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APPLICATIONS OF SOME GROUND IMPROVEMENT TECHNIQUES IN VARIOUS SANDS

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IN
CIVIL ENGINEERING

BY
Nurullah AKBULUT
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Applications of Some Ground Improvement Techniques in Various Sands

M.Sc. Thesis
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Supervisor
Assoc. Prof. Dr. Ali Fırat ÇABALAR

By
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August 2013

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Nurullah AKBULUT

ABSTRACT

APPLICATIONS OF SOME GROUND IMPROVEMENT TECHNIQUES IN VARIOUS SANDS

AKBULUT, Nurullah
M.Sc. in Civil Engineering
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This thesis presents an extensive experimental investigation on the use of some ground improvement applications in various sands. In recent years, investigations in biotreatment, fly ash, and waste tire use have enabled significant advances in understanding the effect of microorganisms and waste materials in the development of soil behaviour. Increase in the amount of waste materials (i.e., fly ash and waste tires) results in an environmental problem in Turkey, as well as in many regions of the world. Fly ash and waste tire applications in Turkey are not widely employed, although it is effectively used in some countries, and thereby decreases the possible effect on the environment. Some applications, in which waste tires and fly ash are used, are lightweight fill material in embankments, insulation layer, and drainage applications. The compressive strength and bearing capacity of biotreated specimens were compared to those of the untreated specimens. Behaviour of a sand with fly ash- lime mixtures were systematically characterized in proctor, permeability, oedometer, and CBR testing apparatuses. A series of laboratory investigation of a sand and processed waste tires (i.e., tire crumb, tire buffing) mixtures at various contents was performed oedometer, permeability and direct shear tests. Additionally, Scanning Electron Microscope (SEM) pictures were taken to examine the microstructure of the specimens. From the experimental results, it was observed that the mechanical properties of the soils tested have been affected by biotreatment, fly ash, and waste tire inclusion.

Keywords: Biotreatment, fly ash, waste tire, sand

ÖZET

ÇEŞİTLİ KUMLARDA ZEMİN İYİLEŞTİRME UYGULAMALARI

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Tez Yöneticisi: Doç. Dr. Ali Fırat ÇABALAR
Ağustos 2013, 74 sayfa

Bu çalışmada, çeşitli kum örnekleri üzerinde yapılan kapsamlı bir deneysel araştırma sunulmaktadır. Son yıllarda, mikrobiyoloji, uçucu kül ve atık lastik kullanımı; mikroorganizmalar ve atık maddelerin zemin davranış gelişiminde etkisini anlamada önemli gelişmeler sağlamıştır. Atık maddelerin (uçucu kül ve lastik) miktarındaki artış, hem Türkiye'de, hem de Dünya'nın birçok bölgesinde bir çevre sorunu ile sonuçlanmaktadır. Uçucu kül ve atık lastik uygulamaları Türkiye'deki mühendislik uygulamalarında yaygın olarak kullanılmamasına rağmen bazı ülkelerde etkili olarak kullanılmakta ve böylece çevre üzerindeki olumsuz etkisi azaltılmaktadır. Atık lastik ve uçucu külün kullanılabileceği bazı uygulamalar; hafif dolgu malzemesi, yalıtım tabakası, ve drenaj uygulamalarıdır. Biyolojik metodla iyileştirilen örneklerin taşıma oranı değerleri incelenmiştir. Uçucu kül kullanılan örneklerde; kum ile uçucu kül-kireç karışımlarının bazı davranışları proktor, geçirgenlik, ödometre ve CBR test cihazlarında tanımlanmaktadır. Farklı oranlarındaki atık lastik (granüler, kıprıntı) ve kum örnekleri üzerinde ödometre, geçirgenlik ve kesme kutusu deneyleri yapılmıştır. Test edilen zeminlerin mikro yapısını incelemek üzere Taramalı Elektron Mikroskobu (TEM) görüntüleri alınmıştır. Biyolojik metodla iyileştirilen, uçucu kül, ya da atık lastik eklenen zeminlerin bazı mekanik özelliklerinin etkilendiği gözlemlenmiştir.

Anahtar Kelimeler: Biyolojik iyileştirme, uçucu kül, atık lastik, kum

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LIST OF SYMBOLS

A	Area (cm^2, m^2)
ASTM	American Society for Testing and Materials
ÇATES	Çatalağzı Thermal Power Station
CBR	California Bearing Ratio
CSS	Crushed Stone Sand
CS	Control Sample
CS4	4 days cured Control Sample
CS8	8 days cured Control Sample
CS16	16 days cured Control Sample
CS32	32 days cured Control Sample
C_c	Coefficient of Curvature
C_u	Coefficient of Uniformity
D_{10}	Effective Grain Size
D_{30}	Particle size at 30% finer
D_{50}	Median Grain Size
D_{60}	Particle size at 60% finer
e	Void Ratio
e_{max}	Maximum Void Ratio

e_{min}	Minimum Void Ratio
e_s	Intergranular Void Ratio
e_{s0}	Initial Intergranular Void Ratio
e_{sf}	Final Intergranular Void Ratio
FA	Fly Ash
FAP	Fly Ash Pellets
FC	Fines Content
FC_t	Transition of Fines Content
G_s	Specific Gravity
k	Coefficient of Permeability (cm/sec)
LBS	Leighton Buzzard Sand
NS	Narli Sand
q	Water Flow (cm^3/sec)
R_d	Relative Density
SEM	Scanning Electron Microscope
SP	Poorly Graded Sand
SW	Well-Graded Sand
TB	Tire Buffing
TC	Tire Crumb
TS	Trakya Sand
t	Time

UCS	Unconfined Compression Strength
USBR	United States Bureau of Reclamation
USCS	Unified Soil Classification System
w_{opt}	Optimum Water Content (%)
XG	Xanthan Gum
1XG4	4 days cured 1% Xanthan Gum
1XG8	8 days cured 1% Xanthan Gum
1XG16	16 days cured 1% Xanthan Gum
1XG32	32 days cured 1% Xanthan Gum
3XG4	4 days cured 3% Xanthan Gum
3XG8	8 days cured 3% Xanthan Gum
3XG16	16 days cured 3% Xanthan Gum
3XG32	32 days cured 3% Xanthan Gum
5XG4	4 days cured 5% Xanthan Gum
5XG8	8 days cured 5% Xanthan Gum
5XG16	16 days cured 5% Xanthan Gum
5XG32	32 days cured 5% Xanthan Gum
ϕ	Angle of Friction (degree)
γ_{dry}	Dry Unit Weight (kN/m^3)
γ_{drymax}	Maximum Dry Unit Weight (kN/m^3)

CHAPTER I

INTRODUCTION

1.1 General

Over the last century, ground improvement techniques, which are rising rapidly as worldwide development for the utilization of unstable or soft soils, have been developed and nowadays many of them are widely used in geotechnical engineering projects. Considerable developments have occurred since the twentieth century, not only in technical matters but also in equipment and rate of production. Ground improvement techniques continue to make considerable progress, both quantitatively and qualitatively, as a result of not only technology developments but also of an increasing awareness of the environmental and economic advantages of modern soil improvement methods. The selection of the correct ground improvement technique at an early stage in design can have an important effect on foundation choice and can often lead to more economical solutions when compared to traditional approaches. Various ground improvement techniques can be characterized as the modification of existing soils or earth structures to provide better performance under design and/or operational conditions (Hausmann, 1990). It is proper to categorize the wide sort of soil improvement techniques in the following ways: densification, consolidation/ dewatering, chemical additives, heating/freezing (Cabalar et al., 2009). Kamon and Bergado (1991) classified commonly used ground improvement techniques by soil type into four categories, as shown in Figure 1.1. The basic principles of these methods have not changed since the early of this century. The main goal of most soil treatment methods whose basic principles has been not changed since the past of mankind (Van Impe, 1989; Terashi & Juran, 2000) is to improve the soil characteristics that match the desired results of a project, such as an increase in density and shear strength to aid problems of stability, the reduction of soil compressibility, control shrinking and swelling, reduce susceptibility to liquefaction, influencing permeability to reduce and control ground water flow or to increase the rate of consolidation, or to improve soil homogeneity. Many techniques for ground improvement are available, compaction (i.e. heavy tamping) and dewatering (i.e. lowering the water level via drains) involve only work on soil, chemical admixture (i.e. grouting) and reinforcement (i.e. geosynthetic sheets, soil nailing) techniques require the use of additional materials as inputs into the process.

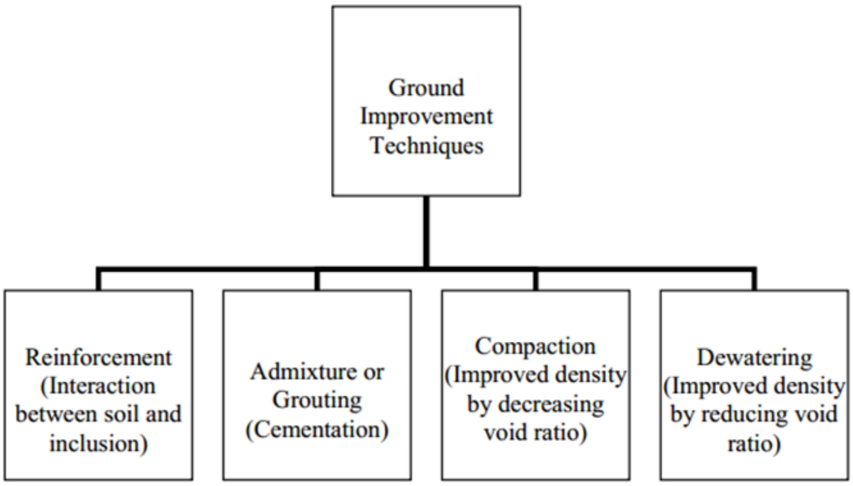


Figure 1.1 Ground Improvement Techniques (Modified after Kamon and Bergado, 1991)

These ground improvement techniques utilize either mechanical energy or man-made materials. The last decade has seen an increasing demand for in-situ deep soil mixing work in the entire world. A common way is to inject cementing agents into the pore space to bind soil particles together. This is succeeding using a sort of cement, jetting, chemical, and compaction grouting techniques (Karol, 2003). All commonly used chemical grouts are hazardous except sodium silicate (Karol, 2003). The hazardous and potential environmental risk for many cementing agents encourages the development of alternative soil improvement methods that are more environmentally friendly and more sustainable. In the past decade ground improvement techniques have become one of the greatest challenges for the geotechnical engineering mainly due to the environmental limitations on public works and over-population in urban areas. The practices, however, have been changing with the time mainly because of the development of new materials, new machinery, and new technologies (Terashi and Juran, 2000). Thus, it is essential to reach an adequate level of different science and technologies. Biotechnology is one these techniques.

1.1.1 Biotechnology

Interest in the use of biological technologies in geotechnical engineering has been rising over the past few years (Mitchell and Santamarina, 2005). In recent years, an ecological and sustainable ground improvement technique that has shown some promise is the use of bioactivity in sand cementation via calcium carbonate precipitation, named as Microbially Induced Calcite Precipitation (MICP). The most commonly used type of MICP is passive precipitation, where the pH of the system is changed as a result of bacterial activity (usually urea hydrolysis), carbonate ions are produced, and then chemical precipitation of CaCO3 takes place inside the soil pore space in the presence of calcium ions. It uses biochemical process that exists naturally in soil to enhance engineering properties of soils. Calcite (CaCO3) is an attractive element to be studied in bio-cementation because calcite formation is commonly found in nature and it is one of the most common and widespread minerals on Earth, constituting 4% by weight of the Earth's crust. It is naturally found in extensive sedimentary rock masses, as limestone, marble and calcareous sandstones in marine (Hammes and Verstraete, 2002). The calcite precipitation cements sand grains together which increases the mechanical properties (i.e. strength, stiffness) of the sand. This bio-cementation enhances the shear strength of soil through the generation of soil particle-binding materials, as the result of introducing bacteria and cementation reagents into the soil. The soil particle-binding materials are mostly silicates, carbonates, sulphides, phosphates, and hydroxides (Ivanov and Chu, 2008). Ivanov and Chu (2008) carried out an proximate cost crosscheck between the raw materials for microbial grouting and the conventional chemical grouting and they recommended that the cost for microbial grouting ($0.5 – $9/m3 of soil) is considerably cheaper than that of chemical grouting ($2–$72/m3 of soil). As a result, there is significant potential for a reduction in environmental concerns on various types of projects; in the long-term, this technique may also prove to be a much more sustainable form and economical of ground improvement. This new research field, focusing on harnessing biological activity to manipulate the local geochemistry and improve the mechanical properties of the soil, is bio-mediated soil improvement. There are several factors essential for promoting successful MICP soil treatment including bacteria type, nutrients, bacteria cell concentration, geometric compatibility of bacteria, fixation and distribution of bacteria in soil, reagents concentration, pH, temperature and injection method.

1.1.2 Use of Fly Ash

Most of the electricity generated in the world is produced by coal combustion. Coal is a complex, heterogeneous material, and the end product of a series of biological and physicochemical processes. When pulverized coal is burnt to generate electrical power, extremely large quantities of fly ash and bottom ash are produced which presents another environmental challenge. This solid waste material produced by the combustion of coal, carried out of the boiler by flue gases and extracted by cyclone separators or electrostatic precipitators and filter bags. Fly ashes are pozzolanic siliceous and aluminous materials which, though themselves possessing little or no cementitious value, will, in finely divided form and in the presence of moisture, react chemically with calcium hydroxide at ambient temperature to form compounds with cementitious properties (ASTM Standard C618-80).Fly ash was started to be used in the United States 1930s with the development of industry that uses electrical energy (Davis et al., 1937). During the early period of 1940 to 1960, usefulness of only Class F fly ash was investigated since that was the generally available ash at that time. Fly ash has acquired considerable importance in the building materials sector. Beneficial use of fly ash in construction projects requiring large material volumes, such as high-way embankment construction, offers an attractive alternative to disposal because substantial economic savings can be attained by the reduction of ash disposal costs and the conservation of natural soils and lands. Large volumes of earthen materials are used in construction sector each year in the world. In many cases, these materials can be replaced with secondary materials, suitable waste materials and construction debris that are normally disposed in landfills, and can generate millions of dollars savings. An effective use of fly ash would provide considerable environmental benefits including reducing water- air pollution, and energy savings (Hausmann, 1990; Collins & Ciesielski, 1992). Little amount of fly ash are used in various application in Turkey, although 80% of fly ash produced across the world is effectively used in many engineering applications, thereby reduce the potential impact on the environment and avoid economical loss caused by the disposal of fly ash (Alkaya, 2002). Lime as well as fly ash could be used for soil stabilization. Numerous studies have been completed in which the engineering properties and the physical and chemical characteristics of fly ash were determined in the laboratory. Uses of fly ash were established for a number of applications and the advantages and disadvantages were identified. Fly ash has been used as bulk fill material in geotechnical fill, such as in construction of embankments, dikes, and road subgrade (DiGioia and Nuzzo 1972, Gray and Lin 1972). The advantages of using fly ash as a bulk fill material include low cost, low unit weight, and good strength. Significant efforts have been made to use fly ashes in stabilization of highways base structures, unpaved roads and soil stabilization.

1.1.3 Use of Waste Tire

Due to the developing industry and growing population, huge amounts of waste disposal are produced. One of the problems associated with socio-economic development of a country is waste disposal. Increase in the number of waste tires results in an environmental problem in many regions of the world. And, as the amounts increase, it becomes harder and more expensive to dispose them safely without threatening the ecology. For example, Masad et al. (1996) stated that there are discarded tires and over tires in stockpiles every year in the United States alone. There are about 28 million tires stockpiled in Canada (Dickson et al. 2001). Every year about 3 million tons of used tires (part worn end of life tires) are generated in Europe, of which 2.4 million tons are end-of-life tires for which value recovery has to be maximized. This amounts to approximately 200 million units. Such wastes cannot generally be deposited in landfills since they require large spaces. Large quantities of waste tires discarded each year can be beneficially used in geotechnical and geoenvironmental applications (Edincliler 2008). The recent trend in finding lightweight solution to geotechnical engineering works has resulted in various construction techniques that are based on developments of new materials and concepts. One of them is the use of domestic and industrial by products. Laboratory investigations showed that strength and compressibility of shredded tire in block form can be engineered to requirements. So, ways to utilize these tire wastes in large volumes are being investigated. Using them as construction materials in civil engineering applications is a common way that has been studied for many years. The lightweight and high drainage characteristics of tire waste lead them to be used as construction materials in embankments. Tire wastes are used as whole tires or as processed tires. Processed tires form tire shreds, tire crumbs, chips and buffings. As the rubber tires do not easily decompose, engineers studying the physical properties of sand– shredded tire mixes have concluded that this mix can be used in many engineering projects. Civil engineering applications for scrap tires include lightweight fill, conventional fill, insulation layer, retaining wall and bridge abutment, and drainage applications (Young et al. 2003).

1.2 Objective of Thesis

An extensive laboratory testing program was undertaken to provide information on the geotechnical properties of a sand treated with biological product (xanthan gum), fly ash and waste tire. The thesis mainly focuses on determining the possibility of using biological product (xanthan gum), fly ash and tire waste to improve the mechanical properties of sand. The purpose of biological study is to make an investigation on the influence of bacterial products (xanthan gum) on the bearing capacity and shear strength parameters of sand. The cementation effects of this bacterial product (xanthan gum) will be studied in detail on sand using California Bearing Ratio (CBR) and Unconfined Compression Strength (UCS) tests. The second study which is on the characterization of fly ash for soil stabilization and utilization has been studied in detail. An extensive series of experiments have been carried out on the mixture of soil-lime-fly ash at various ratios. Consolidation, standart proctor, falling head permeability and CBR tests were completed to investigate some engineering properties of fly ash–lime- sand mixtures. A brief study about certain types of waste tire as a lightweight material is followed throughout the thesis. Also, the laboratory results are evaluated and a summary for the characteristics of the specific lightweight materials as tire crumb and tire buffings, is given. Oedometer, constant head permeability and direct shear tests were performed for various proportions of sand- waste tires mixtures.

1.3 Organisation of Thesis

This thesis is organized into six chapters as follows: Chapter 1 describes introduction for the research, objectives of the research and scope of the work. Theory together with experiences gained from recent laboratory data provides valuable information and practical guidance. Chapter 2 presents a literature review that was performed to summarize the effects of biopolymer, fly ash and waste tire on soil behavior. Chapter 3 indicates the experimental work of three experimental studies which are given in this thesis. The first study which is given is the study on the effect of xanthan gum to some geotechnical properties. This study is under review for a possible publication in the 3rd GeoShanghai International Conference in China. Furthermore the thesis deals with fly ash- lime mixtures in stabilization of a sand. An extensive experimental study on consolidation, compaction, permeability and CBR tests has been carried out in this chapter. Class C fly has been used during this study. This study was published in the 3rd international conference on new developments in soil mechanics and geotechnical engineering conference in North Cyprus. Moreover this chapter covers a study on waste tire inclusions-sand mixtures. In this study it is aimed to present the effect of various shape of waste tires on some geotechnical properties. Two different processed waste tires has been used during this study. This study was published in the 3rd international conference on geotechnical engineering-new developments in analysis, modelling and design in Tunisia. Chapter 4 discusses the materials and methods that were used in the laboratory studies. Also in Chapter 4, the results from each study that were conducted are discussed and illustrated by figures. Chapter 5 presents the overall conclusions with recommendations for future research on the use of biopolymer, fly ash and waste tire on the soil behaviour.

CHAPTER II

LITERATURE REVIEW AND BACKGROUND

2.1 Introduction

The stability of soil is one of the major factors affecting the life and durability of infrastructure projects such as highways, roads, embankments, tunnels, retaining walls, irrigation structures and high-rise buildings. Furthermore, the costs for repairing and maintenance of roads, highways, tunnels and bridges are considerable. For the most part, the attention of researchers has been focused on finding better ways to achieve sustainable, durable, safe and economic infrastructures. One of the important parameters that cause damage and reduce the age of an infrastructure is related to the instability of soil.

2.2 Biotechnology

The explosion of new knowledge of biology and micro-biology in science and technology during the past several years, this led to explanation of the important roles of chemistry and mineralogy in determining soil properties and behavior. It is appropriate to critically assess whether developments in these fields may have potential to lead us to better understanding and new directions in geotechnical research and practice.

Biological treatments has been studied in other fields, i.e. immobilizing calcium and contaminants in surface and ground water treatment (Hammes et al., 2003; Mitchell and Ferris, 2005), restoration of calcareous stone materials (Levrel et al., 1999), and bioremediation (Ferris, 2003), improvement of concrete strength and durability (Muynck et al., 2008; and Achal et al., 2011), concrete technology to remediate cracks (Bang et al., 2001; Ramachandran et al., 2001; Ramakrishnan, 2007; Jonkers et al., 2009), brick durability (Sarda, 2009), and increasing the shear strength, compressive strength and bearing capacity of the soil based on the filling of pores

(Dejong et al., 2006; Canakci and Cabalar, 2003; Akbulut and Cabalar, 2014), soil (or sand) strength (Ruyt and Zon, 2009; Lu et al., 2010; Qian et al., 2010; Gurbuz et al., 2011), sand impermeability (Nemati and Voordouw, 2003; Nemati and Greene, 2005), in petroleum industry to directing the oil flow in the required direction (Lappin-Scott et al., 1988; MacLeod et al., 1988), stabilization of contaminated soils (Khachatootrian et al., 2003), stabilization of metals (Etemadi et al., 2003) and development of shields for zonal remediation (Yang et al., 1994; Perkins et al., 2000). A detailed discussion of the microbiological issues is given by Whiffin (2004), Mitchell and Santamarina (2005), De Jong et al. (2006), Whiffin et al. (2007), van Paassen et al., (2010). This new research field, focusing on harnessing biological activity to manipulate the local geochemistry and improve the mechanical properties of the soil, is bio-mediated soil improvement. There are several factors essential for promoting successful MICP soil treatment including bacteria type, nutrients, bacteria cell concentration, geometric compatibility of bacteria, fixation and distribution of bacteria in soil, reagents concentration, pH, temperature and injection method.

2.2.1 Bio-Polymers

The World Health Organization (1975; 1987) performed toxicity studies for guar gum and xanthan gum, and then found that they do not represent a hazard to health and that there was no need to establish an acceptable daily intake of the substances. Sandford et al. (1984) established that both guar gum and xanthan gum are used in agricultural fertilizers and feed supplements with no harm to the environment. These two exopolymer analogs show little evidence in causing environmental harm should they be used for wetland stabilization and could enhance the growth of plants (Wallace 1986), while improving sediment stability.

Both in-situ and ex-situ applications of biopolymers for soil improvement have been explored. Bio polymer mixed with soils has been shown to reduce hydraulic conductivity and increase shear strength (Kavazanjian et al. 2009, Nugent et al. 2010). Martin et al. (1996) showed that mixing a silt with 0.3% (by weight) xanthan gum, a commercially available biopolymer, can reduce saturated hydraulic conductivity by two orders of magnitude and increase the drained shear strength by up to 30 percent. Biopolymers are used in biodegradable drilling muds due to their propensity for bio-plugging (Hamed and Belhadri, 2009). They have also been used to create permeable reactive barriers for groundwater remediation, with biopolymer degradation accelerated by introduction of a high-pH "enzyme breaker" after placement of the permeable reactive material in an open trench supported with biopolymer slurry (Sivavec at. al, 2003). Khachatoorian et al. (2003) observed "plugging" and permeability decreases of up to 14 orders of magnitude in less than two weeks in sand treated with biopolymer slurry. Some investigators have explored increasing soil shear strength in situ by biopolymer generation, (e.g., Cabalar and Canakci 2005). However, most investigations of applications of in situ biopolymer growth and EPS generation have focused on reduction in hydraulic conductivity to form hydraulic barriers (e.g. Bonala and Reddi, 1998; Seki et al., 1998; Wu et. al, 1997). Furthermore, there are many case histories of clogging of filters in dams, landfills, and water treatment plants due to growth of biofilms (Ivanov and Chu, 2008). For instance, in October 1985 an investigation of subsurface drain clogging at the Ergo Tailings, only six months after being in service, revealed that the geotextile drain filter was clogged due to the growth of arsenic resistant microorganisms (Legge at al. 1985).

2.2.2 Microbially Induced Calcite Precipitation (MICP)

Microbially Induced Calcite Precipitation, or MICP, has been the primary focus of research in bio geotechnical engineering to date. In MICP, the creation of calcium carbonate (calcite) occurs as a consequence of bacterial metabolic activity (Ramakrishnan. et al., 2001; Stocks-Fischer et al., 1999). Calcite precipitation may be achieved by many different processes (DeJong et al., 2010), including urea hydrolysis (Benini et al., 1999; Ciurli et al. 1999), denitrification (Karatas et al 2008 van Paassen et al., 2010; Hamdan et al., 2011), sulfate reduction (Warthman et al., 2000, inducing dolomite precipitation), and iron reduction (Roden et al., 2002, Weaver et al. 2011, inducing ankerite and Other mixed Mineral precipitation). Enzymatic hydrolysis of urea by microbes is the most energy efficient of these processes (DeJong et al., 2010), and urease activity is found in a wide range of microorganisms and plants (Bachmeier et al., 2002). Bacillus pasteurii (American Type Culture Collection 6453), which was recently renamed Sporosarcina pasteurii (ATCC 11859), an alkalophilic Bacterium with a highly active urease enzyme (Ferris et al., 1996), have been used in laboratory studies where bio-augmentation has been performed to produce calcite precipitation (Mortensen et al., 2011). More recently bio-stimulation of native bacteria has reportedly been successful (Burbank et al. 2011, 2012a) and the influence of competing bacterial species/processes explored (Gat et al., 2011). Microscopy techniques show the calcite structure varies with Treatment formulation (Al Qabany et al., 2012), cementation occurs preferentially at particle contacts (Chou et al. 2008; Martinez et al. 2009), calcite precipitation occurs directly on/around individual bacteria, and cementation breakage during shearing occurs within the calcite crystals (DeJong et al., 2010). Table 2.1 shows the urease activity for different bacterial cultures.

Table 2.1 Urease production by different bacterial cultures (Modified from Sarda, 2009)

Bacteria Culture	Urease Activity (Urea/ml)
Bacillus pasteurii NCIM 2477	17,5
Bacillus lentus NCIB 8773	10
Brevibacterium ammoniagenes ATCC 6871	12,5

The main nutrient solution that is commonly used to provide the necessary nutrition for the bacteria, as well as the chemical compounds that are needed for soil cementation, contains $NaHCO_3$, NH_4Cl, $CaCl_2$, urea, and a nutrient broth.Under favorable environmental conditions, S. pasteurii uses urea as an energy source, producing ammonia (NH_3) and carbon dioxide (CO_2), which tends to increase the pH in the proximal environment.This enzymatic hydrolysis of urea occurs in the bacteria cell and is generally described using the following chemical reaction (Sarda et al., 2009):

$$\underset{\text{(urea)}}{NH_2CONH_2} + H_2O \rightarrow \underset{\text{(ammonia)}}{2NH_3} + CO_2$$

$$2NH_3 + H_2O \rightarrow 2NH4^{+} + 2OH^{-}$$

$$CO_2 + OH^{-} \rightarrow HCO_3^{-}$$

$$Ca^{2+} + HCO_3^{-} + OH^{-} \rightarrow CaCO_3 + H_2O$$

The overall chemical reaction process that occurs in the sand matrix is displayed schematically in Figure 2.1. The primary restriction on microbial transport is the size of pore throats within the soil matrix through which the microbes must pass as they move from one pore space to another (Mitchell and Santamarina, 2005). This provides an approximate lower bound limit on treatment by in-situ injection which depends on the particle size relative to the microbe size (Figure 2.2).

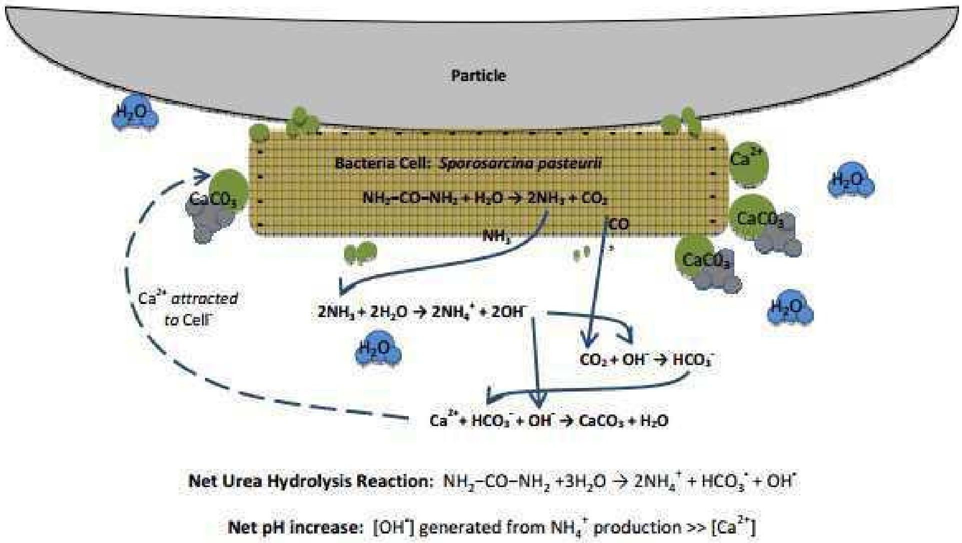


Figure 2.1 Schematic view of biological calcite precipitation in sand matrix (DeJong et al., 2010)

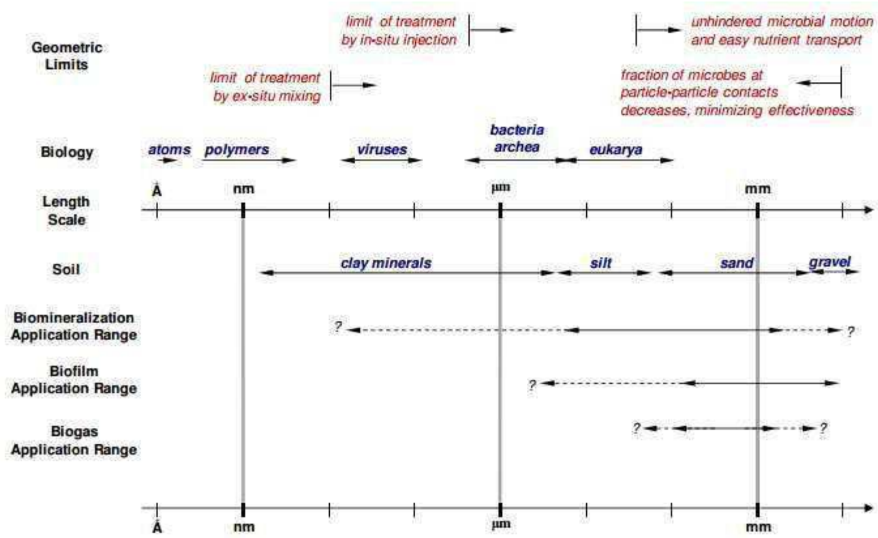


Figure 2.2 Comparison of typical sizes of soil particles and bacteria, geometric limitations, and approximate limits of various treatment methods. (DeJong et al., 2010)

2.2.3 Improvement of Soil Engineering Properties

2.2.3.1 Shear Strength

Canakci and Cabalar (2005) conducted laboratory direct shear tests on biologically treated sand. The sand was subjected to different levels of biopolymer treatment, with a commercially available biopolymer produced by the bacteria, Xanthomonas compestris. Samples treated with different biopolymer contents were cured 7, 28, and 50 days. They found that the shear strength of the sand increased as the biopolymer supply was increased (Figure 2.3 and Figure 2.4). The soil particles were held together by a biopolymeric material known as xanthan gum because of its adhesion. These results support the possible use of biopolymer forming bacteria for ground improvement applications in geotechnical engineering.

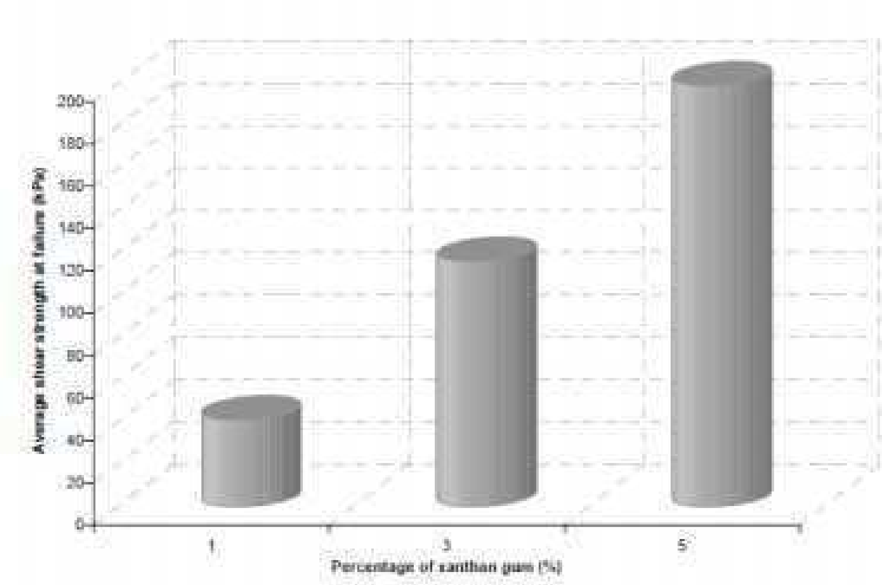


Figure 2.3 Influence of xanthan gum on average shear strength of sand (Canakci and Cabalar, 2005)

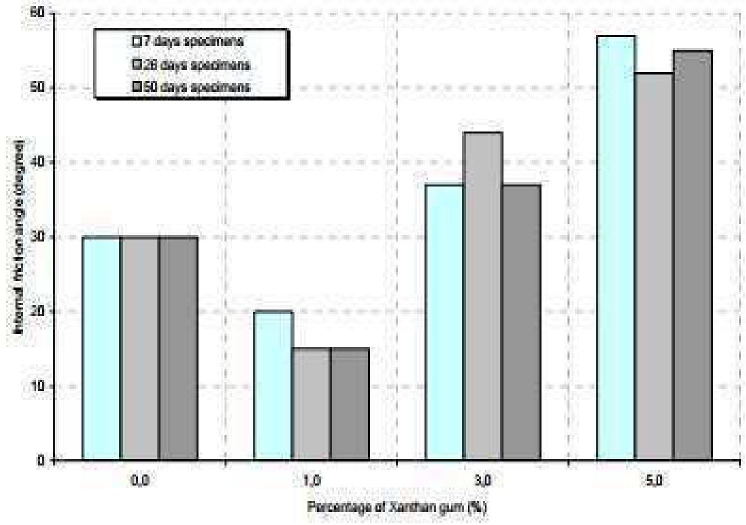


Figure 2.4 Influence of Xanthan Gum on internal angle of friction (Canakci and Cabalar, 2005)

2.2.3.2 Permeability

Because of its importance in some geotechnical problems including the determination of seepage losses, settlement computations, and stability analyses (Boadu, 2000), many of field and laboratory investigations of permeability have been made (Hazen, 1892, 1911; Kozeny, 1927; Carman 1937, 1956; Krumbein and Monk, 1942; Harleman et al., 1963; Terzaghi and Peck, 1964; Masch and Denny, 1966; Wiebega et al., 1970; Shepherd, 1989; Uma et al., 1989; Alyamani and Sen, 1993, Cabalar et al., 2012; Akbulut and Cabalar, 2013a).

Microbial cementation occurs in pores within soil particles, reducing the pore throats and subsequently preventing water flow (Whiffin et al., 2007). Whiffin et al., (2007) determined the primary stages of microbial cementation such as urease activity, ammonium concentration, calcium concentration and calcium carbonate content within the soil specimen. Results from triaxial tests conducted on specimens from the treated sand column show that the soil porosity, strength, and stiffness were all significantly affected by the calcium carbonate content. The porosities of bio-treated soil specimens were up to 90% smaller than those specimens that had not been treated, as shown in Figure 1.6.

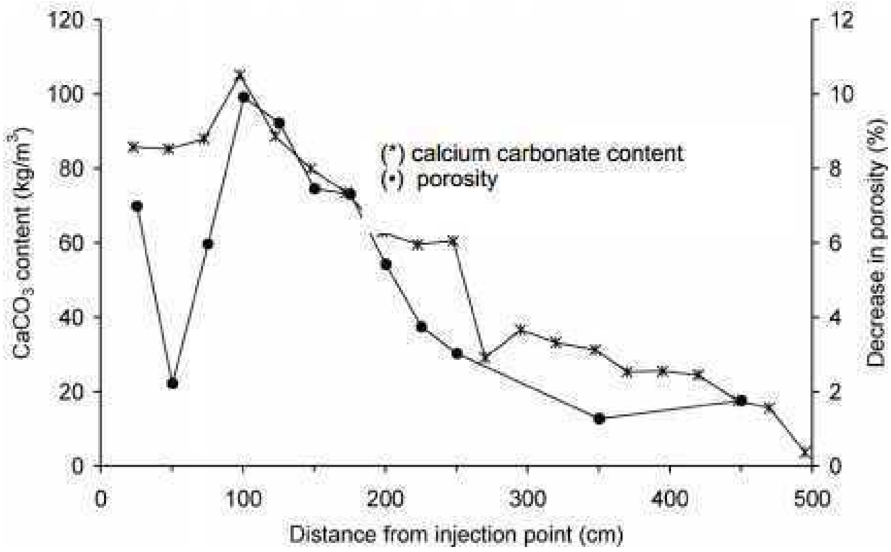


Figure 2.5 Calcium carbonate content and porosity vs. the distance from the injection point along the column length (Whiffin et al., 2007).

Bouazza et al. (2009) indicated that the biotreatment cause a reduction of the initial hydraulic conductivity of sands ($1x10^{-6}$m/s) (Figure 1.7). The decrease in the initial hydraulic conductivity of the silty sand due to biopolymer induced-pore clogging exceeded three and four orders of magnitude, respectively, with 1% sodium alginate and xanthan gum was reported by Bouazza et al. (2009). Furthermore, Bouazza et al., (2009) presented that a reduction of at least four orders of magnitude can be achieved using as little as 0.5% xanthan gum, highlighting its superior pore-plugging effect. The better performance achieved with xanthan gum is probably due to its very high affinity for water, as well as itsability to adsorb to clay minerals. In addition, xanthanpolymer chains self-associate to form large, interconnected networks that can lead to bridging of clay particles, as indicated by Chenu (1993). A longer ageing time generally achieved a lower conductivity, as indicated in Figure 1.8. It can be seen from the Figure 1.8 that the hydraulic conductivity of a 0.5% sodium alginate–soil mix reduced from $2x10^{-9}$ m/s at 7 days curing time to $2x10^{-10}$m/s at 70 days (Bouazza et al, 2009). It is also interesting to note that a reduction of at least two orders of magnitude has been reported by Martin et al., (1996) using xanthan gum mixed with a fine-grained soil matrix (clayey silt) or a flooding process through sand (Khachatoorianet al., 2003). In addition, its adsorption to sand can increase the silty-sand biopolymer composite strength, as observed by Bouazza et al., (2001).

Gollapudi et al., (1995) used microbial "mineral plugging" in a highly permeable geologic formation to reduce the overall system porosity. Leaching in the rock fractures was controlled by microbial plugging. A series of column tests was conducted in order to evaluate the effectiveness of the proposed microbial technique. The flow rate into the fractures via the column was measured during the test, and absence of flow showed the completion of plugging. Their results showed that microbial plugging was highly related to bacterial concentration, pH, the flow rate in the fractures, andthe presence of contaminants. The microbial process was more productive in the presence of fractures, which provided more nucleation sites for the bacteria.

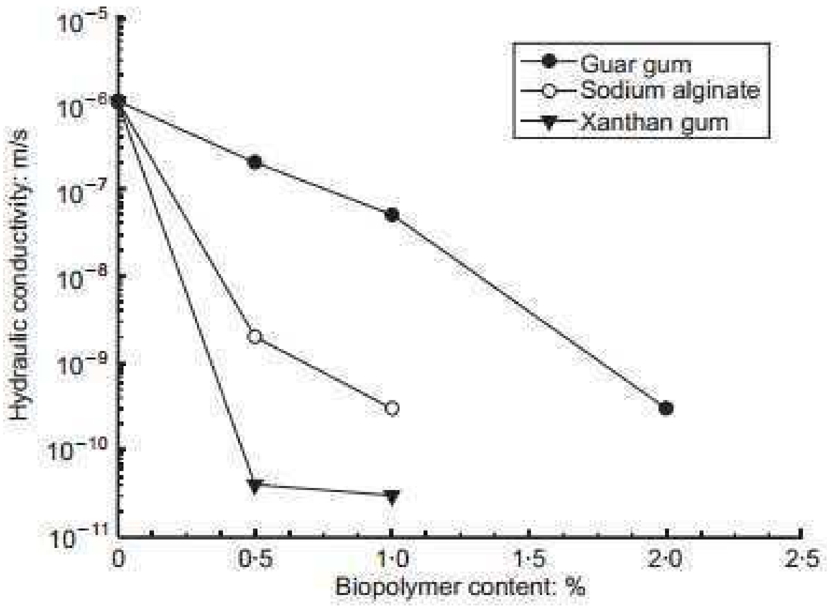


Figure 2.6 Hydraulic conductivity of soil biopolymer mixes after 7 ageing (Bouazza et al., 2009)

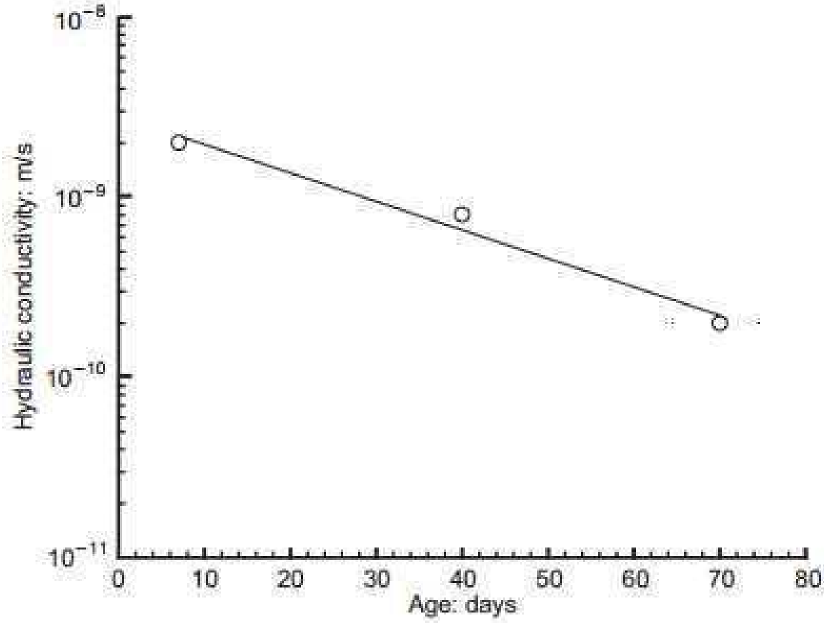


Figure 2.7 Effect of time on hydraulic conductivity of soil biopolymer mix (0.5% sodium alginate) (Bouazza et al., 2009)

2.3 Use of Fly Ash in Geotechnical Engineering

Fly ash is a solid waste material produced by the combustion of coal, carried out of the boiler by flue gases and extracted by cyclone seperators or electrostatic precipitators and filter bags. Fly ash was started to be used in the United States 1930s with the development of industry that uses electrical energy (Davis et al., 1937). An effective use of fly ash would provide considerable environmental benefits including reducing water- air pollution, and energy savings (Hausmann, 1990; Collins and Ciesielski, 1992). Little amount of fly ash are used in various application in Turkey, although 80% of fly ash produced across the world is effectively used in many engineering applications, thereby reduce the potential impact on the environment and avoid economical loss caused by the disposal of fly ash (Alkaya, 2002).

The term "fly ash‟ is often used to describe any fine particulate material precipitated from the stack gases of industrial furnaces burning solid fuels. The amount of fly ash collected from furnaces on a single site can vary from less than one ton per day to several tons per minute. The characteristics and properties of different fly ashes depend on the nature of the fuel and the size of furnace used. Pulverization of solid fuels for the large furnaces used in power stations creates an immediate, urgent problem; dry fly ash has to becollected from the stack gases and disposed of quickly and safely. Fly ashes generally fall into one of two categories, depending on their origin and their chemical and mineralogical composition. Combustion of anthracite or bituminous coal generally produces low-calcium fly ashes; high-calcium fly ashes result from burning lignite or sub-bituminous coal. Both types contain a preponderance of amorphous glass. Composite material made of fly ash is used in many ways and is subject to a variety of different loading conditions, and so different types of stress develop. The compressive strength of concrete, one of its most important and useful and one of the most easily determined properties, is indicated by the unit stress required to cause failure of a specimen. In addition to being a significant indicator of load-carrying ability, strength is also indicative of other elements of quality concrete in a direct or indirect manner. In general, strong concrete will be more impermeable, better able to withstand severe exposure, and more resistant to wear. On the other hand, strong concrete may have greater shrinkage and susceptibility to cracking than a weaker material. Finally, the concrete-making properties of the various ingredients of the mix are usually measured in terms of the compressive strength. From the results obtained it was found that the flyash can be compacted over a large moisture content range thus it has a potential to be used infills and embankments. Also since fly ash is having low permeability thus it further benefits the use in fills and embankments by reducing the chances of damage to the ground water resources. The low specific gravity of fly and the pozzolanic activity of fly ash aids for its use along-with cements for construction purposes and also in manufacturing of bricks.

A literature review reveales that investigations on fly ash and lime utilization for soil stabilization have been conducted in the past years by many researchers (Mitchell & Katti, 1981; Smith, 1993; Consoli et al., 1997; Consoli et al., 2001; Edil et al., 2006; Sezer et al., 2006; Akbulut and Cabalar, 2012, 2013c), as the fly ashes could be used to control of volume change, increase strength ans as a drying agent (Ferguson, 1993). Edil et al., (2006) indicated the effective of fly ash for stabilization of fine grained soils.

Uses of fly ash were established for a number of applications and the advantages and disadvantages were identified. Cooperative tests were conducted by ASTM

Committee C-9 (1962) and studies on the fundamental characteristics of Class F fly ashes were reported by Minnick (1959) during this period. The early studies concluded that substantial amount of the Portland cement in concrete could he replaced with fly ash without adversely affecting the long term strength of concrete (Timms and Grieb, 1956). These were basically sub-bituminous ashes. Since 1970s, a number of studies have been reported dealing with the characteristics of the fly ashes from sub-bituminous. These self hardening fly ashes generally contained larger amounts of calciumas compared to the bituminous ashes. A new class of ash was therefore added to the ASTM specifications which included ashes with more than 15% calcium oxide and combined silica, alumina, and iron oxide content less than 75%. These ashes exhibit many other useful properties. The most use and, therefore, most research have been on the use of fly ash in cement and concrete.

2.3.1 Types and Properties of Fly Ash

2.3.1.1 Definitions and Specifications

Pozzolans are siliceous and aluminous materials which, though themselves possessing little or no cementitious value, and in the presence of moisture, react chemically with calcium hydroxide at ambient temperature to form compounds with cementitious properties (ASTM Standard C618-80). Fly ash is a solid, fine-grained material resultingfrom the combustion of pulverized coal in power station furnaces. The material is collected in mechanical or electrostatic separators. The term fly ash is not applied to the residue extracted from the bottom of boilers. Fly ashes capable of reacting with $Ca(OH)_2$ at room temperature can act as pozzolanic materials. Their pozzolanic activity is assignable to the presence of SiO_2 and $Al2O_3$ in amorphous form.

As according to ASTM C-618, two major classes of fly ash are recognized. These two classes are related to the type of coal burned and are designated Class F and Class C in most of the current literature. Class F fly ash is normally produced byburning anthracite or bituminous coal while Class C fly ash is generally obtained by burning sub-bituminous or lignite coal.

The specific gravity of fly ash is reported to be related to shape, colour as well as chemical composition of fly ash particle. It is adopted as an indirect performance parameter for determining the performance of fly ash in concrete. In ASTM C618, for quality control of fly ash, the uniformity of the fly ash is monitored by limiting the variability of the specific gravity and fineness as measured by the amount retained on 45 μm mesh sieve. The requirement is that any sample tested shall not deviate from the average of 10 previous tests or the total of all tests ifthe number is less than 10, by more than 5%. In general specific gravity of fly ash may very from 1.3 to 4.8 (Joshi, 1968). However, the Canadian fly ashes have specific gravity ranging from 1.91 to 2.94 whereas those of the American ashes have specific gravity between 2.14 and 2.69. Coal particles with some minerallic impurities have specific gravity between 1.3 and 1.6. Opaque spherical magnetite (ferrite spinel) and hematite particles, light brown to black in colour, when present in sufficient quantity in fly ash increase the specific gravity to about 3.6 to 4.8. As the amount of quartz and mullite increases, the specific gravity decreases. Fly ash pulverization releases some of the gases trapped, during quenching inside the large hollow spherical particles, and increases the bulk specific gravity of the fly ash (Joshi 1968, 1979).

2.4 Use of Waste Tire in Geotechnical Engineering

Increase in the number of waste tires results in an environmental problem in many regions of the world. For example, Masad et al. (1996) stated that there are discarded tires and over tires in stockpiles every year in the United States alone. There are about 28 million tires stockpiled in Canada (Dickson et al. 2001). Every year about 3 million tons of used tires (part worn end of life tires) are generated in Europe, of which 2.4 million tons are end-of-life tires for which value recovery has to be maximized. This amounts to approximately 200 million units. Such wastes cannot generally be deposited in landfills since they require large spaces. Large quantities of waste tires discarded each year can be beneficially used in geotechnical and geoenvironmental applications (Edincliler 2008).

As the rubber tires do not easily decompose, engineers studying the physical properties of sand– shredded tire mixes have concluded that this mix can be used in many engineering projects. Civil engineering applications for scrap tires include lightweight fill, conventional fill, insulation layer, retaining wall and bridge abutment, and drainage applications (Young et al. 2003). Ahmed (1993) and Masad et al. (1996) used triaxial testing apparatus to study the shear strength properties of waste tire particles in various size and shapes. Ahmed (1993) said that sand can be strengthened by tire chips. Ahmed (1993) reported that adding tire chips increases the shear strength of sand, with angle of friction up to obtained for dense sand with 30% tire chips. Masad et al. (1996) concluded that the 66 shreded tires and Ottowa sand mixtures have a potential to be used as a lightweight fill material in highway embankments over compressible soils. Tire shreds have been used as lightweight fill material in many embankments and retaining structures (Bosscher et al. 1997; Tweedie et al. 1998; Humphrey et al. 1998; Lee et al. 1999; Dickson et al. 2001; Zornberg et al. 2004). These studies show that the use of tire shred–soil mixtures have lower compressibility and higher shear strength and thus perform better than only tire shreds. Embankments constructed with soil–tire chip mixtures can potentially have steeper slopes because the backfill has higher shear strength and lower unit weight. Steeper side slopes decrease the volume of material needed. Also, because of using lightweight material, settlement of underlying soil is reduced (Tatlisoz et al.1998). Edil and Bosscher (1994) reported that placing tire shreds in sand vertically led to higher shear strength on the plane perpendicular to the shred. Tire wastes can be used as lightweight material either in the form of whole tires, shredded or chips, or in mix with soil. Many studies regarding the use of scrap tires in geotechnical applications have been done especially as embankment materials. These studies included laboratory investigations, numerical and physical model and field investigations (Ahmed, 1993; Bernal, 1996; Masad et. al, 1996; Lee et. al, 1999; Tweedie et. al, 1998, Bergado and Youwai, 2002; Humphrey and Tweedie, 2002; and Edil, 2002; Akbulut and Cabalar, 2013b). And recently, Lee (2002) has introduced the concept of cementing tire rubber bits into rubber blocks using binder materials. Ghani et. al (2002) used shreds instead of bits in a study to improve and established pre-determine yielding behavior of geomaterial developed for use as retaining wallbackfill layers or inclusions. In geotechnical related works, lightweight geomaterials were developed or utilized mostly as filling materials for road construction purposes. The lighter materials minimize foundation requirements, reduce land cutting for mountainous area, prevent settlements and shorten construction times. In the case of retaining wall, lighter fill will reduce the lateral earth pressure thus reducing the structural requirements of the wall including the foundations. The key properties of the various form of tire wastes based geomaterial are as stated in Table 2.2.

Table 2.2 Various form of Scrap Tire Applications in Civil Engineering Works (Ghani et al, 2002)

Category	Density	Strength	Other Geotechnical Properties
Whole Tire	NA	NA	Usually used in reinforced soil applications. Also asflexible culvert and slope stabilization.
Tire Shreds/Chips	3- 5.5 kN/m^3 with compaction 8 - 9 kN/m^3	□ range from 37- 43 degree for loose and up to 85when compacted.	Good draining properties , more than 1 cm/sec.
Tire Shreds/Chips mixed with soil	Up to 13 kN/m^3 for 50/50 soil-tire mixture	Compacted to requirements	Mixing of 38- 40% tire shreds/chips give the optimum economic and technical advantages.
Tire shreds/chipswith binding additives	5- 10 kN/m^3, but density and other properties can be engineered to requirements.	Compressive strength can be engineered to requirements.	Good draining properties. Placementsby pumping or inblocks form.

Perhaps the easiest way of disposing tires is by landfilling. Tires are however not suitable for landfilling since the volumes are large, rubber almost non-degradable andpossess a large energy value that aggravates landfill fires. This growing disposal problemhas been noticed by the environmental authorities in a number of countries and legislation acts has been taken to encourage other disposal options than landfilling, e.g. by banning tire material on landfills with in the European Community, Eur-Lex (1999). The intention of the legislation is to encourage Best Management Practices (BMP) for the reuse of end of life tires and to reduce the occupation of valuable deposit space in the landfills by tire wastes. This strategy is also used in other parts of the world, e.g. Individual states in the USA, USEPA (2006). Tires as a disposal problem have also been discussed with in the United

Nations Environmental Programme (UNEP), resulting in technical guidelines for BMP of scrap tires, UNEP (2002). Among the listed options in the technical guidelines is use of tire wastes as construction material listed.

Based on the positive experiences, mainly from the U.S.A. and Canada and the encouraging regulations towards alternative disposal options, tire wastes as construction material could be of interestin Europe and Turkey. Tire wastes are fragmented end of life tires, mainly from passenger cars but also from heavy vehicles. The fragmentation is performed by a shredder. Primarily tires are shredded for volume reduction before transportation to recovery or disposal processes. The size of the individual wastes is controlled by sieving and reshredding of coarse wastes.

2.4.1 Background of Waste Tire Studies

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

Humphrey et al. (1993) has conducted large-scale direct shear tests with three different tire chips. Tire chips with length smaller than 72mm were used. This study reported that friction angles and cohesion were ranging between 19˚ and 25˚ and 7.7- 8.6 kPa respectively. In the study it is stated that tire chips are useful for constructing lightweight embankments over soft soils.

Tatlisoz et al. (1998) conducted a the large scale direct shear tests with tire chips, sand, sandy silt, sand-tire chips and sandy silt-tire chips mixtures. In the study, it is reported that sand-tire chips mixture had an increasing behavior of shear strength as the volume of the tire chips increased up to 30 per cent, whereas the sandy silt-tire chip mixture did not have a change in the angle of internal friction, but just an increase in the cohesion.

Edincliler et al. (2004) conducted the large scale direct shear tests with tire buffings, sand and sand tire buffings mixtures. In the study, it is reported that 10 per cent by weight tire buffing addition to sand alters the deformation behavior of the mixture by stiffening the material at low strains and softening the mixture at large strains.

Based on beneficial technical use and acceptable environmental impact a selection of best management practice has been performed. In all applications the environmental point of view must be considered. Based on the tire waste material properties the following applications have been identified as potential interest.

- Lightweight Fill
- Thermal Insulation
- Drainage Layer
- Backfill Material
- Elastic Layer
- Limitations in Use

CHAPTER III

EXPERIMENTAL STUDY

3.1 Effects of Bio-Polymer on Some Geotechnical Properties of a Sand

This study is under review for a possible publication in the *3rd GeoShanghai International* Conference in China.

3.1.1 Materials and Method

The materials used in this study were crushed limestone sand, clay and xanthan gum. According to ASTM D1241, from the Gradation Requirements for Soil-Aggregate Materials Table two series of crushed limestone sand were used. Those two series are Type I: Gradation B, and Type II: Gradation F soil aggregate materials.

3.1.1.1 Crushed Limestone Sand

Type I: Gradation B crushed limestone sand which is used in the CBR tests having the minimum and maximum dry densities 1.69 g/cm^3 and 2.09 g/cm^3 respectively.The sand particles, which are crushed, and mainly limestone origin, are between (around) 0.075 mm and 19.0 mm. It is found that D_{10}, D_{30} and D_{60} sizes are around 0.075, 1.40 and 11.0 respectively. Thus, the coefficient of uniformity (Cu) and the coefficient of curvature (Cc) have been approximately calculated 146.6 and 2.38 respectively.

Type II: Gradation F crushed limestone sand has been used during the unconfined compression tests and having a minimum and maximum dry densities 1.19 g/cm^3 and 1.49 g/cm^3 respectively. The grain size distribution of this sand is between 0.075 mm and 4.75 mm. It is found that D_{10}, D_{30} and D_{60} sizes are around 0.07, 0.18 and 0.75 respectively. Thus, the coefficient of uniformity (Cu) and the coefficient of curvature (Cc) have been approximately calculated 10.7 and 0.6 respectively.

Figure 3.1 indicates the particle size distributions of the crushed limestone sand used during the experimental study. Some properties (D_{10}, D_{50}, C_u, C_c, G_s, e_{max}, e_{min}) of the sand are listed in Table 3.1. Scanning Electron Micrograph (SEM) picture show the physical shape of the sand used during this investigation (Figure 3.3).

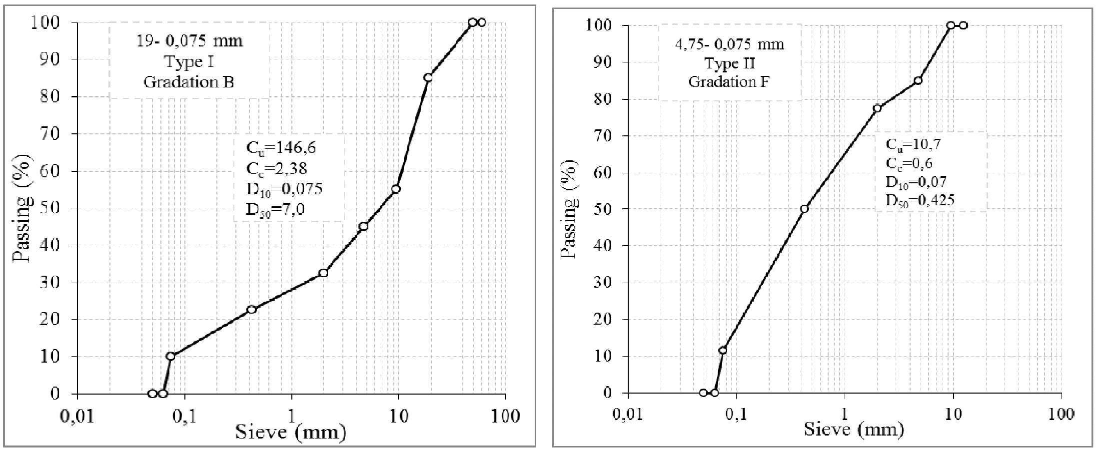


Figure 3.1 Grain size distribution (a) Type I: Gradation B, (b) Type II: Gradation F

Table 3.1 Some specifications of the crushed limestone sand

Gradation Requirements for Soil-Aggregate Materials (ASTM D1241)	Type I Gradation B	Type II Gradation F
Unified Soil Classification System (USCS)	GW	SP
Specific Gravity, G_s	2,50	2,50
Maximum void ratio, e_{max}	0,48	0,95
Minimum void ratio, e_{min}	0,19	0,68
Coefficient of uniformity, C_u	146,6	10,7
Coefficient of curvature, C_c	2,38	0,60
Median Grain Size, D_{50}	7,00	0,425
Effective Grain Size, D_{10}	0,075	0,070

Figure 3.2 Picture of the crushed limestone sand

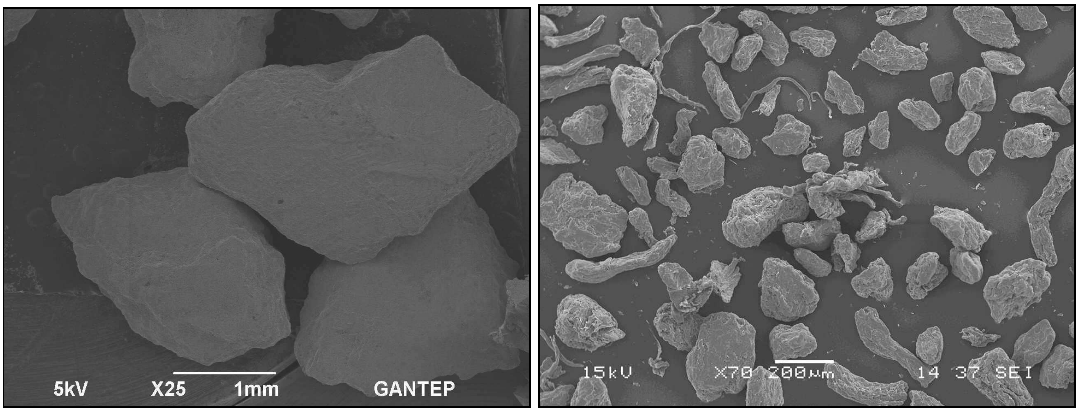


Figure 3.3 SEM pictures of the (a) crushed limestone sand (b) xanthan gum used during the tests

3.1.1.2 Xanthan Gum

'Biopolymer' is a term used to describe the viscous suspension that can be obtained from a variety of biological sources. Biopolymers consist of polysaccharides and water. Polysaccharides are very large molecules composed of linked strands of simple sugars. The large number of hydroxyl and carboxyl functional groups enables polysaccharides to interact strongly with water to produce a viscous suspension Xanthan gum defined as No. G-1253 and prepared by fermentation of dextrose with *Xanthomonas Campestris*, and crushed limestone sand were used in the tests. Xanthan gum is an anionic polysaccharide produced by *Xanthomonas campestris* (Sutherland 1994). Since small concentrations of xanthan gum can greatly increase the viscosity of a solution, it is a commonly used substance. Additionally, xanthan gum solutions are pseudoplastic, which means that the viscosity of a xanthan gum solution decreases with an increased shear rate (Milas et al. 1985).

3.1.1.3 Clay

Clay which is used during the experimental study has been obtained from the campus of Gaziantep University. According to the Unified Soil Classification System (USCS) this clay is defined as low plasticity clay (CL) and has plastic limit (PL) and liquid limit (LL) values, 29 and 47 respectively. Crushed limestone sands with 10% clay were tested in all the testing equipments by various xanthan gum mix ratios. Why it was used is to bind cohesionless crushed limestone sand and xanthan gum. Some specifications of the clay used are given on Table 3.2

Table 3.2 Some specifications of clay

Liquid limit (%)	Plastic limit (%)	Plasticity Indeks (%)
47	29	18

3.1.2 Sample Preparation and Test Procedure

The experimental work was directed mainly towards an investigation of xanthan gum and crushed limestone sand used in combination as blending materials by weight at different proportions on water demand. 0, 1, 3 and 5% xanthan gum contents were studied by the variation of 4, 8, 16 and 32 days curing time. It is crucial to maintain consistency between the specimens prepared for the testing. Therefore, great care was taken to have reasonable repeatability during the preparing of the specimens to be tested. In order to investigate whether the use of xanthan gum achieves any effect on mechanical properties, Unconfined Compression and California Bearing Ratio tests were also performed on specimen of clean limestone sand with 10% clay.

3.1.2.1 California Bearing Ratio Test Procedure

The optimum water content for the CBR test has been obtained from the modified standard proctor test (Figure 3.4). After obtaining the optimum water content, the experimental study involved performing a series of laboratory CBR tests have been conducted on crushed limestone sand-clay mixtures with various ratios (0, 1, 3 and 5%) of xanthan gum according to the ASTM D1883- 99 standards. Specimens were prepared by compacting crushed limestone sand-clay mixtures with xanthan gum in wet state in five equal layers in a steel CBR mould of 150mm diameter and 175 mm high. The specimens were compacted in the mould using 56 blows per layer satisfactory. Similar procedure was adopted for compacting other four layers in the mould. The tests were performed as per procedures described in ASTM D1883- 99 standard. All samples have been unsoaked top and bottom for 4, 8, 16 and 32 days. A surcharge plate of 2.44 kPa was placed on the specimen prior to testing. The loads were carefully recorded as a function of penetration up to a total penetration of 6.0 mm. Finally, load penetration curves were drawn for each case and corrections were applied using the standard procedure. From the load-penetration curves so obtained California bearing ratio values were determined. Since for all the cases considered in the present investigation, the CBR value at 5.0 mm penetration was observed higher than that of 2.5 mm penetration even on repetition. Therefore CBR values reported in the present investigation are both 5.0 mm and 2.50 mm penetration.

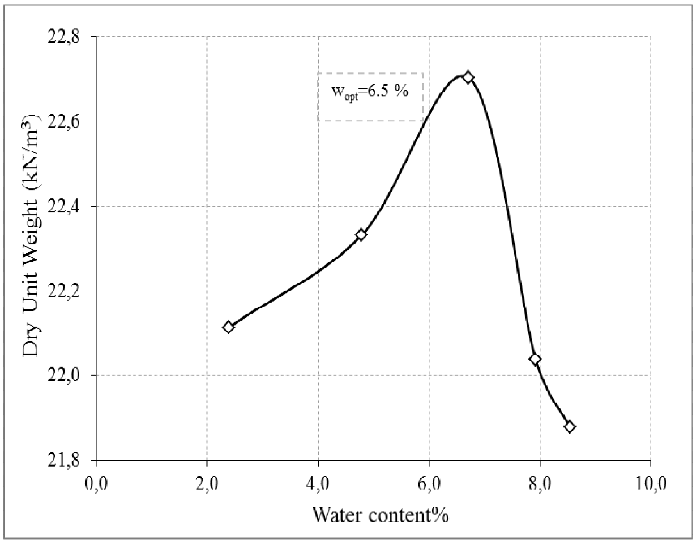


Figure 3.4 Compaction results for CBR test

Figure 3.5 Pictures of testing process for CBR

3.1.2.2 Unconfined Compression Test Procedure

Shimadzu universal testing machine, a conventional laboratory testing method, was used to determine the compressive strength characteristics of the specimens. According to the ASTM standard, the unconfined compressive strength (q_u) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test. The optimum water content for the UCS test has been obtained from the standard proctor test (Figure 3.6). After obtaining the optimum water content UCS tests have been conducted on crushed limestone sand-clay mixtures with various ratios (0, 1, 3 and 5%) of xanthan gum according to the ASTM standards. Specimens were prepared by compacting crushed limestone sand-clay mixtures with xanthan gum in wet state in three equal layers in a plastic pipe mould of 54 mm diameter and 100 mm high. The specimens were compacted in the mould using 19 blows per layer satisfactory. The number of blows has been calculated according the energy transformation of standart proctor test. A special hammer and plate system with a plastic pipe have been used (see Figure 3.8). The tests were performed as per procedures described in ASTM D2166 standard. All samples have been unsoaked for 4, 8, 16 and 32 days. Finally, all specimens have been tested after a specified curing period.

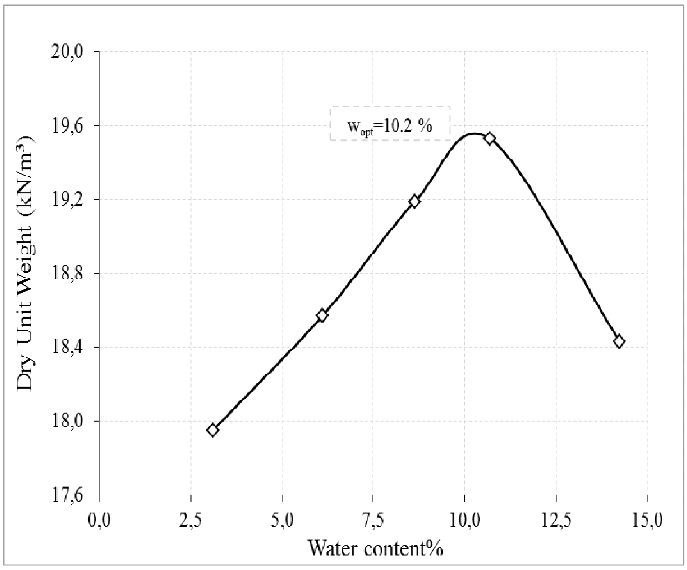


Figure 3.6 Compaction results for UCS test

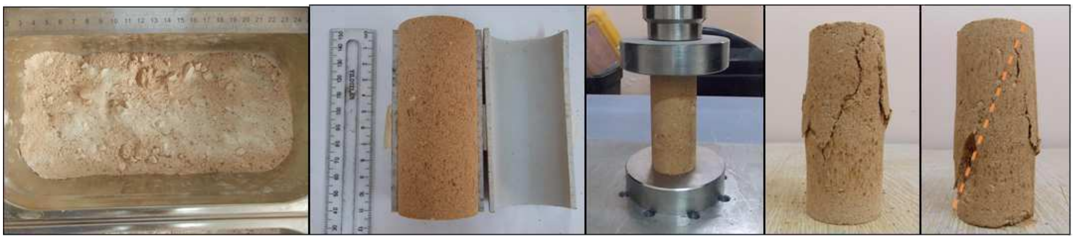


Figure 3.7 Pictures of testing process for UCS

Figure 3.8 Equipments of the UCS test used during the experimental work

3.2 Use of Fly Ash- Lime Mixtures in Stabilization of a Sand

This study was published in the 3rd *International Conference on New Developments in Soil Mechanics and Geotechnical Engineering* Conference in North Cyprus.

3.2.1 Materials and Method

Locally available river sand, collected from Adana region, (Ceyhan) was used in this study. The sand has a specific gravity of 2.68, median grain diameter (D_{50}) of 1.20 mm, coefficient of uniformity (C_u) of 6.87 and coefficient of curvature (C_c) of 1.04. The grain size distribution and the Scanning Electron Microscope pictures of the soil are shown in Figure 3.9. The sand was classified as 'SW' according to the Unified Soil Classification System (USCS). The maximum and minimum dry unit weights of the sand were determined as 19.79 kN/m^3 and 16.73 kN/m^3 respectively. As it can be seen from the Figure 3.10, it is a well- graded sand, between the diameter of 0.075 mm and 4.75 mm. Table 3.3 presents some specifications of the sand.

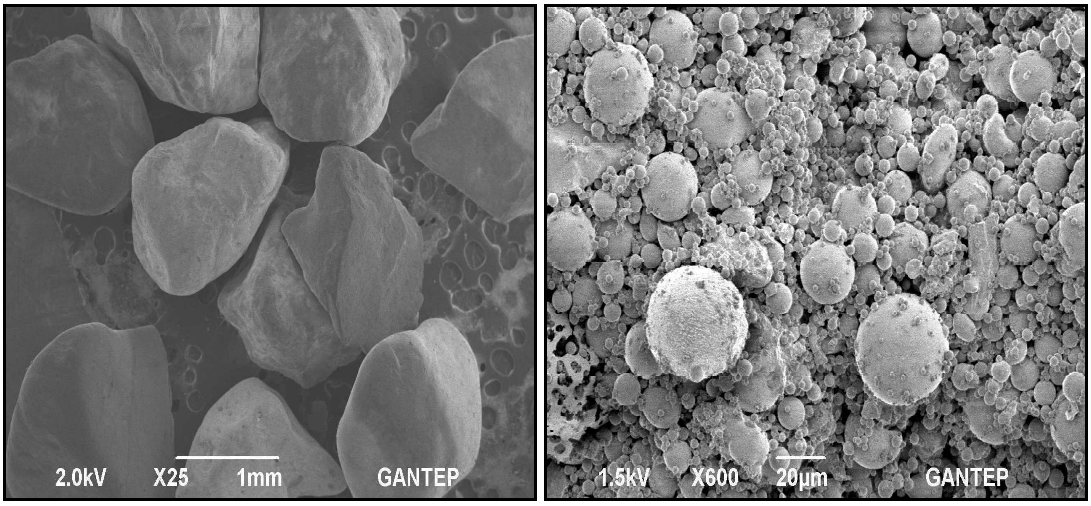


Figure 3.9 SEM picture of the (a) sand, (b) fly ash used during the tests

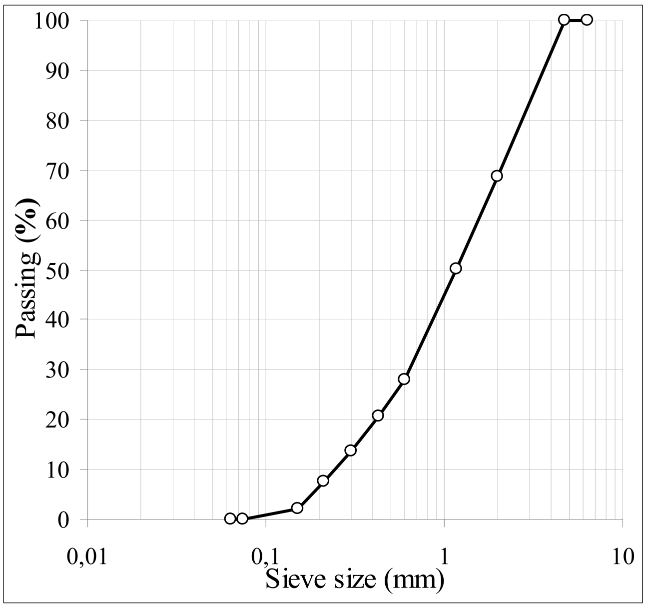


Figure 3.10 Grain size distribution of the sand used during the experimental study

Table 3.3 Some specifications of the sand

Unified Soil Classification System (USCS)	SW
Specific Gravity, G_s	2,68
Maximum void ratio, e_{max}	0,615
Minimum void ratio, e_{min}	0,328
Angle of Friction, ϕ	44^o
Coefficient of uniformity, C_u	6,87
Coefficient of curvature, C_c	1,04
Median Grain Size, D_{50}	1,20
Effective Grain Size, D_{10}	0,24

Fly ash used during the experimental works was obtained from Çatalağzı Thermal Power Station (ÇATES) in Zonguldak, Turkey. According to its chemical composition (26,43% of CaO), the fly ash used is identified as class C, and has a specific gravity (G_s) of 2,10. Table 3.4 presents the composition of the fly ash used in the study. CL 80 S type hydrated lime was also used in this study. It is a product of AKAY Industry and Mining Company and produced in accordance with TS EN 459-1.

Table 3.4 Composition of Zonguldak Çatalağzı fly ash

Composition (%)	**Fly Ash**
CaO	26,43
SiO_2	25,07
Al_2O_3	11,25
Fe_2O_3	12,84
MgO	3,66
SO_3	0,52
K_2O	0,44
Na_2O	0,65
Loss on ignition	18,68
Specific gravity, G_s	2,10
Blaine fineness (m^2/kg)	739

ELE oedometer testing apparatus (ASTM D2435) was used to examine the consolidation characteristics of the sand with fly ash- lime mixtures. The maximum dry unit weight (γ_{drymax}) and optimum water content ($w_{opt,}$ %) for each test was determined by standard compaction test (ASTM 698). ELE falling head permeability testing equipment, a conventional laboratory testing method used to determine the permeability of fine grained soils with intermediate and low permeability, was employed (ASTM D5084). California Bearing Ratio (CBR) test was also performed (ASTM D1883- 99) to obtain CBR values.

3.3 Applications of the Oedometer, Permeability and Direct Shear Tests to the Study of Waste Tire Inclusions- Sand Mixtures

This chapter was published in the *3rd International Conference on Geotechnical Engineering- New Developments in Analysis, Modeling and Design* in Tunisia/Hammamet.

3.3.1 Materials and Method

Two different processed waste tires (i.e., tire crumb, tire buffings) inclusions were used at varying contents as soil reinforcement within a sand, namely 2.5%, 7.5%, and 15% by weight. The sand (Leighton Buzzard Sand B Fraction) used in this study supplied from Cambridge**,** U.K was having a specific gravity of 2.65, median grain diameter (D_{50}) of 0.81 mm, coefficient of uniformity (C_u) of 1.46 and coefficient of curvature (C_c) of 0.93. The grain size distribution of the sand is shown in Figure3.11. The sand was classified as ‘SP’ according to the Unified Soil Classification System (USCS). The maximum and minimum dry densities of sand as determined from the relative density test were 1.45 g/cm^3 and 1.76 g/cm^3, respectively. As it can be seen from the grain size distribution on Figure 3.11, it is poorly graded and between the diameter of 0.6- 1.18 mm, and from Figure 3.12c that the sand particles are rounded a quartz. Some specifications of the Leighton Buzzard Sand Fraction B (LBB) are given below in Table 3.5.

Tire crumb (TC) is a granular material obtained by processing scrap tires. Tire crumb used in this study is purchased from a company in Gaziantep. The grain size distribution curve of tire crumb is shown in Figure 3.11. The tire crumb used in the experiments has an aspect ratio of about 1.0- 1.3.

Tire buffings (TB) are fiber- shaped materials obtained from tire retread process. The grain size distribution of tire buffings is shown in Figure 3.11. Tire buffings used in this study are purchased from a tire retread company in Adana. Tire buffings had various lengths and are obtained by eliminating the ones left above sieve no 16 (1.18mm) and the ones below sieve no 30 (0.6 mm). The buffings used during the experimental studies have an aspect ratio of about 3.2- 3.8. Pictures for a comparison of tire buffings and tire crumbs are shown in Figure 3.12.

In this experimental study, 'ELE Oedometer', 'ELE Permeability' and 'ELE Direct Shear' have been designed in accordance with ASTM D 2435, ASTM D 2434 and ASTM D 3080 respectively.

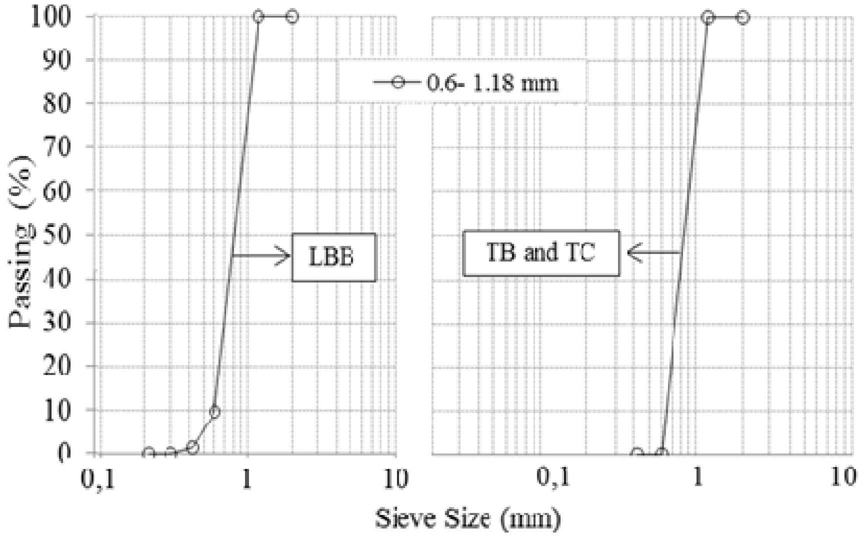
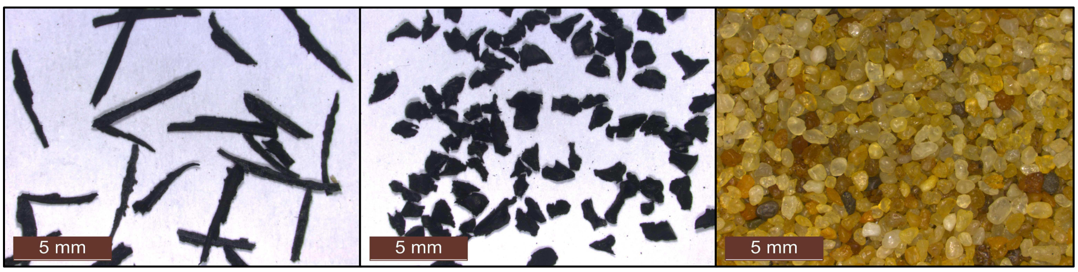


Figure 3.11 Grain size distributions of the materials used during experimental study

Table 3.5 Some specifications of the sand

Unified Soil Classification System (USCS)	SP
Specific Gravity, G_s	2,65
Maximum void ratio, e_{max}	0,83
Minimum void ratio, e_{min}	0,51
Angle of Friction, ϕ	33^o
Coefficient of uniformity, C_u	1,46
Coefficient of curvature, C_c	0,93
Median Grain Size, D_{50}	0,81
Effective Grain Size, D_{10}	0,60



(a) **(b)** **(c)**

Figure 3.12 Picture of (a) tire buffings (fiber), (b) tire crumbs (granular) and (c) Leighton buzzard sand B fraction

CHAPTER IV

RESULTS AND DISCUSSIONS

4.1 Effects of Bio-Polymer on Some Geotechnical Properties of a Sand

The strength and deformation behaviour of biologically cemented sand were determined at various ratios (0, 1, 3 and 5%) of xanthan gum and curing time (4, 8, 16 and 32 days). Using these test results, compressive strength and bearing capacity were determined, which can be used for assessing the stability of (biologically) cemented ground construction or designing a treatment procedure based on the required stability. The tests reported in this study show that the behaviour of sand-clay mixtures with various xanthan gum contents. Testing program and the results obtained from the experiments are shown in Table 4.1 and Table 4.2. The datas in Table 4.1 which are written in paranthesis are given for 5.0 mm CBR results.

Table 4.1 Experintal results of CBR test (MPa)

Mixtures	Curing Time			
	4 days	8 days	16 days	32 days
Control Samples	5.3 (*10.5*)	6.2 (*12.1*)	8.4 (*15.7*)	15.9 (*24.6*)
1% Xanthan Gum	5.9 (*12.7*)	7.3 (*18.0*)	11.2 (*21.2*)	22.7 (*35.9*)
3% Xanthan Gum	14.6 (*32.6*)	21.6 (*35.4*)	29.0 (*42.8*)	36.9 (*54.2*)
5% Xanthan Gum	24.8 (*38.2*)	30.4 (*46.5*)	40.0 (*60.0*)	54.5 (*73.3*)

Table 4.2 Experintal results of UCS test (MPa)

Mixtures	Curing Time			
	4 days	8 days	16 days	32 days
Control Samples	0.11	0.16	0.27	0.46
1% Xanthan Gum	0.17	0.36	0.51	0.67
3% Xanthan Gum	0.28	0.62	0.78	0.88
5% Xanthan Gum	0.36	0.66	0.89	1.02

The compressive strength of the sand with various ratios of xanthan gum was examined by conducting unconfined compression test at 4, 8, 16 and 32 days curing time. As four examples, Figure 4.1 present the stress–strain curves for clean sand with various xanthan gum contents (0, 1, 3 and 5%) at 4, 8, 16 and 32 days of curing time. Addition of xanthan gum to crushed limestone sand in the quantities of 0%,1%, 3% and 5% by weight increased the stress from 0.11 to 0.36 MPa for clean sand samples (Figure 4.1a), 0.17 to 0.67 MPa for 1% xanthan gum samples (Figure 4.1b), 0.28 to 0.88 MPa for 3% xanthan gum samples and from 0.36 to 1.02 for 5% xanthan gum samples. It has been observed that the the increase in microbial xanthan gum increases the strength of samples.

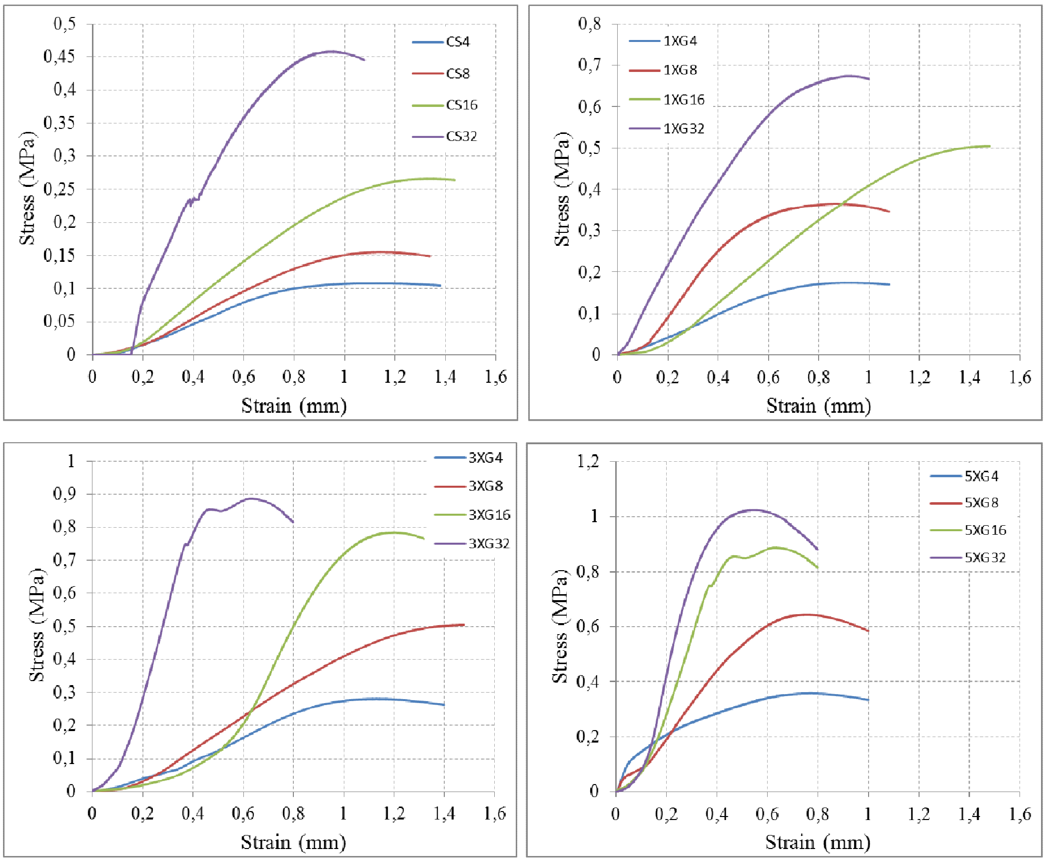


Figure 4.1 Stress- Strain curves of UCS tests

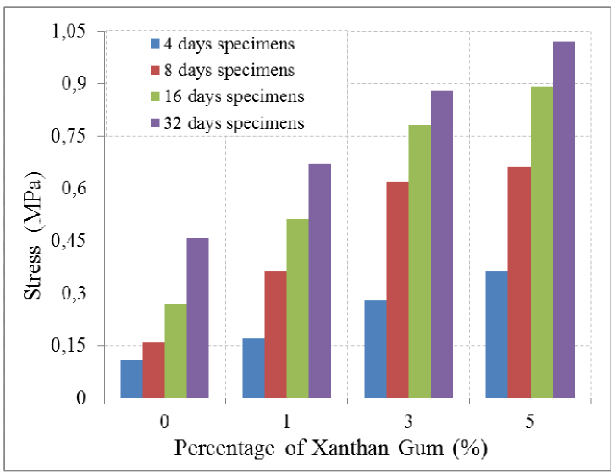


Figure 4.2 Effect of xanthan gum on stress

CBR values increases with incerasing xanthan gum content and curing time. CBR testing results on the sand- clay- xanthan gum mixtures are shown in Figure 4.3 and 4.4. The tests on sand- clay- xanthan gum mixtures at various mix ratios were prepared at the optimum water content determined previously in modified compaction tests. Following the compaction of the specimens at five layers with 56 blows per layer, a surcharge plate of 2.44 kPa was placed on the specimen prior to testing and had been keeping waiting unsoaked. The loads were carefully recorded as a function of penetration up to a total penetration of 6.0 mm. Load penetration curves were drawn for each case, and required corrections were applied based on the procedures identified in standards (Figure 4.3). Then, CBR values obtained from the load-penetration curves were plotted (Figure 4.4). CBR values reported in the present investigation are estimated using the stresses at 5.0 mm penetration, as the CBR values at 5.0 mm penetration was observed higher than that at 2.5 mm penetration.

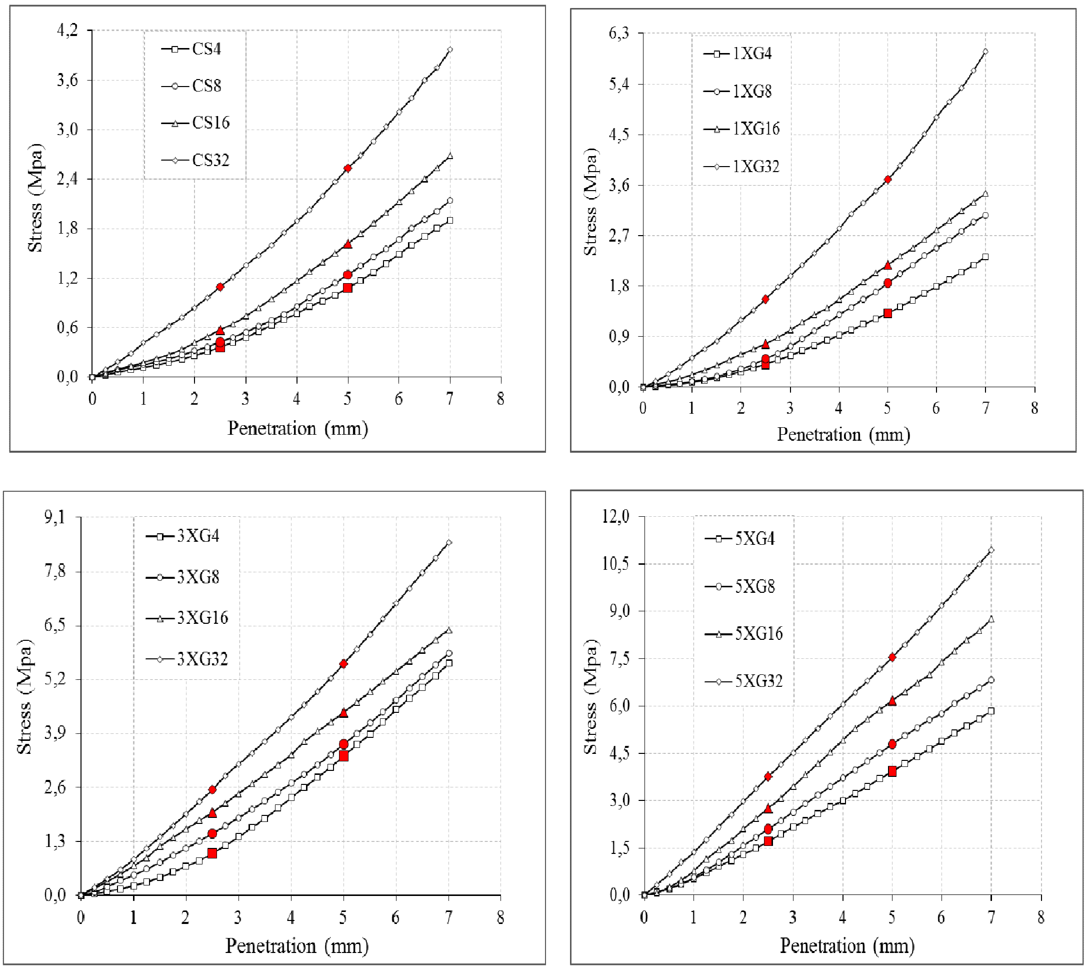


Figure 4.3 Effects of xanthan gum on the penetration vs. stress relationships of the crushed limestone sand with 10% clay

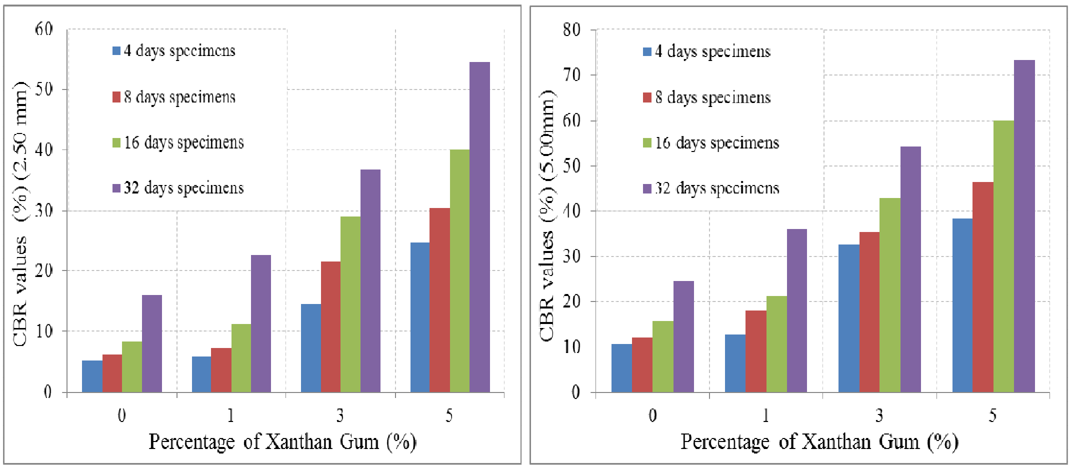


Figure 4.4 Effects of the xanthan gum content on the CBR value of the sand with 10% clay

4.2 Use of Fly Ash- Lime Mixtures in Stabilization of a Sand

Sands with 5% lime were tested in all the testing equipment's by various fly ash mix ratios (5%: FA5 10%: FA10, 15%: FA15, 20%: FA20 and 25%: FA25 by weight). Testing program and the results obtained are shown in Table 4.3

Table 4.3 Summary of specimen data

		FA5	FA10	FA15	FA20	FA25
Maximum dry unit weight, (γ_{drymax}), (kN/m^3)		21,35	20,80	20,33	19,60	18,35
Optimum water content, (w_{opt}), (%)		7,5	8,7	9,1	9,6	10,1
CBR value, (%)		10,3	16,1	23,6	32,2	42,9
Permeability, (k), (m/sec), (x10^{-4})		8,73	5,68	4,05	3,63	2,84
Void ratio, (e)	e_0	0,530	0,600	0,613	0,638	0,657
	e_f	0,459	0,470	0,476	0,500	0,523
Intergranular void ratio, (e_s)	e_{s0}	0,731	0,940	1,104	1,307	1,531
	e_{sf}	0,650	0,784	0,926	1,114	1,328

Results of the oedometer tests conducted in various specimens prepared in loose cases are shown in Figures 4.5 Figure 4.6 and Figure 4.7. Figure 4.5 and Figure 4.6 presents the variation of void ratio (e), intergranular void ratio (e_s) with different oedometer pressures (σ_v). Figure 4.7 presents intergranular void ratio (e_s) vs. fly ash content (%) under different oedometer pressure values. It is seen that the initial void ratio values (e_o) of the mixtures increase with the increasing fly as content. The compressibility of the sand- lime- fly ash mixture increases with the increasing fly ash content. Similarly, e_s values increases as the fly ash content increases at all oedometer pressure values. The readers are referred to the study by Monkul (2005) for more details in estimating the e_s values. The tests were carried out immediately after the specimens had been prepared in the testing equipment. Therefore, it should be kept in the mind that it could be expected to have different results at different curing times.

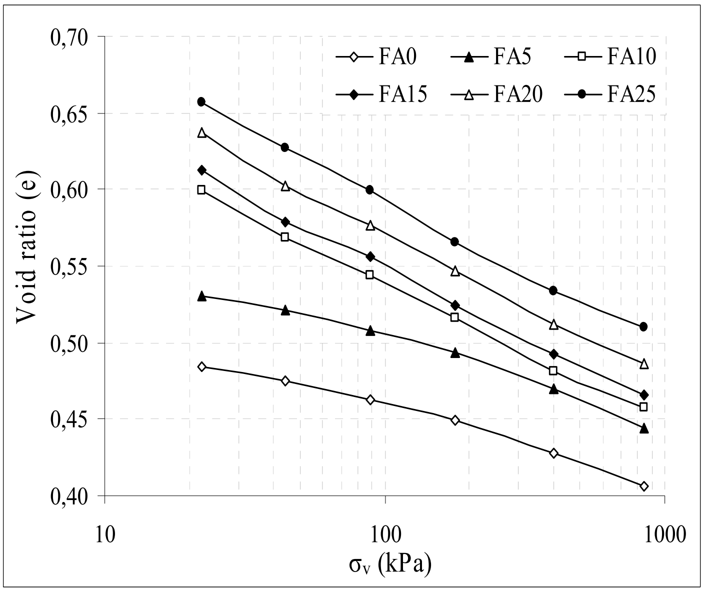


Figure 4.5 Variation of void ratio with different oedometer pressures

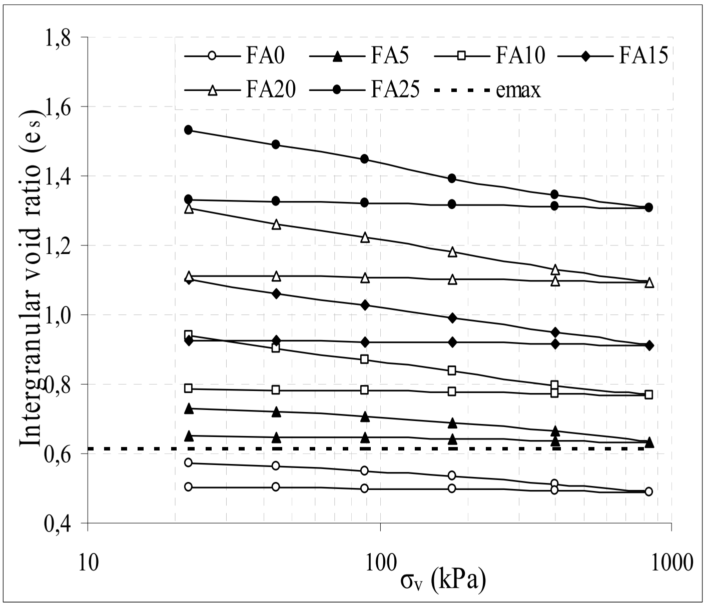


Figure 4.6 Variation of intergranular void ratio with different oedometer pressures

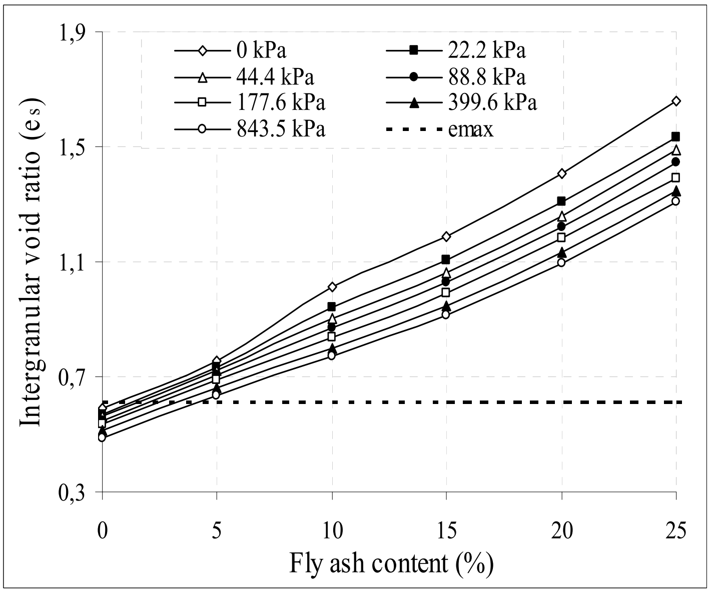


Figure 4.7 Intergranular void ratio vs. fly ash content under different stresses

From the standard proctor test results (Figure 4.8) it is seen that addition of fly ash increases the optimum water content (w_{opt}) and decreases the maximum dry unit weight (γ_{drymax}). Also, a less variation of dry unit weight over a much wider range of water contents is observed, as the fly ash content increases. This could be because of the presence of lower specific gravity fly ash.

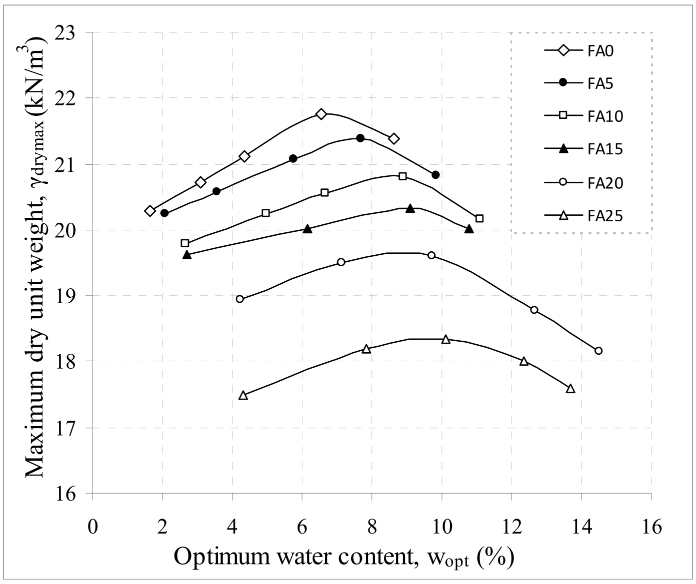


Figure 4.8 Effect of fly ash on the sand with lime in compaction tests

Based on the tests employed in the falling head permeability testing equipment, a sharp decrease in permeability (k) values with an increase in fly ash content is observed (Figure 4.9). This could be because of the filling of voids, and/or cementations taken place between the sand grains. Completing the procedures required to be followed for standard compaction tests, and waiting for two days in a soaked case, the falling head permeability tests were carried out to determine the k values.

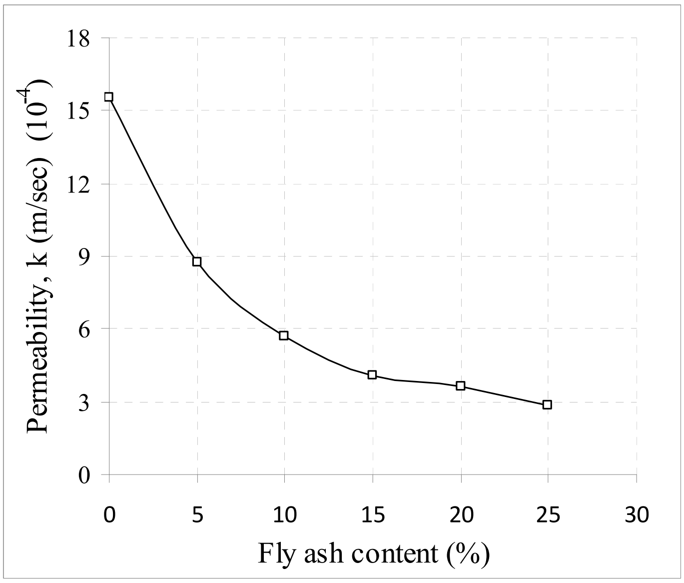


Figure 4.9 Effect fly ash on the sand with lime in coefficient of permeability (k)

CBR values increases with increasing fly ash content. CBR testing results on the sand- lime- fly ash mixtures are shown in Figures 4.11. The tests on sand- lime- fly ash mixtures at various mix ratios were prepared at the optimum water content determined previously in compaction tests. Following the compaction of the specimens at five layers with 56 blows per layer, a surcharge plate of 2.44 kPa was placed on the specimen prior to testing. The loads were carefully recorded as a function of penetration up to a total penetration of 6.0 mm. Load penetration curves were drawn for each case, and required corrections were applied based on the procedures identified in standards (Figure 4.10). Then, CBR values obtained from the load-penetration curves were plotted (Figure 4.11). CBR values reported in the present investigation are estimated using the stresses at 5.0 mm penetration, as the

CBR values at 5.0 mm penetration was observed higher than that at 2.5 mm penetration.

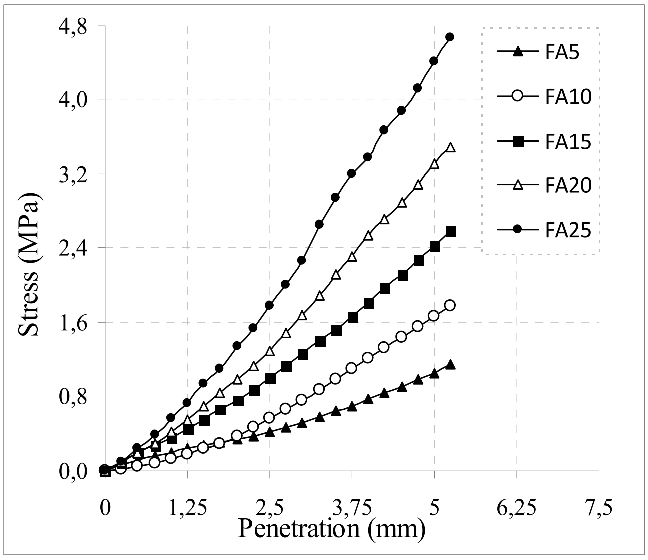


Figure 4.10 Effects of fly ash on the penetration vs. stress relationships of the sand with 5% lime

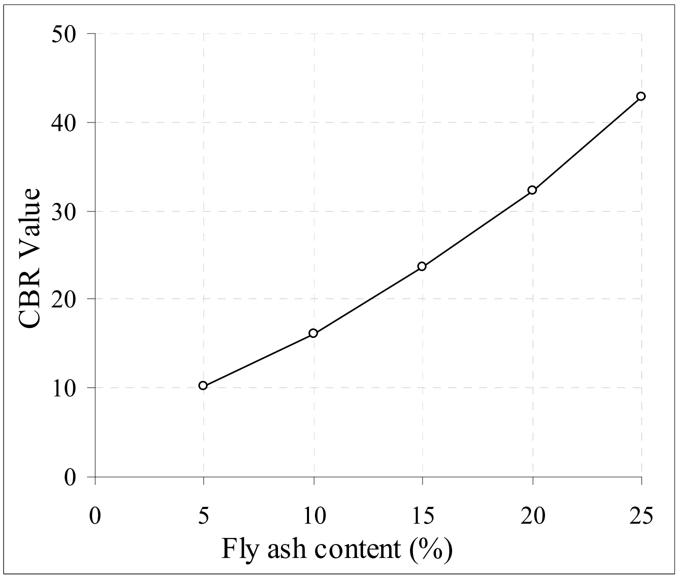


Figure 4.11 Effects of the fly as content on the CBR value of the sand with 5% lime

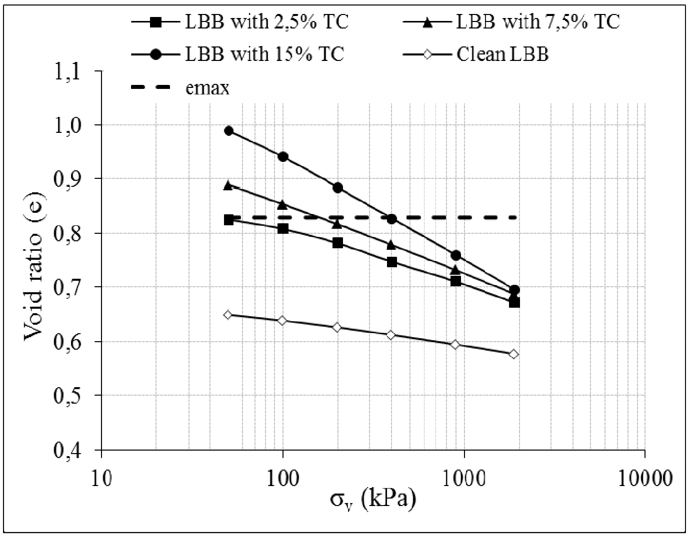
4.3 Applications of the Oedometer, Permeability and Direct Shear Tests to the Study of Waste Tire Inclusions- Sand Mixtures

Mixture characteristics, load- deformation response, compressibility and permeability are reported for all tested mixtures in this section. From the 1-D compression results (Figure 4.12a, 4.12b) it was found that the presence of both waste tires in the specimens tested had a marked effect on the compressibility of the material under load. The author postulates that waste tires particles occupy the voids between coarse rotund sand particles. Based on the amount of waste tires particle available, the Leighton Buzzard Sand particles are in contact with each other and the behaviour of the samples tested are controlled by the Leighton Buzzard Sand particles. When the contacts between the Leighton Buzzard Sand particles reduce, the behaviour of the samples becomes to rubber like. Void ratio with fines content and oedometer stress are shown in Figure 4.12. As can be seen from the Figure 4.12, initial void ratios for the samples are scattered between 0.65 and 1.1 because of their initial conditions. The governing role of either waste tires or sand grain matrices on the overall behaviour of the sample should be expected to change during one dimensional compression. The interchange of this governing role can be expressed using intergranular void ratio concept. Monkul and Onal (2006) proposed an equation for calculation of the intergranular void ratio as follows:

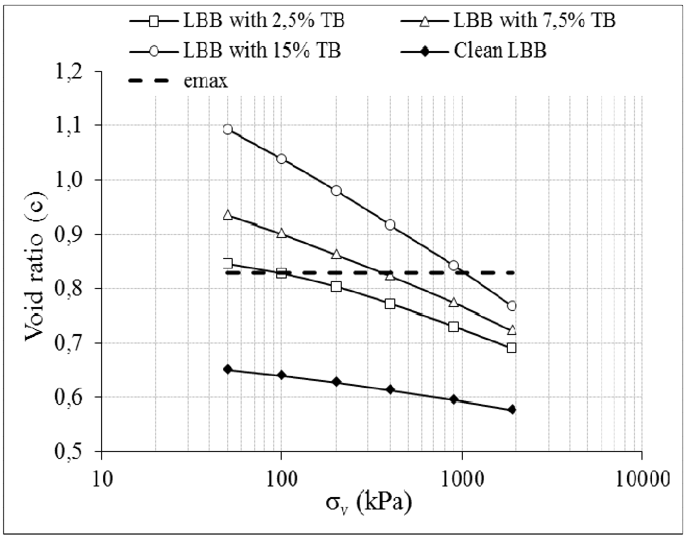
$$e_s = \frac{e + \dfrac{G_{mix} \cdot FC}{G_f \cdot 100}}{\dfrac{G_{mix}}{G_s} \cdot \left(1 - \dfrac{FC}{100}\right)}$$

G_s and G_f in the above equation are the specific gravity of sand grains and waste tires forming the soil respectively. G is the specific gravity of soil itself, which is assumed to be the weighted average of the specific gravities of the grains forming the mixtures. FC is the fines content in the mixture. Using the equation above, intergranular void ratios for the samples in this study are found to be kept in a larger band (i.e. 0.65–1.93) than the void ratios (Figure 4.13).

Transition waste tires content (FC_t) values vary between 0.80% and 3.00% for tire buffings, 1.15%- 3.80% for tire crumbs depending on the applied stress (Table 4.4). FC_t values increase as the effective stress increases. That means, under higher stress, transition arrangement of the Leighton Buzzard Sand grain matrix takes place at higher values of both waste tires content. The same intergranular void ratio (e_s) can be observed at different combinations of the waste tires content (FC) and global void ratio (e) (Monkul and Ozden, 2004). In other words, intergranular void ratio value of 0.83 (e_{max}), where transition occurs, can be observed with a higher transition waste tires content and a lower global void ratio combination with increase in effective stress. The results tabulated in Table 4.4 have been estimated using the intergranular void ratio vs. tire content under different stresses (a) tire crumb, (b) tire buffing (Figure 4.14).



(a)



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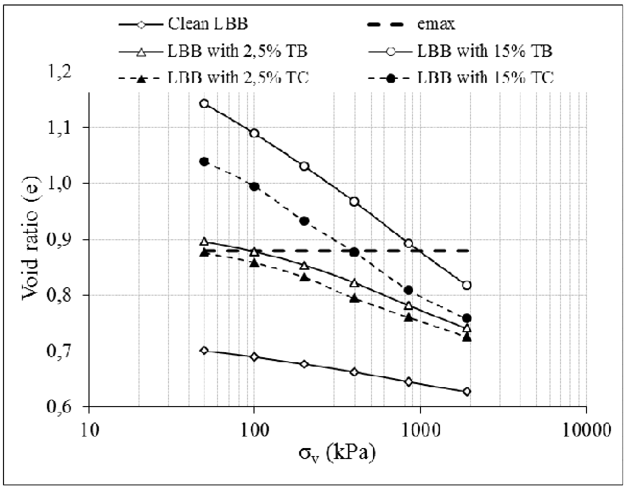
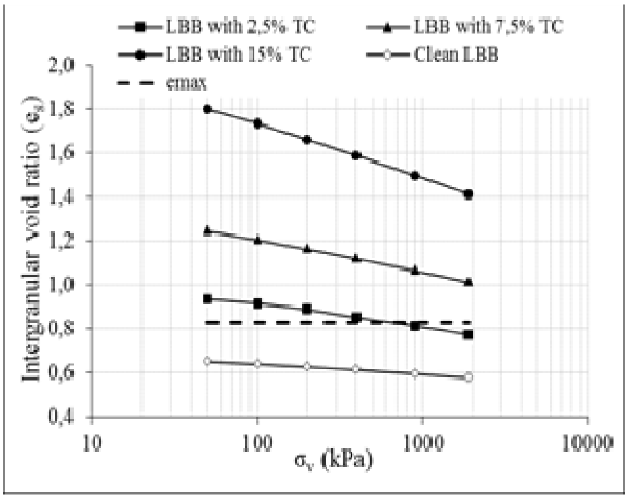
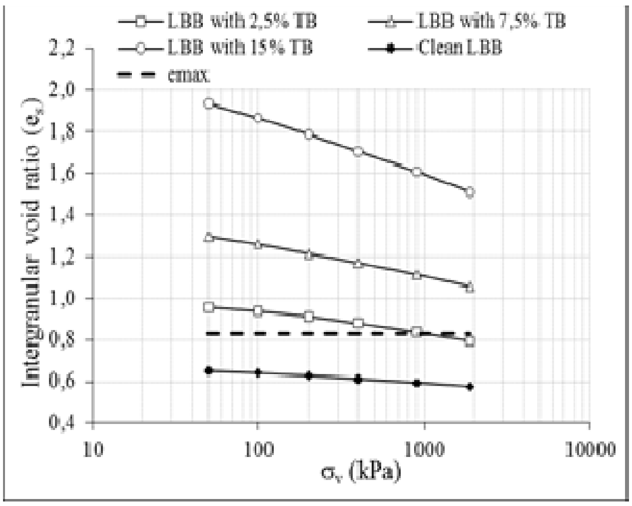


(c)

Figure 4.12 Variation of void ratio with different oedometer pressures, (a) tire crumb, (b) tire buffing, (c) comparison for both type of waste tires

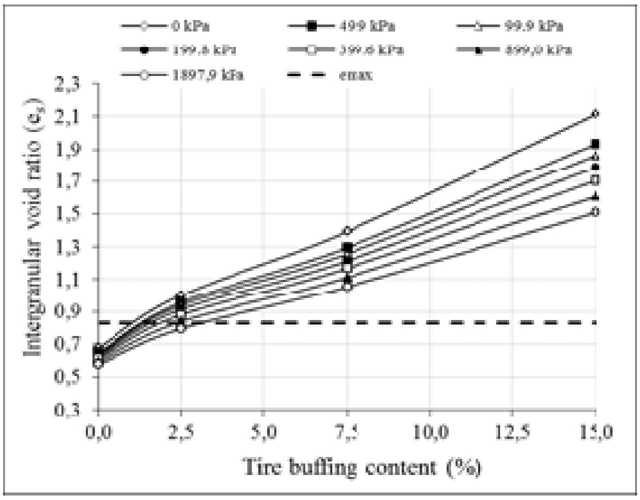


(a)

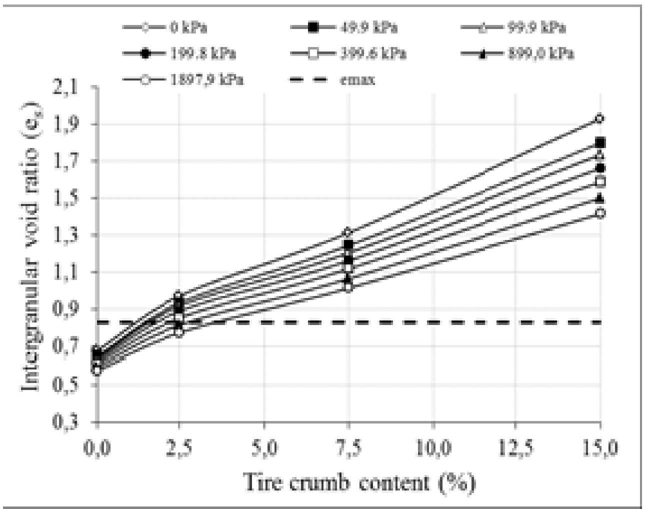


(b)

Figure 4.13 Variation of intergranular void ratio with different oedometer pressures, (a) tire crumb, (b) tire buffing



(a)



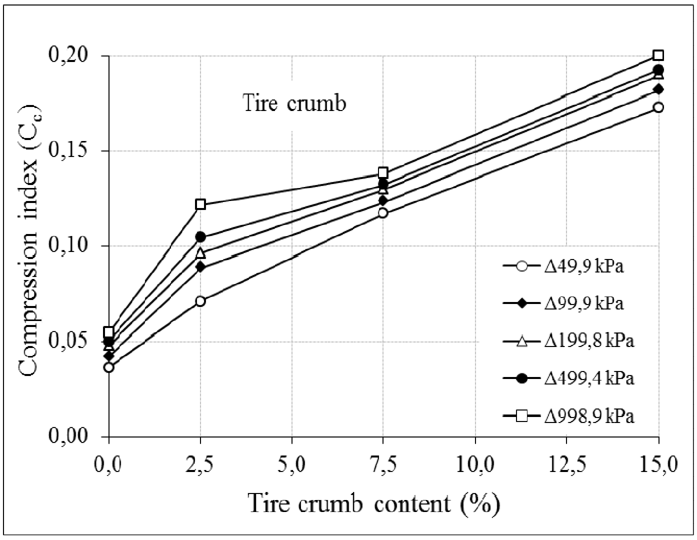
(b)

Figure 4.14 Intergranular void ratio vs. tire content under different stresses (a) tire crumb, (b) tire buffing

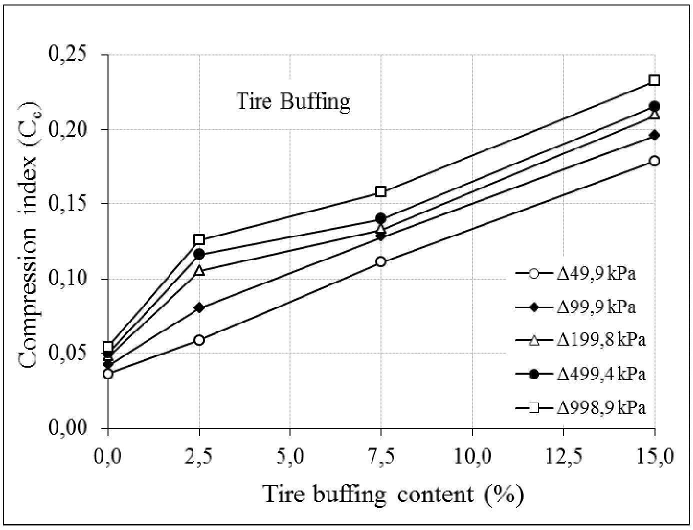
Table 4.4 Tire transition zones under different stresses

Vertical effective stress, σ_v (kPa)	Tire buffing	Tire crumb
	FCt (%)	FCt (%)
0	0,80	1,15
49,9	1,10	1,30
99,9	1,15	1,35
199,8	1,30	1,75
399,6	1,75	2,00
849,0	2,00	2,90
1897,9	3,00	3,80

Variation of global compression index (Cc) with waste tires content is shown in Figure 4.15. C_c values are calculated for 49.9 kPa and 998.9 kPa effective stresses. Compression indices of both sets of data increase relatively linearly with waste tires content. The C_c values for TB tests varied between 0.058 and 0.125 for 2.5% tire content, 0.111 and 0.157 for 7.5% tire content, and 0.178 and 0.232 for 15% tire content. The C_c values for TC tests varied between 0.070 and 0.120 for 2.5% tire content, 0.117 and 0.138 for 7.5% tire content, and 0.173 and 0.200 for 15% tire content.



(a)



(b)

Figure 4.15 Compression index vs. tire content under different stress changings (a) tire crumb, (b) tire buffing

Actually, it is important that the waste tires are used for drainage material in various engineering applications. Constant head permeability tests have been carried out to determine the coefficient of permeability (k, hydraulic conductivity) of on the sand-waste tires mixtures. The obtained results vary between 0.072- 0.083 for tire buffings, 0.072- 0.078 for tire crumbs. Reddy and Marella (2001) stated that the wide range of hydraulic conductivity values of the waste tires can be found that could be attributed to teh differences in waste tires and compositon, compaction level, and normal stress. Considering the density values in the Table 4.3, results are accordence with the already available information in the literature (Figure 4.16).

Table 4.5 Hydraulic conductivity values for different shape of waste tire

Waste tire type	0,00%		7,50%		15,00%	
	k (cm/s)	ρ (g/cm^3)	k (cm/s)	ρ (g/cm^3)	k (cm/s)	ρ (g/cm^3)
Tire Buffing (TB)	0,072	1,44	0,081	1,25	0,083	1,09
Tire Crumb (TC)	25,07	1,44	0,076	1,31	0,078	1,12

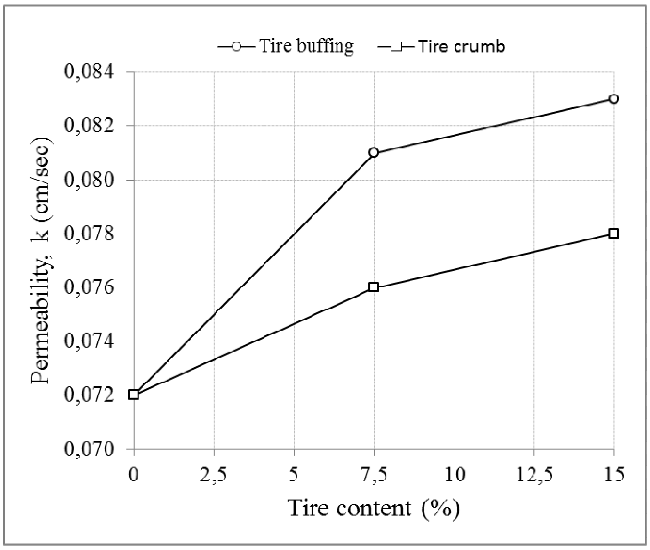


Figure 4.16 Coefficient of permeability (k) values for the materials tested.

The shear strength of the sands with waste tire was measured by conducting direct shear test at 27.3, 40.9 and 68.1 kPa normal stresses. As three examples, Figures 4.17, 4.18 and 4.19 present the maximum shear stress–horizontal displacement curves for clean sand with various waste tire contents (0, 2.5, 7.5 and 15%) at 27.3, 40.9 and 68.1 kPa vertical stresses for both waste tires.

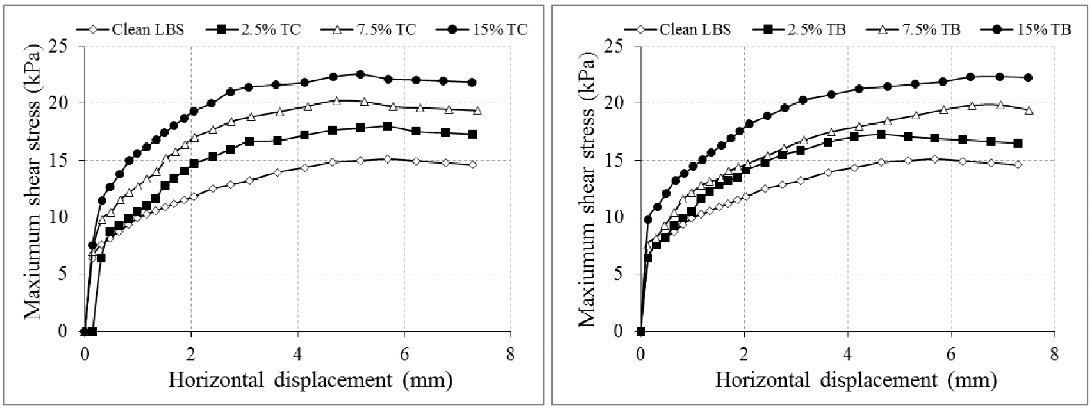


Figure 4.17 Maximum shear stress–horizontal displacement curves for both waste tires with various mix ratios at vertical stress of 27.3 kPa

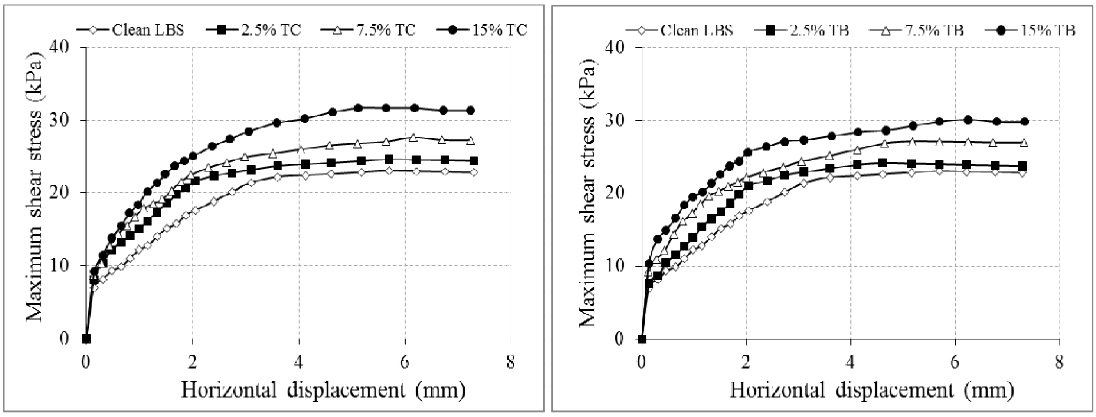


Figure 4.18 Maximum shear stress–horizontal displacement curves for both waste tires with various mix ratios at vertical stress of 40.9 kPa

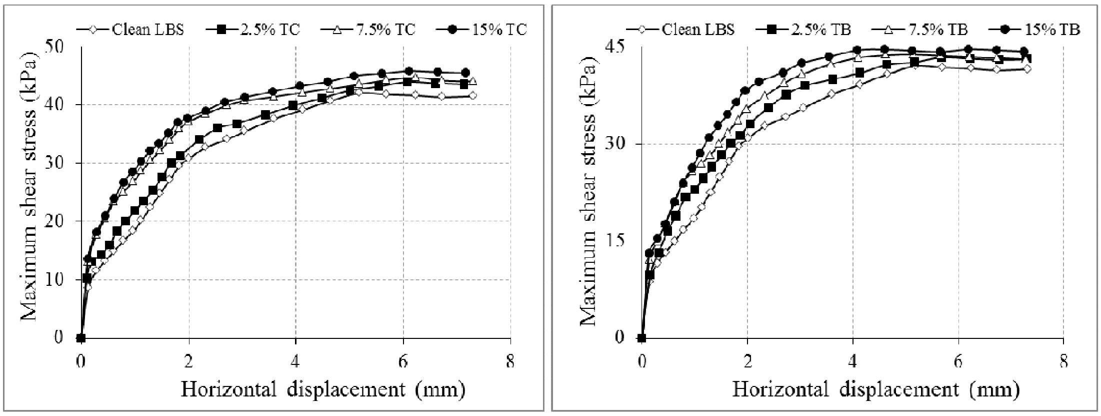


Figure 4.19 Maximum shear stress–horizontal displacement curves for both waste tires with various mix ratios at vertical stress of 68.1 kPa

Internal angle friction and cohesion results of the samples are presented in Table 4.6 and Table 4.7.

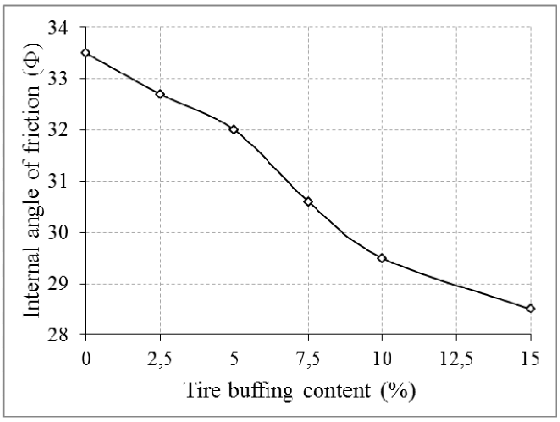
Table 4.6 Direct shear test results of tire buffing

		0%	2.5%	5.0%	7.5%	10%	15%
Angle of friction, (Φ)		33,5	32,7	32,0	30,6	29,5	28,5
Cohesion, (c), (kPa)		0	0	0,6	3,4	5,7	7,6
	27.3	15,1	17,3	18,5	19,8	20,9	22,3
Load, (kPa)	40.9	23,1	24,1	24,9	27,1	29,3	30,1
	68.1	41,9	43,1	43,6	43,8	44,1	44,6

Table 4.7 Direct shear test results of tire crumb

		0%	2.5%	5.0%	7.5%	10%	15%
Angle of friction, (Φ)		33,5	32,9	32,5	31,2	30,3	29,5
Cohesion, (c), (kPa)		0	0	0,8	3,3	5,4	7,7
	27.3	15,1	17,9	18,8	20,2	21,0	22,5
Load, (kPa)	40.9	23,1	24,6	25,4	27,6	29,7	31,7
	68.1	41,9	43,9	44,2	44,8	45,0	45,9

Increasing the percentage of the tire content decreased the internal angle of friction for both waste tire types (Figure 4.20). As can be seen from the Figure 4.21, amount of decrease in internal friction angle for tire buffing is higher than that of the tire crumb, which could be because of the waste tire shape.



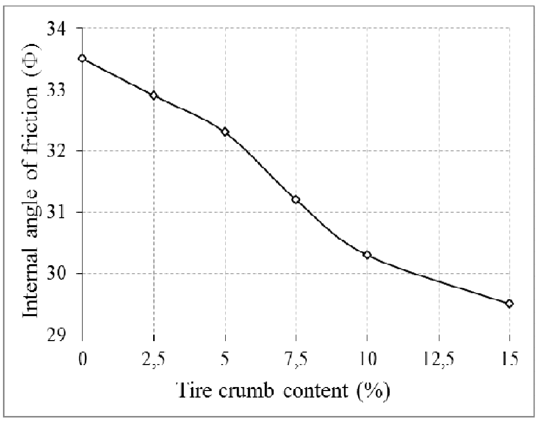


Figure 4.20 Effect of tire content on the internal angle of friction for (a) Tire buffing and (b) Tire crumb

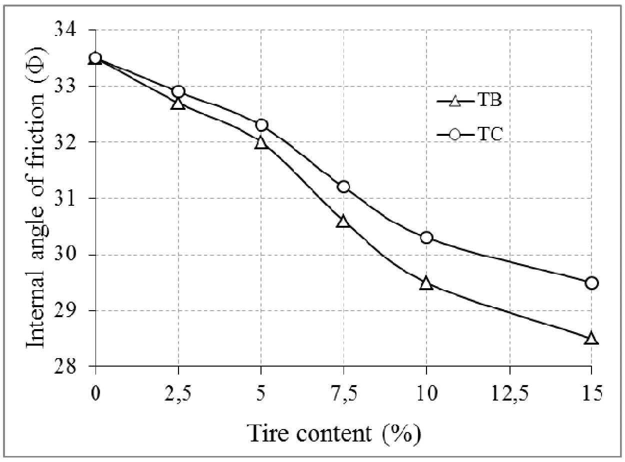


Figure 4.21 Comparison for the internal angle of friction for both waste tires

It is observed in the Figure 4.22 that the cohesion of mixtures is increasing with an increase in the tire content. As overall, the waste tire shape does not change significantly the cohesion values (Figure 4.23).

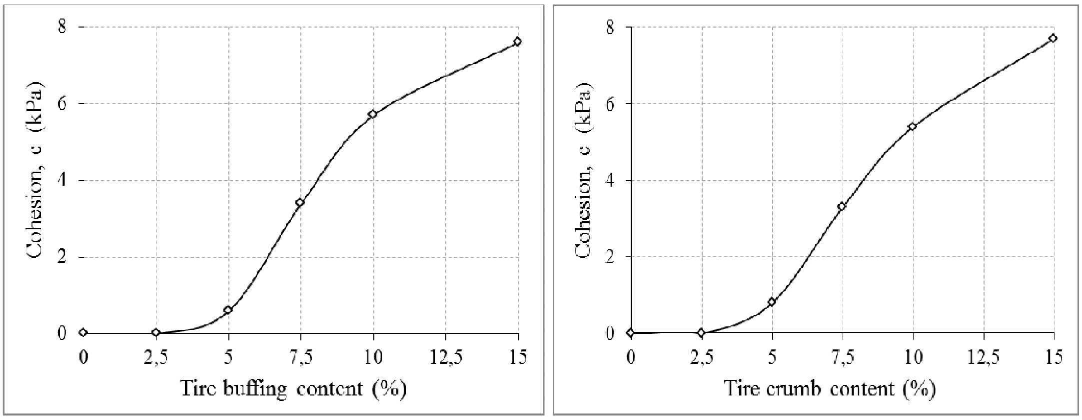


Figure 4.22 Effect of tire content on cohesion for (a) Tire buffing and (b) Tire crumb

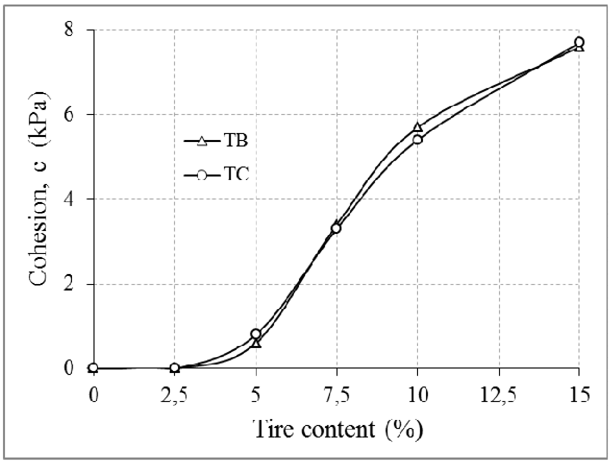


Figure 4.23 Comparison for the cohesion for both waste tires

CHAPTER V

CONCLUSIONS

Some applications of ground improvement were performed in various sands during this study. Consolidation, standard proctor, falling head permeability, unconfined compression and California Bearing Ratio tests were completed to investigate some engineering properties of various sand mixtures. These sand mixtures were derived by bacterial product (xanthan gum), fly ash and waste tire. To perform these tests, methods for mixing, was created to usefully describe the mixtures. One common method of ground improvement utilizes cementation of soil particles is to improve the strength and deformation behaviour of soil. Various chemical soil cementation techniques are currently used in practice, many of which have adverse environmental effects.

The goal of the research program that is described here is to enhance the mechanical properties of soil naturally. It has been shown that biologically induced cementation can significantly improve mechanical properties of a sand. The strength and deformation behaviour of biologically cemented sand were determined at various ratios (0, 1, 3 and 5%) of xanthan gum and curing time (4, 8, 16 and 32 days). Using these test results, compressive strength and bearing capacity were determined, which can be used for assessing the stability of (biologically) cemented ground construction or designing a treatment procedure based on the required stability. The tests reported in this study show that the behaviour of sand- clay mixtures with various xanthan gum contents leads to these conclusions;

1. Some improvements in CBR and UCS tests are shown to be associated with an increase in xanthan gum content in the specimens of sand with 10% clay.

2. From the control samples (CS) which are not containing xanthan gum shows an increase with curing time. Therefore, it should be kept in mind that even without

any mixing material that it could be expected to have different results at different curing times.

3. Addition of xanthan gum increases the strength of unconfined compression samples. This increment is between 0.11-0.36 MPa for 4 days cured samples, 0.16-0.66 MPa for 8 days cured samples, 0.27-0.89 MPa for 16 days cured samples and 0.46-1.02 MPa for 32 days cured samples

4. It has been observed that the CBR values of 5.00 mm penetration are higher than 2.50 mm for all mixture and curing times.

5. CBR values increases as the xanthan gum content increases. This increment also increases with the increase of curing time

6. Unconfined compression and CBR mixtures form an extensive cementation when have been cured and mixed with xanthan gum. This cementation induces the improvement that is seen in compressive strength and bearing capacity.

7. Unconfined compression and CBR tests with high xanthan gum percentages show an increment in stiffness, compressive strength and bearing capacity at high xanthan gum concentrations in comparison to that with low xanthan gum percentages. Furthermore the compressive strength and bearing capavity is to be associated with an increase in curing time.

8. Some improvements are shown to be associated with an increase in fly ash content in the specimens of sand with 5% lime.

9. Initial void ratio (eo), and intergranular void ratio values of the mixtures increase with the increasing fly ash content at all oedometer pressure values employed in the experimental studies.

10. Addition of fly ash increases the optimum water content (wopt) and decreases the maximum dry unit weight (γdrymax).

11. A decrease in permeability (k) values with an increase in fly ash content is observed.

12. CBR values increases as the fly ash content increases.

13. The fly ash tests which were carried out immediately after the specimens had been prepared in the testing equipment and therefore, it should be kept in the mind that it could be expected to have different results at different curing times.

14. The presence of both waste tires in the specimens tested had a marked effect on the compressibility of the material under load. Initial void ratio (eo), and intergranular void ratio (es) values of the mixtures increase with the increasing tire content at all oedometer pressure values employed in the experimental studies.

15. Transition waste tires content (FCt) values vary between 0.80% and 3.00% for tire buffings, 1.15%- 3.80% for tire crumbs depending on the applied stress.

16. Compression indices of both sets of data increase relatively linearly with waste tires content. The Cc values for TB tests varied between 0.058 and 0.232 for various tire content. The Cc values for TC tests varied between 0.070 and 0.200 for various tire content.

17. An increase in permeability (k) coefficient with an increase in tire content is observed. Tire buffings are more effective on the permeability results comparing to the results obtained using tire crumbs. The author considers that this could be mainly due to the differences in shape of the tires.

18. Increasing the percentage of the tire content decreased the internal angle of friction for both waste tire types.

19. It is observed that the cohesion of the mixtures with various ratios of tire content is increasing with an increase in the tire content.

20. Waste tire shape does not change significantly the cohesion for both waste tire types.

The cementation of sands and how it varies with degree of cementation is a very interesting question to explore. As indicated in the results of this study further work is needed to understand cementation at various levels. Laboratory tests probing different biological products, different curing times, to define valuable information.

Dynamic properties of the test (damping and modulus reduction) should also be examined using resonant or torsional shear tests.. This detailed laboratory investigation would be needed to guide selection of numerical models for predicting the performance of the mixtures via simulations. Numerical simulations could be used to understand and predict the effects of bio-treatment, fly ash and waste ire inclusion on the dynamic response of the soil and structures (a site response analyses). Soil improvement has many potential applications and each of these require research test trials for the specific treatment delivery methods needed. For the field to advance, test sites will need to be performed for the in situ treatment to be evaluated and improved for industry use.

It is also important to note that the tests reported in this study do not provide a complete simulation of the actual loading, geometry and environmental conditions experienced by field situations. To increase confidence in predicting the behavior of soils field tests are needed to be carried out to obtain realistic and consistent results. The results reported in the present thesis provide only a preliminary database for further researchers.

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