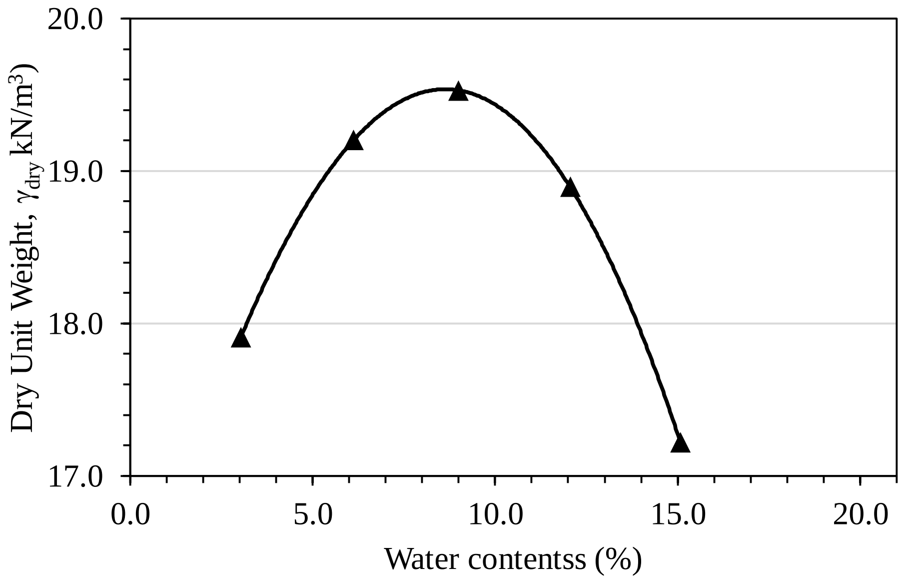
Compaction result of M.G sample

Exp Set	Mold+ soil (gr)	Mold W (gr)	Mold V (cm^3)	Unit Weight (kN/m^3)	Can+ Wet (gr)	Can+ Dry (gr)	Water content (%)	Dry Unit Weight (kN/m^3)
1	6030	4288	936.75	18.24	74.45	72.3	5.1	17.36
2	6165	4288	936.75	19.66	77.22	73.8	7.9	18.21
3	6320	4288	936.75	21.28	81.82	76.7	11.1	19.15
4	6220	4288	936.75	20.23	91.62	84.1	14.2	17.72
5	6185	4288	936.75	19.87	65.47	59.8	17.2	16.96



Compaction result of C.G sample

Exp Set	Mold+ soil (gr)	Mold W (gr)	Mold V (cm^3)	Unit Weight (kN/m^3)	Can+ Wet (gr)	Can+ Dry (gr)	water content (%)	Dry Unit Weight (kN/m^3)
1	6050	4288	936.75	18.45	69.10	67.97	3.0	17.91
2	6234	4288	936.75	20.38	87.90	84.62	6.1	19.20
3	6320	4288	936.75	21.28	72.00	68.37	9.0	19.53
4	6310	4288	936.75	21.18	66.55	62.25	12.1	18.90
5	6180	4288	936.75	19.81	101.20	90.68	15.1	17.22



Appendix 3. liquid and plastic limit results

Casagrande liquid limit test result

No.	Blow No.	Con No.	W of con (gr)	W of Con+ wet (gr)	W of Can+ dry (gr)	W_c (%)
1	16	50	25.99	41.15	36.82	39.9
2	22	35	20.89	36.44	32.14	38.2
3	30	29	27.97	40.74	37.27	37.3
4	42	25	30.93	44.43	40.8	36.7

Plastic limit test result

No.	Con. No.	W_1	W_2	W_3	Water Content (%)	Plastic Limit
1	16	15.36	22.63	21.13	25.9965	25.9 (%)
2	17	15.02	22.74	21.14	26.1437	
3	86	11.29	18.75	17.26	24.9581	
4	25	11.6	19.60	17.91	26.78	

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