Performance Evaluation of Problematic Soils Reinforced with Plastic Wastes

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ABSTRACT

Over the past years, because of the increase in the environmental and economic issues, there has been an increasing interest in the use of waste products for stabilization of problematic soils such as soft clays, expansive soils, liquefiable soils, etc. The use of plastic water bottles has been growing very fast and a huge amount of plastic wastes has been produced every year. The aim of this study is to recycle the empty plastic water bottles as a reinforcing material for the stabilization of problematic soils; Bedis beach sand and soft silty Tuzla clay. The laboratory tests were performed on reinforced soils with different plastic waste percentages: 0.5%, 0.75% and 1.0% of dry weight of soil and at two relative density states: 30% and 60%. For Bedis beach sand, direct shear box test was performed for determining the shear strength parameters and California Bearing Ratio, CBR test was performed for determining the bearing capacity of soil. For Tuzla soil, one dimensional swell and consolidation, unconfined compression, and California Bearing Ratio tests were performed. Test results indicated that reduction in the unconfined compressive strength of the reinforced Tuzla soil was obtained because of the reduction in the dry density of the reinforced soil. However, recycling plastic bottles as reinforcing material in the same soil improved the volume change behaviour. With only 0.75% plastic waste reinforcement, reduction in the swelling and the compressibility characteristics of the Tuzla soil was obtained.

Keywords: California bearing ratio, cohesion, direct shear box, loose sand, recycling, shear strength, plastic waste.

ÖZ

Son yıllarda, çevresel ve ekonomik konulardaki artış nedeni ile, yumuşak killer, kabaran zeminler ve sıvılaşabilir topraklar gibi problemli zeminlerin stabilizasyonunda atık ürünlerin kullanımında giderek artan bir ilgi oluşmuştur. Plastik su şişeleri kullanımı çok hızlı bir şekilde artmaktadır ve her yıl plastik atıklar büyük miktarda üretilmektedir. Bu çalışmanın amacı boş plastik su şişelerini geri dönüşüm amaçlı gevşek kum: Bedis sahil kumu ve yumuşak siltli kil: Tuzla zemini ıslahı için iyileştirme malzemesi olarak kullanmaktır. Laboratuar deneyleri takviye edilmiş toprak zemin üzerinde farklı plastik atık şişe oranlarında: zemin kuru ağırlığının % 0.5% 0.75 ve % 1.0 ve farklı rölatif sıkılık oranlarında: % 30 ve% 60 gerçekleştirilmiştir. Bedis kumu için direkt kesme kutusu ve Kaliforniya Taşıma Oranı, CBR testleri, kayma direnci parametrelerini belirlemek için yapılmıştır. Tuzla zemini için, tek boyutlu kabarma ve konsolidasyon, serbest basınç, ve Kaliforniya Taşıma Oranı testler uygulanmıştır. Test sonuçları takviye malzemesi olarak kullanılan geri dönüşüm plastik su şişe malzemelernin Bedis kumunun maksimum ve kritik içsel sürtünme açılarında artışa neden olduğunu göstermektedir. Bedis kumu için kayma direnci parametrelerinin iyileştirilmesinde optimum plastik atık yüzdesinin % 0.75 olduğu saptanmıştır diğer yandan CBR için optimum plastik atık yüzdesi % 0.5 olarak belirlenmiştir. Test sonuçları takviyeli Tuzla toprağının kuru yoğunluğundaki azalma nedeni ile takviyeli toprağın serbest basınç dayanımında azalma olduğunu göstermiştir. Ancak, aynı toprakta, plastik şişe geri dönüşüm malzemesinin takviyeli zeminin hacim değişikliği davranışını geliştirdiğini göstermiştir. Yalnızca % 0.75 plastik atık donatılı toprakta, Tuzla zemininin şişme ve sıkışabilirlik özelliklerinde azalma elde edilmiştir.

Anahtar Kelimeler: California taşıma oranı, kohezyon, direk kesme kutusu, gevşek kum, geri dönüşüm, kayma mukavemeti, plastic atık.

I dedicate this thesis for my parents Dr. Esameldin Farah, Mrs. Huwida Hassan and my siblings Ramah, Farah, Hassan, Ahmed and Mohammed.

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c	Cohesion
C_c	Coefficient of curvature
$\mathrm{C_c}$	Compression index
$\mathrm{C_s}$	Swelling index
$\mathrm{c_u}$	Coefficient of uniformity
$\mathrm{C_v}$	Coefficient of consolidation
˚C	Celsius
e	Void ratio
$\mathrm{e_{max}}$	Maximum void ratio
$\mathrm{e_{min}}$	Minimum void ratio
$\mathrm{G_s}$	Specific gravity
K	Hydraulic conductivity
$\mathrm{m_v}$	Compressibility coefficient
$\mathrm{q_u}$	Unconfined compressive strength
τ	Shear strength
SP	Poorly graded sand
$\mathrm{W_{op}}$	Optimum moisture content
$\rho_{\mathrm{d(max)}}$	Maximum index density
$\rho_{\mathrm{d(max)}}$	Maximum dry density
$\rho_{\mathrm{d(min)}}$	Minimum index density
ϕ_{peak}	Peak internal friction angle
ϕ_{cri}	Critical internal friction angle
σ	Normal stress

σ_p' Preconsolidation pressure

LIST OF ABBREVATIONS

ASTM	American society for testing and materials
CBR	California bearing ratio
CL	Inorganic clay of low plasticity
DEM	Discrete element method
LL	Liquid limit
Ls	Linear shrinkage
MDD	Maximum dry density
ML	Inorganic silt of low plasticity
OMC	Optimum moisture content
PET	Polyethyethylene terephthalate
PI	Plasticity index
PL	Plastic limit
UCS	Unconfined compressive strength

Chapter 1

INTRODUCTION

1.1 Aim of the study

The notion of reinforcing soil dramatically alters the behaviour of soil as a construction material. The introduction of the soil reinforcing methods enabled engineers to effectually use problematic soils as trustworthy construction materials in a spacious range of civil engineering applications. The construction on Reinforced soil is an effectual and trustworthy technique for enhancing the strength and stability of soils. The method of reinforcement is utilized in a several of applications varying from retaining structures and embankments to subgrade stabilization beneath footings and pavements. In the literature, very limited studies are accessible on the utilizing of plastic waste mixed with soil. The bottled water is the most brisk growing drinkables industry in the world. The recycling of the waste of water bottles has become one of the major outface worldwide. The potential advantages of using the waste of water bottles are that the plastic waste can be consumed in beneficial geotechnical engineering applications. The exploiting of plastic waste in blending with soil indicates that one of the most promising approaches is the use of fiber-shaped waste materials in the combination soil (Babu and Chouksey, 2010).

North Cyprus is a country surrounded by Mediterranean Sea, which has cities confronted sea costal such as Famagusta and Girne. Those areas are needed for civil engineering construction. They are, near to the sea are covered by sandy soil. Those sandy soils in some locations are weak and need some improvements before constructing on them. The areas near the sea have fascinating views and many buildings and hotels are preferable built near the sea. However, North Cyprus is covered by number of problematic soils such as swelling soils, liquefiable soils and highly compressible soft clays etc. Those soils threats the civil engineering structures because of their deficient properties such as low shear strength, high compressibility, and expansiveness behaviour. For those reasons, the deficient soils requires improvement in order to get safe ground to construct on.

This research work will examine the approach of using plastic waste as reinforcement material in different types of soils. The plastic waste mixed soil behaves as reinforced soil, similar to fiber- reinforced soils. In this study, different percentages of plastic waste will be mixed with the soil and the strength and the compressibility of the reinforced soil will be investigated. The strength, compressibility, bearing capacity improvement and settlement reduction of the soil reinforced with plastic waste will be in the design of shallow foundations. This investigation includes detailed outcomes on the effect of plastic waste chips of water bottles on the shear strength of a sandy soil and engineering properties of a silty clay soil such as linear shrinkage, unconfined compressive strength, California bearing ratio, swelling and consolidation. The influence of plastic chips of plastic bottles on different problematic soils have been investigated by previous researchers (Consoli et al., 2002; Consoli et al., 2003; Calvo, 2006; Calvo et al., 2007; Babu and Chouksey, 2010 and Botero et al., 2015).

In this study, all the physical and engineering property tests on sandy soil have been performed by using dry soil. The plastic chips was added to the dry sand in different percentages 0.5, 0.75 and 1% in direct shear tests. Laboratory tests were performed on disturbed samples for determining the physical and engineering properties of the silty clay soil percentages: 0.1, 0.5 and 0.75% and the unconfined compressive strength was determined and the results were compared with the untreated soil. For performing the California bearing ratio, swelling and consolidation tests, and 0.75% of plastic waste was added into the silty clay soil and the physical and engineering properties of the reinforced soil were determined.

In the present study, a series of laboratory tests have been performed on sandy and silty clay soil. For sandy soil, sieve analysis, specific gravity, minimum index density, maximum index density, direct shear box and CBR tests were performed. All the tests were performed on the natural and the reinforced sand with plastic chips. For silty clay soil, liquid limit, plastic limit, linear shrinkage, hydrometer, compaction, unconfined compressive strength, California bearing ratio, one-dimensional swell and consolidation tests have been performed. All the tests have been performed on plastic waste reinforced silty clay soil except liquid limit, plastic limit and hydrometer tests.

1.2 Research Outlines

This study consists of five chapters. The first chapter discusses the aim of the study (research problem, research objectives and research methodology). The second chapter provides an extensive literature review. The procedures have been followed in the laboratory work was clarified in Chapter 3. The laboratory work results are analyzed and discussed comprehensively in Chapter 4. The last chapter, which is Chapter 5, summarizes the findings of the research with conclusions and recommendations for future studies.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

In the literature, there are very little studies on the reinforcing of soils by using plastic waste (Babu et al., 2011). In North Cyprus, there have been some investigations on the reinforcing of the soil by using waste materials (such as fiber and shredded tires) but nothing was done to see the effect of plastic waste on the soil behavior. In this study, plastic waste bottles will be used to see the effect of plastic waste on the physical and engineering properties of different types of soils. This chapter illustrates the basic information on soil shear strength, reinforcing techniques and the effect of plastic waste bottles on the engineering properties of soils.

2.2 Sand

2.2.1 Shear Strength of soils

For designing the civil engineering structures, in specific foundations of structures, the shear strength of the soils is playing an important role. That is because of, problems encountered in bearing capacity of shallow foundations and piles, stability of slopes of dams and embankments, and lateral earth pressure on retaining walls (Das, 2008).

Mohr-coulomb theory of failure criterion shows the rupture in material. It clarifies that failure along a plane in a material happens by a critical combination of shear stress and normal stress, and the failure takes place when shear stress and normal stress are together (Das, 2008). The relationship between normal stress and shear stress on the failure plane can be shown as

$$\tau = f(\sigma) \tag{2.1}$$

where τ and σ are the shear stress at failure and the normal stress on the same failure plane, respectively. Coulomb described shear stress as a function f (σ) and the shear stress equation is given as

$$\tau = c + \sigma \tan\phi \tag{2.2}$$

where c and ϕ are symbols representing cohesion and the angle of internal friction of the soil.

For granular soils, the shear strength of a soil can be clarified as

$$\tau = c + \sigma' \tan\phi \tag{2.3}$$

where σ' is the effective normal stress. The cohesion (c) is equal to zero and the normal stress is equal to effective normal stress. The internal friction angle, ϕ is equal to effective internal friction angle ϕ. The shear strength parameters: internal friction angle and cohesion can be determined by one of these methods, the direct shear test or the triaxial test (Das, 2008).

2.3 Some factors affecting the shear strength of granular soils

2.3.1 Shape of the particles

Cabalar et al. (2013) have been conducted an investigation on different types of sands from different locations to gather different physical properties. Specifically, the concern was about the shape (roundness and sharpness) of the particles and its effect on the shear strength. The Triaxial shear strength test was performed on four types of sands with different particle shapes. The results of the study showed that the crushed stone sand has higher strength in cyclic triaxial test than the others with rounded and angular shapes. Also, the crushed stone sand has the biggest internal friction angle in comparison with the other types.

Researchers (Gilboy, 1928; Clyton et al., 2004, Cabalar, 2011) discussed that any system of soil classification or analysis neglecting the particle shape and the effect of fines percentages would be incomplete.

The shape of the particles has also been perceived as a crucial parameter that influencing the shear behavior of cohesionless material (Shimobe and Moroto, 1995; Miura et al., 1998; Dyskin et al., 2001)

Some recent researchers (De Graff-Johnoson et al., 1969; Santamrina and Cho, 2004; Cho et al., 2006) have clarified that accruing the angularity or attenuating sphericity and roundness lead to an increase in maximum (e_{max}) and minimum (e_{min}) void ratios.

Angular quarry materials have been examined to see the effect of particle shape on shear strength. Test results indicated that the angular quarry materials conveyed higher shear strength than sub angular and sub rounded river materials (Holtz and Gibbs, 1956).

Cho et al. (2006) have examined the relationships between the packing density, stiffness, shape of the particle and strength of sands by considering smoothness, roundness and sphericity. The study showed that irregular shape reflects an increase in e_{max}.

Li et al., (2013a) indicated that the particle shape of gravel influences the critical and peak internal friction angle of clay-gravel soil mixture. The increasing gravel percentage, raised the critical internal friction angle, however it decreased the peak internal friction angle.

2.3.2 The size of the particles

Triaxial tests studies have been performed on blend of gravel and sand in different attribution and the results reflected that the shear strength has increased with gravel percentages of 50 to 60% (by weight) (Holtz and Gibbs, 1956).

Simony and Houlsby, (2006) found out that with little portion of gravel the peak shear strength of sand was more than that for pure sand at similar density

A recent research has been carried out to investigate the influence of various sands sizes, which have been collected from different locations, but they have the same morphology like angularity, sphericity, roundness and roughness. The investigation emerged that although, the ultimate internal friction angle has been influenced by the particle size for sands, there were no differences in peak internal friction angle values for the three dissimilar particle sizes of sand. (Vangla and Latha, 2015).

The size of the particles has a noticed influence on the engineering characteristics of sands. Various types of sand owning wide range of particle size and structural characteristics were studied through direct shear tests and image analysis. The peak and critical friction angles were noticeably affected by the gradation of the sand and the shape (Lim et al., 2012).

A study has been conducted in four different particle size distributions on sands. The tests have been performed on ring shear apparatus. The results have clarified that well graded samples have peak shear strength more than narrowly graded and intermediate graded specimens. In addition, it was observed that the particle size distribution has a remarkable impact on peak shear strength of soils (Igwe et al., 2012).

Igwe et al. (2005) have investigated the effect of varied sizes of sands on shear strength. The well graded sample had the biggest value of peak strength comparably with the others. It explained that the higher value of strength of well graded specimen is due to the good interlocking in the particles touching which increases the resistance to the shear stresses.

2.3.3 The impact of relative density

Igwe et al. (2012) have done a research on the impact of relative density on four various sands specimens. The specimens were prepared in loose, medium dense, dense states. The specimen of sand of gap graded reflected the lowest values of maximum and critical shear strength in all density conditions (Loose, medium and dense). They concluded that the peak shear strength increases with the increase of density as well as the critical strength.

Xiao et al (2014) have accomplished a research on the effect of relative density (initial void ratio) and confining pressure on the peak shear strength and critical state shear strength. The experimental results showed that the density was not have a big effect on the critical state friction angle, which means that with increasing the initial confining pressure the ultimate state friction angle was decreasing. Furthermore, the increase of initial void ratio and initial confining stress lead to decline in peak friction angle for rock fill material specimen.

2.3.3.4 Dilatancy

A series of large direct shear tests have been fulfilled on sand and a mixture of sand with gravel to observe the behaviour of pure sand and the mixture in terms of strength and dilation (Simoni and Houlsy, 2004). The experimental outcomes have represented that the pure sand does not have dilation. But, the medium dense pure sand had dilation behaviour. Furthermore, with the introducing of the gravel the dilation has noticeably increased and the shear strength has also increased. The addition of gravel to the sand has enlarged the constant volume friction angle. The behaviour of dilation is observed in the cases of increasing the percentage of coarse size particles or the density of the specimen (Simoni and Houlsy, 2004).

Many researchers (Bolton, 1986, Vaid and Sasitharan, 1992, Salgado et al., 2000, Yang and Li, 2004, Lashkari, 2009, Chakraborty and Salgado, 2010) have focused on the relationships between the peak friction angle and maximum angle of dilation. They have agreed in their findings that the relationship between the dilatancy and critical state is leading to understand the behaviour of soil. Also, they agreed that the density and stress both of them manipulate the dilatancy rate and related to that the strength parameters are affected.

A research has been fulfilled in rock fill materials to perceive the dilatancy under low confining pressure. This material reflected dilation behaviour. This has explained most probably because of low confining stress, which could not compress the material to resist the dilation (Charles, 1975).

Bolton (1986) suggested an index for relative density to spot the variations in the peak friction angle of sands. This index of relative density have been utilized in the theory of penetration resistance (Salgado et al., 1997a, b, 1998, Salgado and Randolph, 2001).

Bhandari and Powrie (2013) have focused on the manner of behaving of intact Reigate silver sand under various low effective stresses onset from 12.5 to 100 kPa to see the influence of fabric structure on the yield and failure of the sand. The results of the analysis illustrated that the peak strength have been attained before the dilation (volume expanding) and there was delaying in the dilation with increasing the effective stress. In the case of low effective cell pressure, the deformation distributed beyond early strain.

2.3.4 The influence of confining pressure

Sayeed et al. (2012) have attempted to study the influence of confining stresses on the mechanical behaving of granular materials. Their study was conducted by using the method of discrete element (DEM). From the outcomes of simulation and experimental studies there were reasonable similarity between them. The results have exhibited that the confining pressure have not affected the dilatancty index even with changing the pressure from very low to high pressure.

Different experimental works have been focusing on the effect of different confinement pressure on the granular materials. The experimental works have drawn that the angle of internal friction and dilation decline with enlarging the confinement pressures (Marachi et al., 1972; Tatsuaka et al., 1986; Gupta, 2009).

Sitharam (1999) has performed numerical simulation to study the engineering behaviour of granular material with the changing of confining pressure by using discrete element modeling (DEM). The study figured out that the increasing of confining stress influences the stiffness, which is increased and the change of volume, which is decreased. In addition, with increasing the increment of confining pressure the internal friction angle slightly decrease.

Karman (1911) has explained that the confining pressure has an impact on the mechanical behaviour of cohesionless granular material. The confinement stress is governing the shear strength of material.

The results of triaxial tests performed on granular soils have presented that the higher the lateral pressure lesser the internal friction angle and less tendency to dilate (Corps of engineers, 1947; Leslie, 1963; Marsal; 1967; Marachi et al., 1972; Varadarajan and Mishra, 1980 and Gupta et al., 1995).

2.4 Cohesive soils

2.4.1 Problematic soils

The problematic soils are known by the deficiency of their properties, which are unstable and uneconomical for the construction in civil engineering approaches. These types of soils are taking a wide area and they are challenging in civil engineering field. In civil engineering specialization, a researcher has stated that the problematic soils causes threats to the civil engineering structures, which can cause failures (Ola, 1983).

The problematic soils are deficient in their properties. For this reason, they are not proper materials to be used as earth material to construct on or supported with the civil engineering structures. These types of soils hold characteristics such as high compressibility, excessive swelling and low shear strength (Tonoz et al., 2003; Nalbantoglu, 2004).

Rogers et al. (1997) have intensified that problematic soils are connected in their weak characteristics with geotechnical properties such as mineralogical and chemical component, plasticity, changing of volume and hydraulic conductivity. The shortage in the properties of problematic soils can be assigned to the surcharge, temperature, pH, and the distribution of the particles, the soil history and the nature of pore fluid chemistry of soil (Ahnberg, 2006).

There are different types of problematic soils, such as soft clay, collapsible soils, highly expansive clays, and high sensitive clay. In this section the expansive soils will be discussed.

2.4.1.1 Expansive soils

Chen (1975) has described that the expansive soils are located in tropical zones. This type of soil is existed in many portions of countries such as south-western United States, Africa, Canada, South America, India, Europe, Australia, China and Middle East. Swelling soils are seriously influencing the construction projects. The discovering of swelling soil increases with the increase of constructional activities specifically in the developing countries. This type of soils have large amount of montorillonite, which is highly active clay mineral. This clay mineral is responsible for volume change of soils.

The changes in the environmental conditions in the field is due to cyclic fluctuations, which are affecting the expansive clays behaviours. These fluctuations come from altering in water table depth. These changes are due to drainage problems, and from human activities (Ufc, 2004).

A study has been conducted in Sudan to show the problems of expansive soil in the construction. The expansive soils in Sudan have caused considerable damages to sewage systems, buildings, water lines and irrigation layouts. The annual estimated damages from expansive soil exceeds 6,000,000$ (Wayne et al., 1984). Around one-third of Sudan area is covered by swelling soil. From the results of the study the researchers have concluded that preliminary investigations and precautions should be made before construction. For example, for the sewers the flexible sewer connections and suitable site grading should be applied to prevent leaking of water on expansive soil layers. Also, they recommended designing of grade beams and suspended slab to be deigned to adjust soil heaving (Wayne et al., 1984).

The world population increases the universal development, this universal development needs more landed area to be utilized. For this reason, civil engineers are imposed to use inappropriate expansive soil area. These areas require remediation to stabilize and change their physical and geotechnical characteristics (Nelson et al., 1992).

2.4.2 Shear strength of cohesive soil

The shear strength of cohesive soil is dependent on two parameters, which are cohesion(c) and the internal friction angle (ϕ). To evaluate the shear strength parameters the direct shear test or triaxial test can be performed on clays. The most appropriate test for cohesive soils is the triaxial test because of the control of the drainage conditions. The cohesion (c) for normally consolidated clays is approximately equal to zero. But, the cohesion for overconsolidated clays is greater than zero (Das, 2008).

2.5 Reinforcement materials for soils

2.5.1 Reinforcing by cement

The reinforcement by using cement is a fast process, which can be applied in a spacious range of soil. The addition of cement has reflected noticeable improvement in unconfined compressive strength, workability and shear strength of soil (Sariosseiri and Munhunthan, 2009).

A study has been conducted on the improvement of sand by using cement. The results of the study showed that with the increase of cement percentage the material is becoming more strong, brittle and stiff. From this study, the researchers have reached to a conclusion that the increasing of cement percentage turns out to increase the cohesion (Abdulla and Kiousis, 1997).

Mohamedzain and Al-Rawas (2011) have strengthened sabkha sandy soil with cement. This soil has high compressibility and low shear strength. The little settlement and good bearing capacity are needed to construct buildings and infrastructure. The cement has been added to the sandy soil with different percentages from 2.5 to 10% and the curing has been applied for 7, 14, 28 days. The using of cement has been found to be effective in increasing shear strength of sandy soil of Sabkha.

Zaho et al. (2013) have attempted to study the influence of cement on the highly expansive soil to observe the swelling potential and the strength of the soil. From the results of the tests, the cement has enhanced the highly expansive clay. The value of liquid limit decreased dramatically and the plastic limit increased significantly, that means the cement decreased the swelling characteristics in 7 days curing. For the unconfined compressive strength of soil, the strength has increased noticeably during

7 day curing. This increase has been noticed after 7 days curing because of the pozzollanic reaction and cement hardening.

2.5.2 Reinforcing by fiber

Fiber is a common material, which can be found in two states. The first state is natural fiber, such as cotton fiber, coconut fiber etc. The second state is industrial fiber, for example polypropylene fiber, plastic glass etc.

Sadek et al (2010) have studied the effect of three types fiber on the shear strength of sand. The results showed that the fiber was effective in increasing the ductility and shear strength of sand.

Some researchers have concluded that regardless the soil type, the shear strength is increased linearly with increasing the fiber percentage up to specific fiber fraction. After that, no additional increase in the shear strength was obtained (Hoare, 1979; Gray and AlRefeai, 1986; Jones et al., 2001).

A restricted data is available on the fine and coarse sands which have been enhanced by using fiber. Using 0.5% of fiber has showed development in the shear strength and the improvement in fine sand was quite higher comparably with coarse sand (Ranjan et al., 1996; Michalowski and Zaho, 1996; Michalowski and Cermak, 2003).

A laboratory research has been accomplished to study the effect of Coir waste, which is considered as fiber material, in marine clay to follow the change of volume of this marine clay. The outcomes from tests showed that using Coir waste as stabilizer for expansive soil is effective and efficient. Swell and compression indexes have been reduced for the treated soil with Coir waste. Furthermore, with the introducing of coir waste, the volumetric shrinkage minimized (Jayassree et al., 2014).

Polypropylene fiber has been utilized in two types of clays, which have low plasticity. The two clay samples have been mixed with different percentages of fiber 0% to 1,25%. The unconfined compression tests have showed that for both type of clay samples the unconfined compressive strength has reached its maximum when the percentage of polypropylene fiber is 0.75% and after this percentage, the strength has reduced (Senol et al., 2014).

2.5.3 Enhancing by shredded tire

Each year, the number of disposed tires are around 240 million in the United States and the number of stock is approximately 5 billion tires (Markets, 1991; Tarricone, 1993).

For the shredded tire material, a study has been conducted to see the effect of waste tire material on the shear strength of the soil. Direct shear test was accomplished and the results showed that the friction angle was varying from 19° to 25° and cohesion was ranging from 4.3 to 11.5 kPa due to tire reinforcement of the soil (Humphrey et al., 1993).

The shredded waste tire material has been mixed with sand to see the effectiveness of this material on shear strength. From the results of the investigation, the shredded tire material found to be effective in increasing the shear strength of sand. The internal friction angle has increased from 34° to 67°. From results, it was concluded that this material can be used in road fills and in the infiltration collection systems on steep slopes as construction material (Foose et al., 1996).

Edil and Bosscher (1993) have studied the impact of adding shredded tires to sandy soil to see the changes in the shear strength of sand. The laboratory work results have reflected that the addition of 10% tires increased the strength of the sandy soil in dense state.

Seda et al. (2007) have conducted a research on the highly expansive soil treated with waste of tire rubber to observe the impact of it on swelling potential. The tire rubber has been introduced to the swelling soil in small percentages. From the laboratory results, the researchers have concluded that introducing 20% tire rubber increased the compressibility, but it minimized the swelling pressure and swelling percentage.

Tiwari et al (2014) used the shredded rubber tire to modify clayey soil. This shredded rubber tires have various sizes and it has been mixed with the clay soil in different percentages to note the behaviour of soil with this material in concern of shear strength, permeability and compaction. The results have illustrated that when 6% tires was added into the soil, the shear strength has developed considerably as well as the coefficient of consolidation.

2.5.4 Reinforcing by plastic waste

The international bottled water association (IBWA) stated that the selling of bottled water has enlarged by around 500% throughout the previous decade and 1.5 million tons of plastic are utilized to bottle water in each year.

A general survey has been conducted to see the number of thrown waste bottles in a second. The survey showed that about 1500 bottles are thrown in the trash in each second. Polyethyethylene terephthalate (PET) was found to be the most common plastic waste in the civilized area (de Mello et al., 2009). Chen et al. (2010) stated that finding a way to reuse the plastic waste is a good notion to reduce greenhouse gas (GHG) emission and fossil fuel consumption.

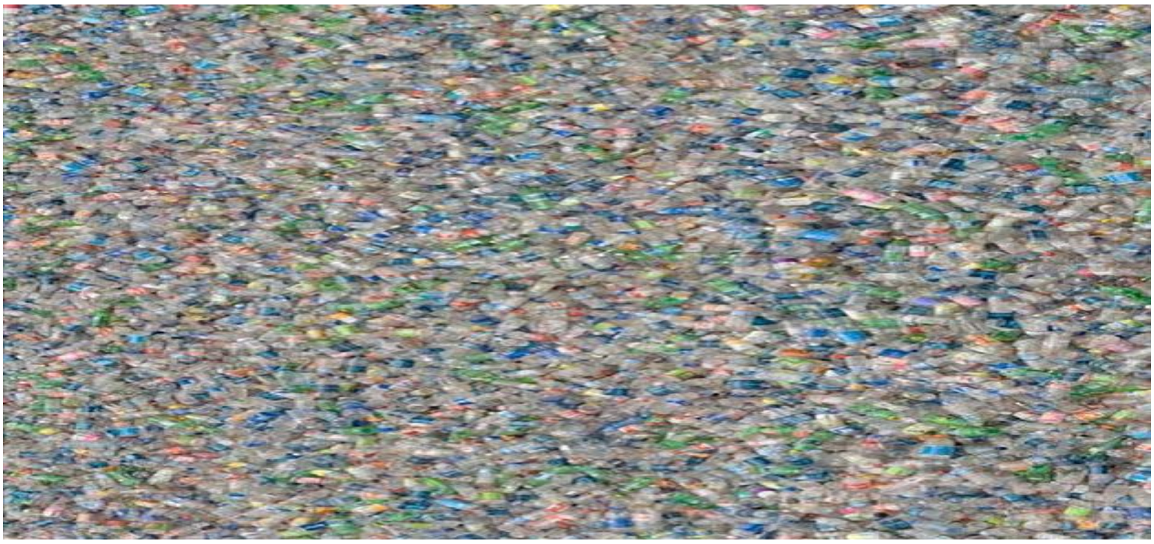


Figure 2.1: Waste water bottles in the garbage Babu and Chouksey (2011)

Figure 2.1 illustrates the waste water bottles in the garbage area. Babu and Chouksey (2011) have focused on using waste of plastic bottles as reinforcing material in sand,. The plastic waste has been added in the form of chips in different percentages. The experimental outcomes have illustrated that the inclusion of plastic waste chips have improved the shear strength of sandy soil. The improvement of shear strength was related to the increase of friction between soil and plastic waste.

An experimental study has been achieved on the reinforcing of cemented and uncemented sand with polyethylene fibers extracted from plastic wastes. The conclusion from the results illustrated that the plastic waste improved the stress-strain response of cemented and uncemented sands (Consoli et al., 2002).

A series of tests have been done to observe the effect of polyethylene terephthalate fiber, which is a recycled waste plastic bottle, on the strength of uniform fine sand

(Consoli et al., 2003). Also, the fiber has been used with cemented sand. The laboratory results reflected that the material enhanced the peak and critical strength of the treated soil with cement and the untreated soil. Also, the material has decreased to some extent the brittleness of the cemented sand (Consoli et al., 2002).

Two samples of expansive soils have been collected from different locations and have been studied to see the behaviour of the soil with the addition of plastic waste, which is synthetic fiber. The laboratory results reflected significant enlargement in the mechanical characteristics of soils. The fiber had a good effects on the expansive soil. Furthermore, when the length of fiber increased the resistance of soil also increased (Calvo, 2006; Calvo et al., 2007).

Plastic waste material is used with silty soil to evaluate its effectiveness on the shear strength parameters of the soil. The material has been mixed with percentages from 0-1%. The tests results presented that the internal friction angle decreased with the increase in plastic fraction. But the cohesion increased with the increase in the addition of plastic (Botero et al., 2015).

Chapter 3

MATERIALS AND METHODS

3.1 Introduction

In this thesis, the last two chapters discussed the introductory information about this research. In This chapter the materials and methods used to study the effect of plastic chips on the geotechnical properties of soils are presented.

A detailed laboratory work is programed to investigate the impact of waste plastic bottles chips on the engineering properties of sand and silty clay soils. The laboratory work involved experimental tests for identifying the natural and stabilized properties of soils and compare them with each other. The laboratory tests have been carried out in accordance to American standards, ASTM except linear shrinkage and hydrometer tests were performed according to British standard 1377.

3.2 Materials

3.2.1 Experimental soils

3.2.1.1 Sandy soil: Bedis Beach Sand

Various laboratory tests have been performed on sandy soil, which has been taken from Bedis Beach, Famagusta, North Cyprus. The sandy soil has been excavated from the surface of the beach.

The approximate location (Latitude 35.13 and Longitude 33.93) of the specified area from which the sand was taken is shown in Figure 3.1 and the properties of the natural Bedis sand are given in Table 1.

Figure 3.1: Geographical location of sandy soil (Google map)

The sand taken from the site is placed into plastic containers and transported to Soil Mechanics Laboratory, Eastern Mediterranean University, EMU. The soil was placed in big trays and put in an oven at 110˚C for at least one day before performing any test. All the tests on sand have been performed in dry state.

Table 3.1: Physical properties of natural Bedis beach sand

Physical properties	Values
Minimum void ratio[a], e_{min}	0.55
Maximum void ratio[a], e_{max}	0.73
Minimum index density[a], $\rho_{d(min)}$ (g/cm^3)	1.57
Maximum index density[a], $\rho_{d(max)}$ (g/cm^3)	1.76
Specific gravity[b], (Gs)	2.72
Coefficient of uniformity[c], Cu	1.78
Coefficient of curvature[c], *Cc*	0.88
Soil classification[d]	SP

a According to Impact method, Joseph E Bowles 1986
b According to ASTM D 854-06
c According to ASTM D 2487-06
d According to ASTM D 2487-00 (Unified Soil Classification System)

According to maximum and minimum index densities the required void ratios and index densities for 30% and 60% relative density state are calculated. The void ratio for 30% and 60% relative densities were 0.68 and 0.62, respectively. The dry density for the sand at 30% and 60% relative density states are calculated to be 1.62 g/cm^3 and 1.68 g/cm^3, respectively.

In this study, an optical microscope with a magnification of 10x (scale 200μm) has been used to study the particle shape. Some pictures have been captured for individual particles and five particles's pictures were presented in Figure 3.2. These pictures were randomly chosen and almost all of them had sub rounded shapes.

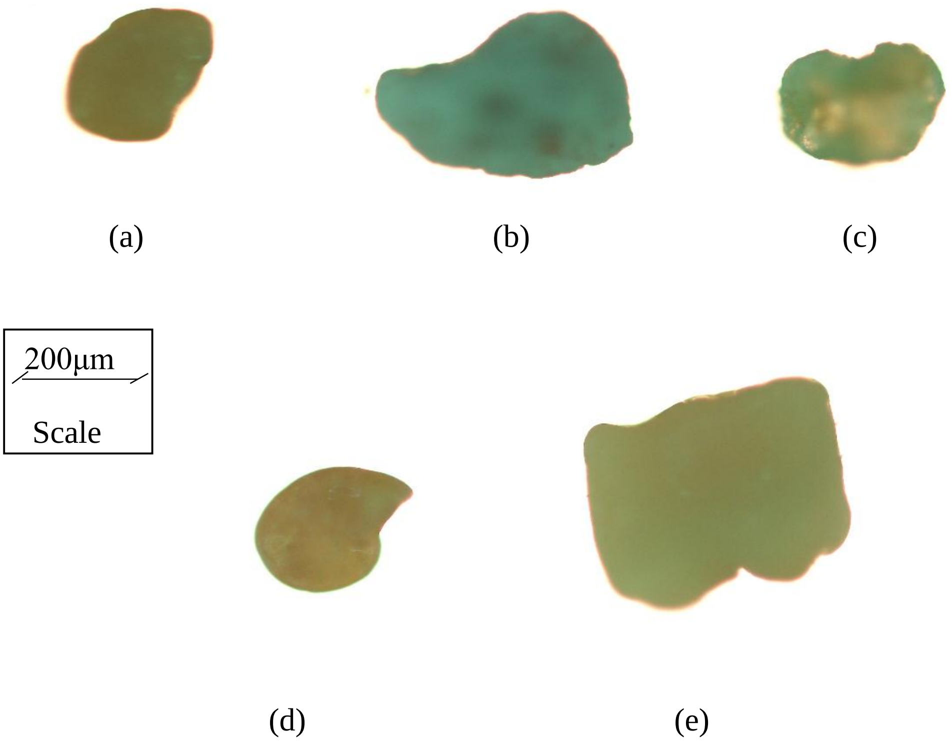


Figure 3.2: Particle shapes of randomly selected sand particles

In this investigation, sieve analysis was performed to classify the natural sand. Direct shear test has been carried out on natural and reinforced sand with plastic chips to determine the shear strength parameters before and after improvement.

3.2.1.2 Tuzla soil

The second soil used in this study was taken from Tuzla, Famagusta, North Cyprus. This ground is excavated to a depth of 4.0-4.5m below the ground surface and samples taken from that depth.

The approximate location (Latitude 35.16 and Longitude 33.88) of the chosen area from which the soil was taken is shown in Figure 3.3. The physical and index properties of natural Tuzla soil are shown in Table 3.2.

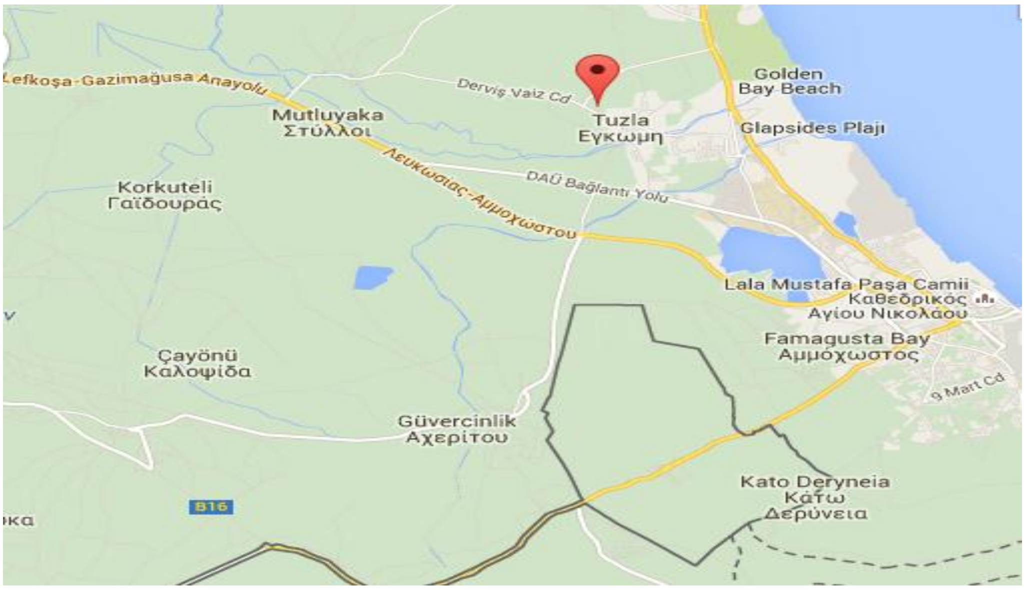


Figure 3.3: Geographical location of Tuzla clay (Google map)

Tuzla soil taken from the site is placed in plastic containers and transported to Soil Mechanics Laboratory, EMU. Then, the soil was placed in trays and put in an oven at 50° C for minimum 5 days to dry. After drying, the soil was pulverized. All of the tests have been performed on pulverized soil. The dry soil has been mixed with some amount of water and then kept in tight plastic nylon bags for curing for at least 24 hours to assure uniform distribution of water in the soil sample. This 24 hour curing time facilitates the curing of the soil, which enable the measurements of index properties of the soil.

Table 3.2: Physical and index properties of Tuzla soil

Index properties	Values
Fraction of clay size (<2μm)[a] (%)	38.0
Fraction of silt size (2μm-74μm)[a] (%)	54.0
Fraction of sand size (>74μm)[a] (%)	8.00
Maximum dry density[b], $\rho_{d(max)}$ (g/cm^3)	1.69
Optimum moisture content, w_{op} (%)	22.0
Specific gravity[c], (Gs)	2.65
Liquid Limit[d], LL (%)	48.0
Plastic Limit[d], PL (%)	28.0
Plasticity Index[d], PI	20.0
Linear shrinkage, Ls (%)	18.0
Activity[d]	0.53
Soil classification[e]	CL-ML

a According to ASTM D 422-98
b According to ASTM D 698-07
c According to ASTM D 854-06
d According to ASTM D 4318
e According to ASTM D 2487-00 (Unified Soil Classification System)

In this study, the optimum moisture content and maximum dry density have been used for preparing samples for swell, consolidation and shear strength tests.

3.2.2 Plastic bottles

The empty plastic water bottles have been collected from Eastern Mediterranean University cafeteria. This plastic waste is polyethylene terephthalate (PET). The plastic bottles have been cut into small sizes by using a cutting machine used for cutting papers at the printing office of the university. By using this machine, the plastic bottles has been cut into strips and then by utilizing a scissor, these strips were cut into small pieces. The sizes of the plastic chips were approximately 12mmx4mm. A photo of the plastic chips used in this study is in Figure 3.4.

Figure 3.4: Plastic bottles chips used as a reinforcing material in this study

3.3 Methods

The methods of testing used in this study are presented in the following sections. Each test was repeated at least three times in order to increase the reliability of the results.

3.3.1 Linear shrinkage

Linear shrinkage test was done in accordance to British standard BS 1377. The moisture content of preparing the sample was corresponding to 15 blows in the liquid limit test. The linear shrinkage is a percentage, which is one unit subtracted from the ratio of average length after shrinkage divided by the standard length of the mold.

For the treated sample with plastic chips (0.75%) the same procedures was followed as for the natural soil. The only difference is the weight of plastic is subtracted from the dry soil weight.

The equation 3.1 was used to determine the linear shrinkage of native soil.

$$Ls = (1-(L_{avg}/L_0)) \qquad (3.1)$$

where:

L_0 = Initial length of brass mold (mm).

L_{avg}= Average length after shrinkage (mm).

LS= Linear shrinkage in percentage.

3.3.2 Minimum index density

This test was conducted following the impact method (Bowles, 1992). The compaction mold has been considered to perform the test. The dry sand sample has been pour by using a small funnel inside the mold in the loosest state to get the maximum void ratio. The funnel has been rounded inside the mold until the soil reached the top of the mold. From the volume of the mold and the weight of soil inside the mold the maximum void ratio can be calculated. According to this value, the minimum index density can be obtained.

3.3.3 Maximum index density

The maximum index density has been achieved by following the impact method. The normal compaction mold and circular weight (12 kg) have been used to get the maximum index density. The dry sand soil sample was poured into the mold by using plastic funnel. The mold has been filled in five successive layers. Each layer was compacted by the circular weight by hitting the mold by rubber hummer 25 times. The last layer is reached with the top of the mold. From the mold and the soil weight, volume of the mold and volume of compacted soil inside the mold the maximum index and minimum void ratio have been computed. Figure 3.5 shows the circular weight and compaction mold used in this study.

(a) (b)

Figure 3.5: The tools for impact method (a) Circular weight (b) Compaction standardized mold

3.3.4 Direct shear box test

In this study, two relative density (30% and 60%) have been selected and the sand samples were prepared at these relative density values. From the equation of relative density (Equation 3.1), the void ratio with specified relative density was computed. Then, from the calculated void ratio, the mass of soil was calculated. For 30% relative density state, the soil mass was poured in one layer with slight vibration and tamped to reach the required density state in the box. For 60% relative density state, the soil was poured in three successive layers, each layer was slightly shaken and tamped to reach the desired relative density state. The sand specimens prepared in this way in direct shear box apparatus was subjected to three different normal stresses values: 20, 30 and 50 kPa. The three normal stress values and the corresponding shear stresses at failure were used to determine the internal friction angle. The rate of displacement applied in the study is 1.06 mm/min according to Prahash, 1995.

$$\mathbf{D_r = ((e_{max} - e)/ (e_{max} - e_{min}))} \tag{3.2}$$

The percentages of plastic chips added into the soil were 0.5, 0.75, and 1% of the dry weight of the soil. Figure 3.6 shows the reinforced sand with plastic chips.

Figure 3.6: Plastic chips reinforced sand

3.3.5 Standard Proctor compaction test

This test was used to determine the compaction characteristics of the soil. Six samples in different moisture contents were prepared and compacted in the mold and the compaction characteristics (the maximum dry density and optimum moisture content) of the soil were determined.

3.3.6 Unconfined compression test

For natural and reinforced soil, the unconfined compression test was done by following the criteria of ASTM D 2166-06. The natural pulverized dry soil was blended with water and tight in plastic nylon for one day and then the soil was compacted in standard compaction mold at optimum moisture content and from the compacted soil, three samples were extracted for unconfined compression test. For the reinforced soil, the samples were compacted at optimum moisture content obtained for the natural soil and the corresponding dry density values obtained at this water content for the reinforced soils were given in Table 3.3.

Table 3.3: Dry density values obtained for the reinforced soils compacted at the optimum moisture content (22%) of the natural soil

Plastic percentage (%)	Dry density, ρ_d (g/cm^3)
0.1	1.69
0.5	16.6
0.75	16.4

Throughout the study, the reinforced specimens in all tests (unconfined compression, swell, consolidation and CBR) were prepared at the dry density values given in Table 3.3.The natural Tuzla soil and the reinforced samples after failure are presented in Figure 3.7.

(a) Natural Tuzla soil

(b) Reinforced soil

Figure 3.7: Unconfined compression test specimens after failure

3.3.7 One dimensional swelling test

A standardized oedometers were utilized to perform swelling test on the natural and reinforced soil. For natural soil,the soil specimens were compacted and prepared at optimum moisture content. For reinforced soil (0.75% plastic chips), the specimen was mixed with optimum moisture content and kept in tight nylon bag for 24 hours and then the plastic waste percentage mixed with soil and the mixture is kept for 24 hours and then the compaction was performed. The dry density and moisture content of natural reinforced soil are presented in Table 3.3.

The compacted samples have been put on the oedometers and a surcharge pressure of 7 kPa was applied on the specimen and the samples were inundated with distilled water. The samples were left for 24 days for swelling until equilibrium was reached.

3.3.8 One dimensional consolidation test

After the swelling test, the consolidation test was conducted. The loads were applied starting from 1kg to 64kg. Each loading has been left for one day. Then, the unloading has taken place. After that, the samples were removed and the final moisture content was calculated.

3.3.9 California Bearing Capacity Ratio, CBR

This test was used to obtain the maximum penetration of the natural and the reinforced soils. A compacted soil was utilized and surcharge load was stratified by using loading machine. The CBR test was performed on Tuzla soil compacted at optimum moisture content and maximum dry density for natural soil. The reinforced soil (0.5% plastic chips), were prepared at moisture content and dry density presented in Table 3.3.

The CBR test was also performed on dry sand at two relative density state 30% (loose state) and 60% (dense state).

For 30% relative density state, the specified mass of sample is poured in one layer. But, for 60% relative density state the specified mass of sample is poured in three successive layers, each layer was compacted by weight of 4.5g Kg to reach the required density state.

Chapter 4

RESULTS AND DISCUSSION

4.1 Introduction

All the laboratory work conducted in this study was according to the methods explained in detail in Chapter 3. In Chapter 4, a detailed analysis and discussion of all laboratory results are presented. The results of the laboratory work are divided into two parts. The first part studies the physical and engineering properties of the natural soils: Bedis sand and Tuzla soil, and the second part discusses the effect of plastic chips on the engineering behaviour of Bedis sand and Tuzla soil.

4.2 Natural soils properties

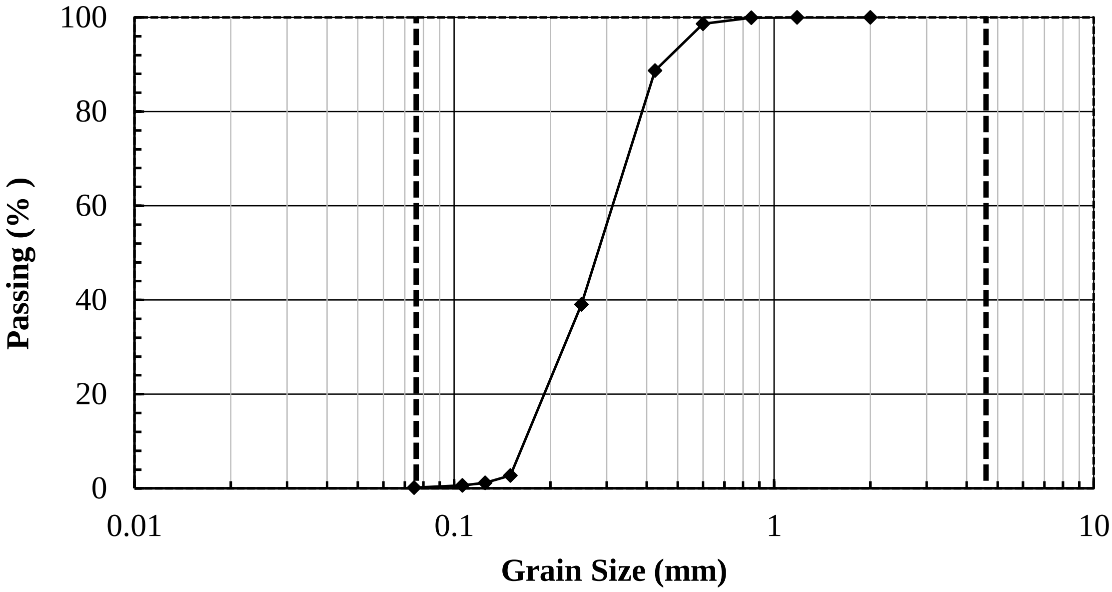
As mentioned above, there are two types of soils used in this study-: Bedis beach sand and Tuzla soil with high volume change characteristics.

4.2.1 Sandy soil: Bedis sand

The physical properties of natural Bedis sand were shown in Table 3.1.

4.2.1.1 Sieve analysis

The particle size distribution curve for natural sandy soil is shown in Figure 4.1.



Clay &Silt	Sand	Gravel

Figure 4.1: Particle size distribution for natural Bedis beach sand

4.2.1.2 Direct shear test

Figures 4.2 and 4.3 present the shear stress versus horizontal displacement. Using the maximum and critical shear stress values, the peak and critical internal friction angles were determined.

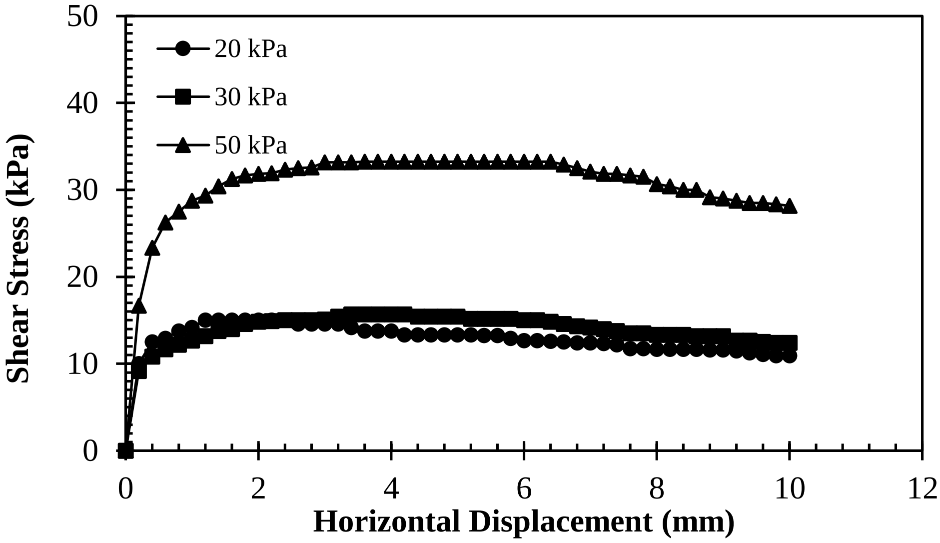


Figure 4.2: Shear stress versus horizontal displacement for natural Bedis beach sand at 30% relative density

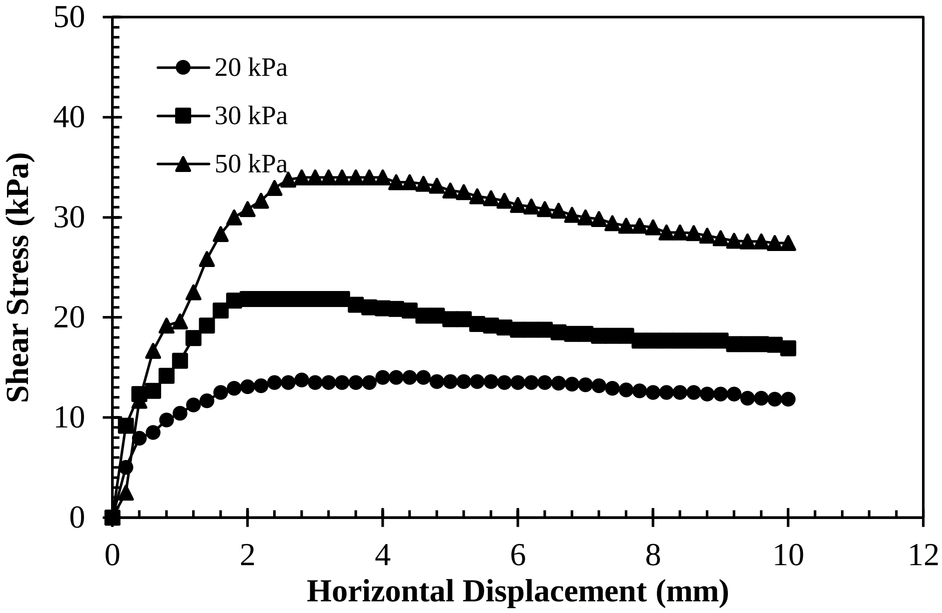


Figure 4.3: Shear stress versus horizontal displacement for natural Bedis beach sand at 60% relative density

The peak, ϕ_{peak} and critical, ϕ_{cri} internal friction angles are presented in Figures 4.4 and 4.5.

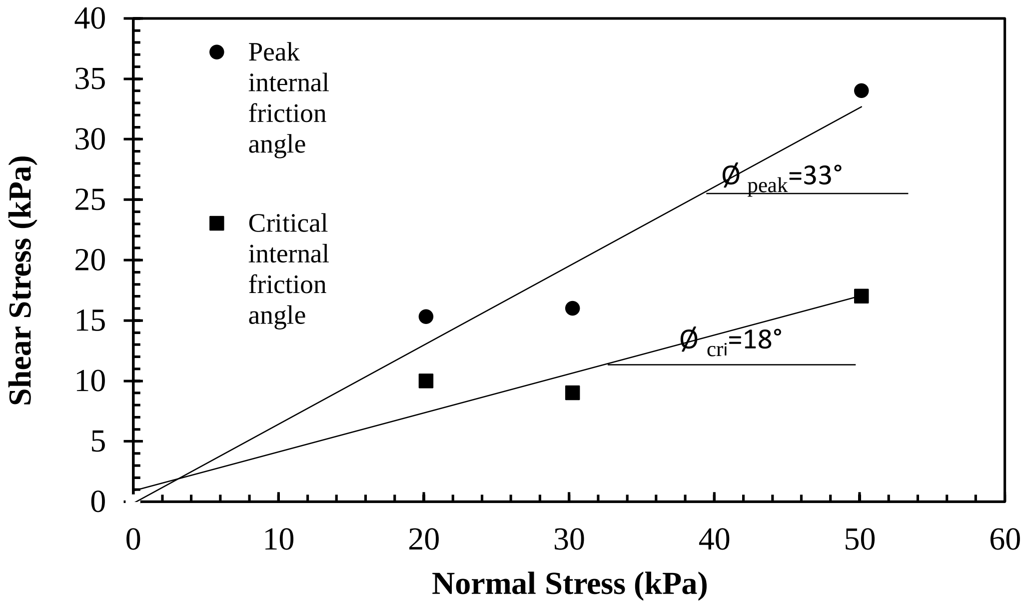


Figure 4.4: Shear stress versus normal stress for natural Bedis beach sand at 30% relative density

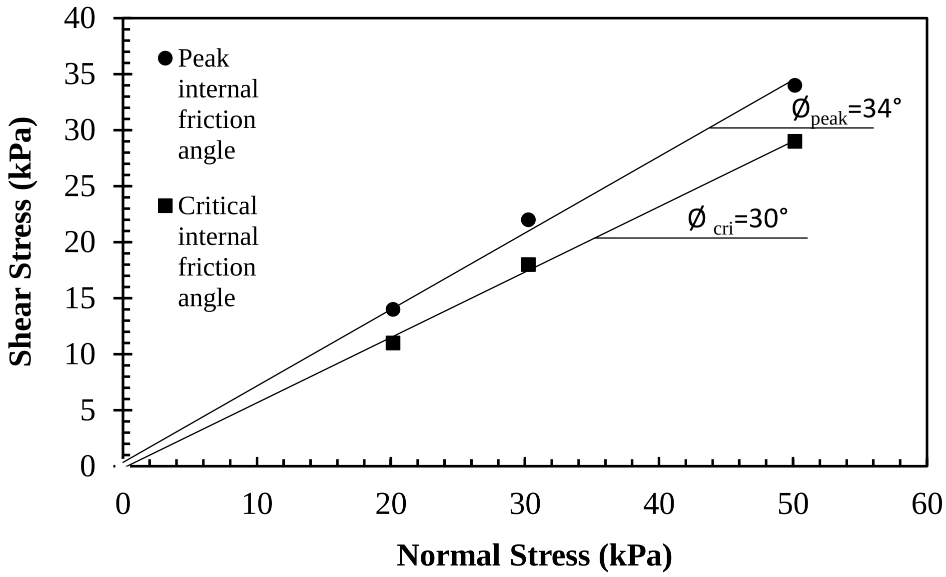


Figure 4.5: Shear stress versus normal stress for natural Bedis beach sand at 60% relative density

From Figures 4.4 and 4.5, the results indicate that with increasing relative density, the peak and critical internal friction angles are increasing due to the increase in the interlocking of the sand particles.

4.2.1.3 California Bearing Ratio, CBR test

The CBR test was performed according to ASTM D1883, which was explained in detail in materials and methods chapter. This test was performed to determine the maximum penetration of the soil under specified moisture content and density. The sand samples were prepared in two different relative densities: 30 and 60% ($\rho_{d30\%}$ =1.62g/cm^3 and $\rho_{d60\%}$=1.68). The values of CBR for natural sand were 9 and 11 at 30% and 60% relative densities, respectively. Figure 4.6 presents the CBR curves for both relative densities.

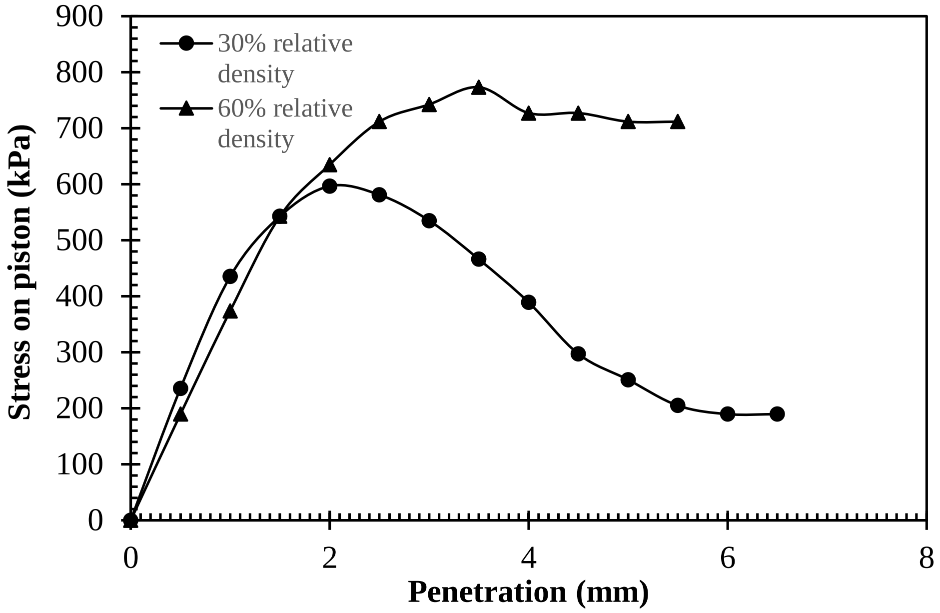


Figure 4.6: California bearing ratio curve for 30%&60% relative density for Bedis beach sand

The unusual drop in the curve happened for sand because of that the sand has only friction and there is no cohesion. For this reason, after reaching the maximum penetration the sand particles started to move over each other this causes the drop in the curve.

4.2.2 Tuzla soil

The natural properties of Tuzla soil were given in Table 3.2.

4.2.2.1 Hydrometer Test

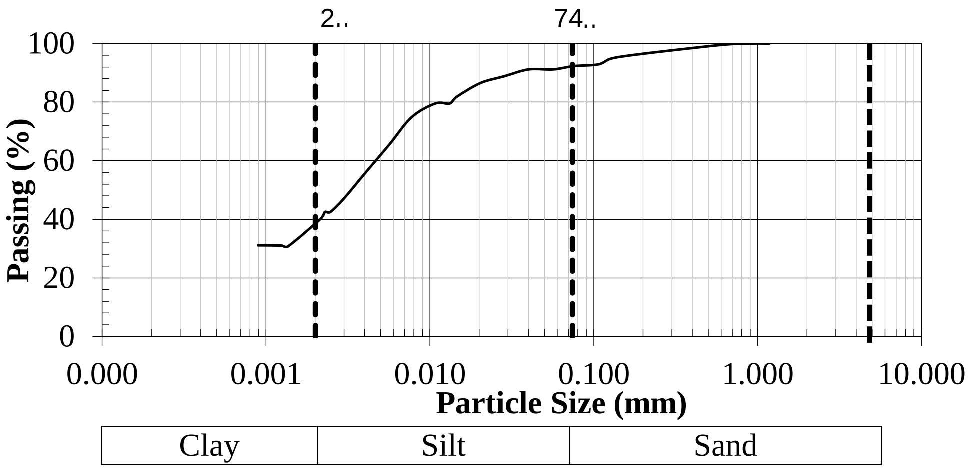


Figure 4.7: Particle size distribution of natural Tuzla soil

4.2.2.2 Standard Proctor compaction test

The test was done to gain the compaction characteristics, which are the optimum moisture content (OMC) and maximum dry density (MDD). The compaction curve of natural Tuzla soil is shown in Figure 4.8.

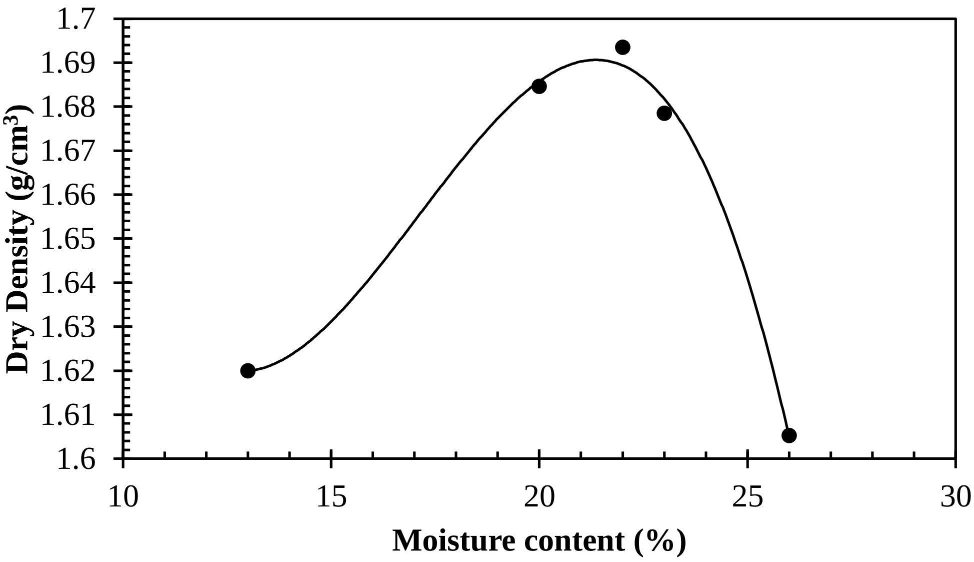


Figure 4.8: The compaction curve of natural Tuzla soil

The optimum moisture content and the maximum dry density of the soil are 22% and 1.69 g/cm^3 respectively as presented in Table 3.2.

4.2.2.3 Atterberg limits test

The liquid limit and plastic limit represents the Atterberg limits. The liquid limit and plastic limit test of natural soil were conducted according to ASTM D 4318. The values of liquid limit and plastic limit were 48% and 28% respectively. The plasticity index of the soil was 20%.

The values of liquid limit and plasticity index were taken to classify the soil according to Unified Soil Classification System. The classification of soil was CL-ML, which is low plasticity silt and clay.

4.2.2.4 Linear shrinkage test

The linear shrinkage test was performed in conformity with the British standard, BS 1377. The value of the linear shrinkage for the natural Tuzla soil was calculated to be 18 %.

4.2.2.5 Unconfined compression test

The test of unconfined compression test was done to measure the unconfined compressive strength and the undrained cohesion of the natural Tuzla soil. Figure 4.9 represents the curve for unconfined compression test. The unconfined compressive strength (q_u) of the compacted soil at optimum moisture content was determined to be 404 kPa.

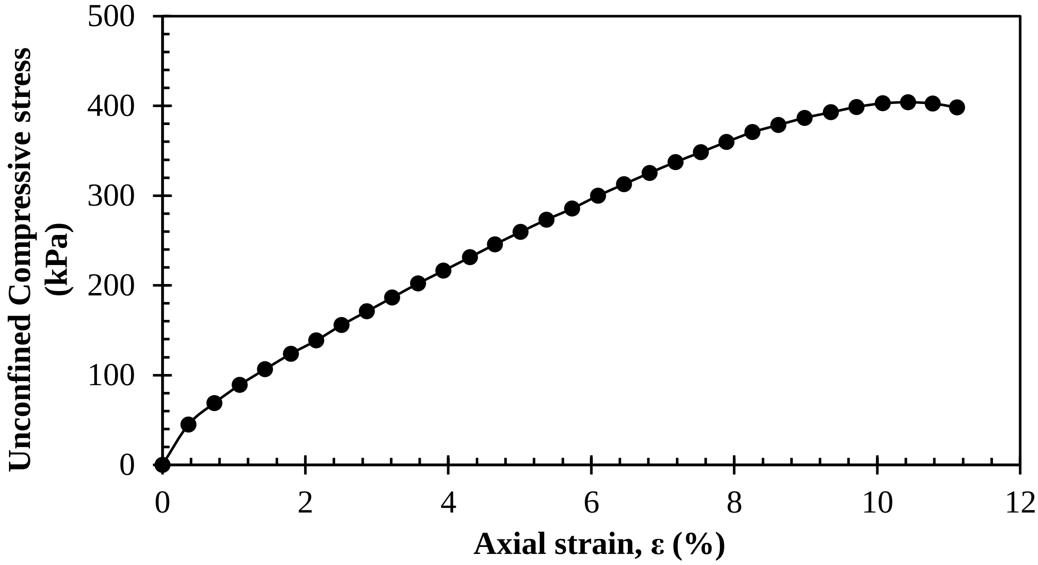


Figure 4.9: Unconfined compressive strength, q_u of natural Tuzla soil

The undrained cohesion of the natural soil is the value of q_u divided by 2. The value was calculated to be 202 kPa.

4.2.2.6 California Bearing Ratio, CBR test

The CBR value is determined as the ratio of the unit load (kN/m^2) needed to provide a depth of penetration (2.54mm) of the penetration piston to the standard unit load (6984.76 kPa) required to obtain the same depth of penetration on a standard sample of crushed stone.

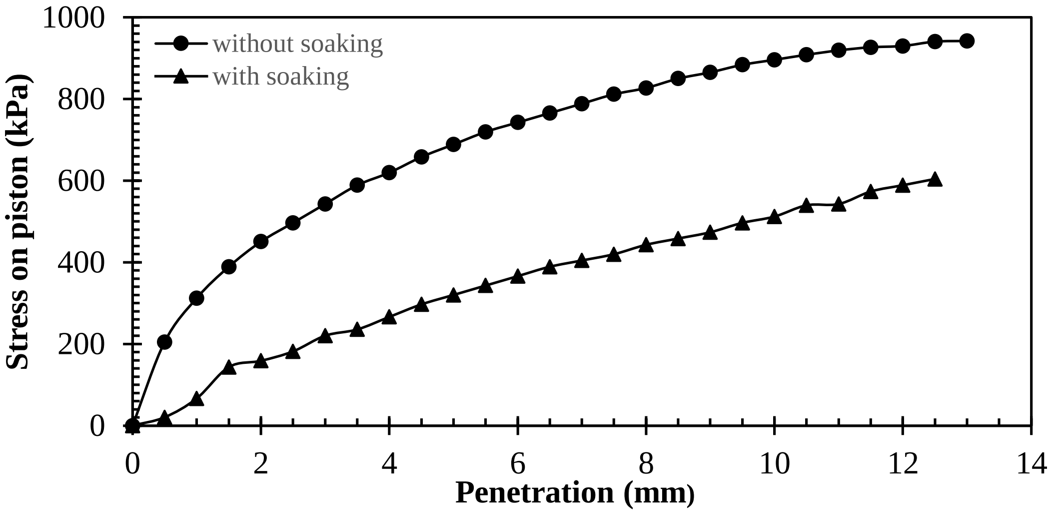


Figure 4.10: California bearing ratio curves for soaked and unsoaked samples

The unsoaked CBR value for natural soil is 7.2. After soaking, the CBR value of the same soil decreased to 2.7. According to Bowles (1970) the soil in the soaked condition is a very poor subgrade material for road construction. Reduction in the CBR values in the soaked and unsoaked specimens indicates the big lost in the strength of soil after soaking.

4.2.2.7 One dimensional swelling test

The oedometer device was used to perform the test. The sample was left to swell under 7 kPa surcharge stress almost three weeks until no change in the swell was recorded. The swelling percentage has been calculated by dividing the change in the height of the specimen to the initial height of the soil. The maximum percentage of primary and rate of swelling of native Tuzla soil are shown in Figure 4.11 and they were found to be 4.5 and 5.0% respectively.

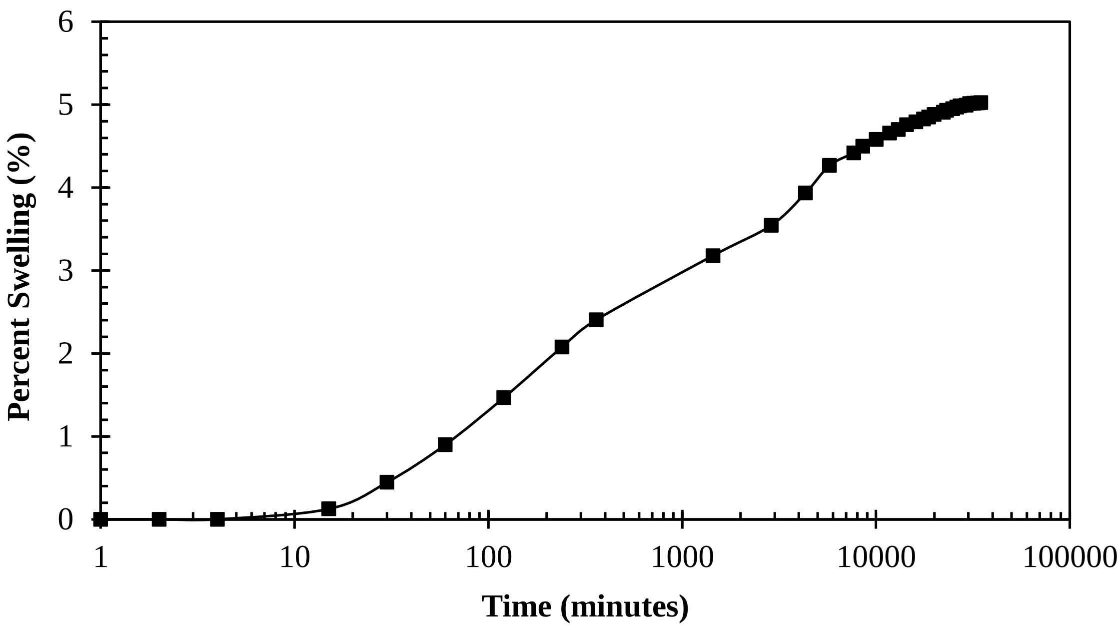


Figure 4.11: Swelling versus time curve for natural Tuzla soil

4.2.2.8 One dimensional consolidation test

The test of one dimensional consolidation was performed in conformity with ASTM D2435-04. The results obtained from one dimensional consolidation test were used to draw the void ratio-logarithm of effective stress curve as in Figure 4.12. The compression index (C_c), rebound index (C_r) and consolidation coefficient (C_v) are calculated from the consolidation curve shown in Figure 4.12 and the values are shown in Table 4.1.

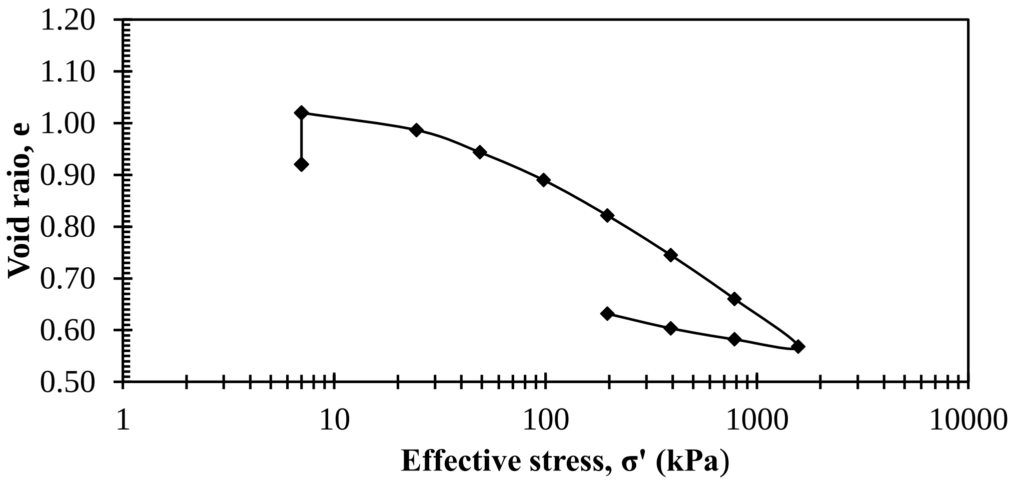


Figure 4.12: Void ratio versus log pressure curve of natural Tuzla soil

Table 4.1: Consolidation parameters of natural soil

Compressibilty characteristics	Values
Compression index, C_c	0.31
Swelling index, C_s	0.09
Coefficient of consolidation, C_v (m^2/s)	1.46E-08
Time required for 90% consolidation, t_{90} (min)	17
Hydraulic conductivity, K (m/s)	3.27E10-11
Preconsolidation pressure, σ_p' (kPa)	100
Swelling pressure, (kPa)	80

4.3 The influence of plastic chips on the engineering behaviour of soils

4.3.1 Sandy soil: Bedis Beach Sand

4.3.1.1 The impact of plastic chips on the shear strength

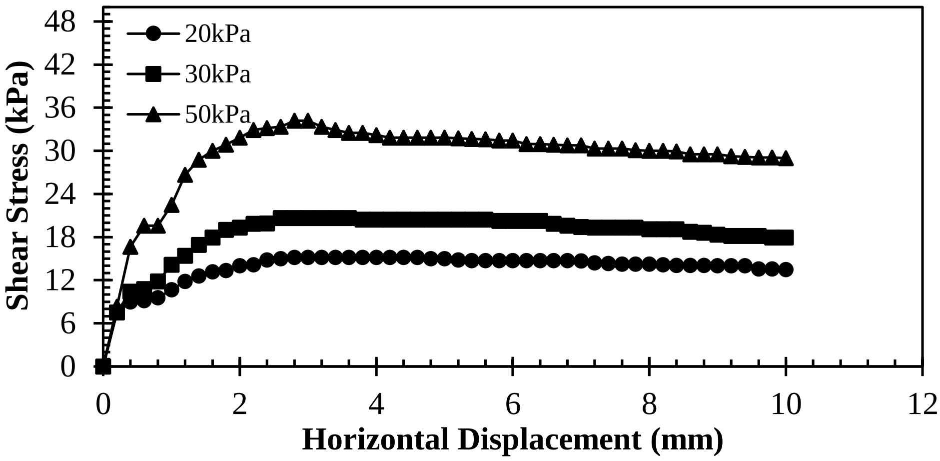


Figure 4.13: Shear stress versus horizontal displacement for reinforced Bedis sand with 0.5% plastic at 30% relative density

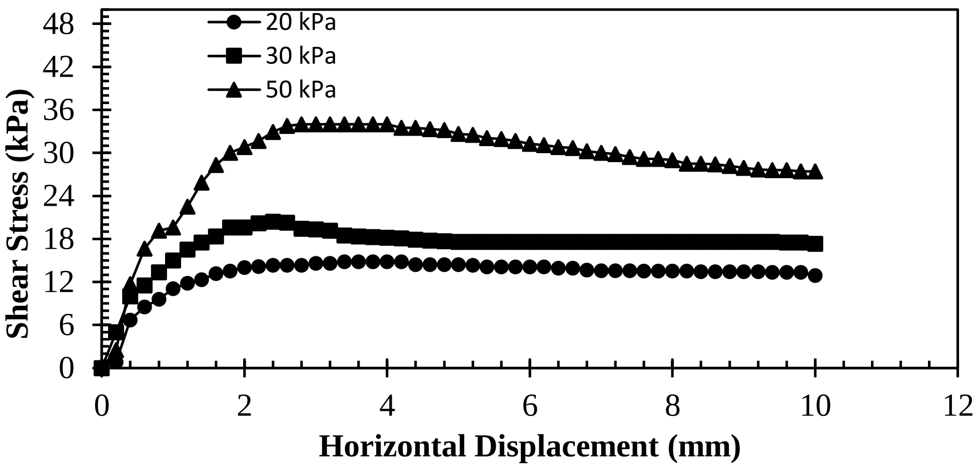


Figure 4.14: Shear stress versus horizontal displacement for reinforced Bedis sand with 0.5% plastic at 60% relative density

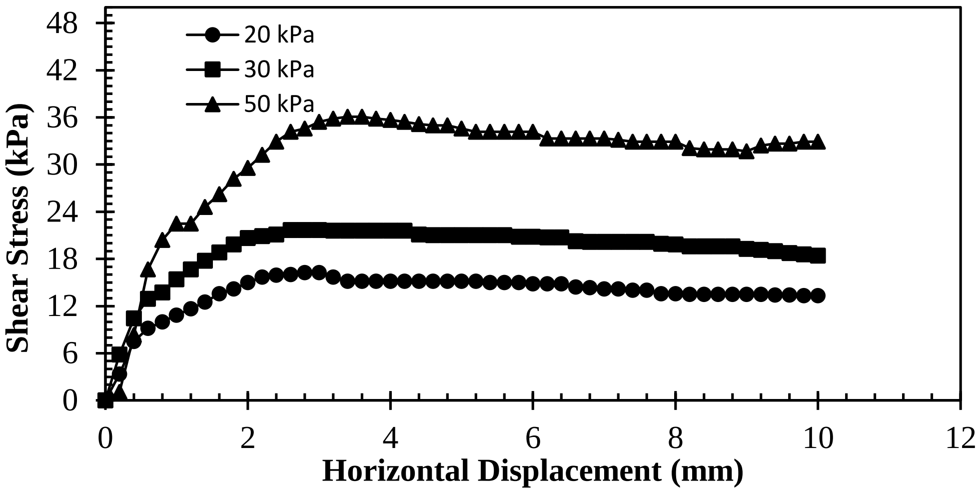


Figure 4.15: Shear stress versus horizontal displacement for reinforced Bedis sand with 0.75% plastic waste at 30% relative density

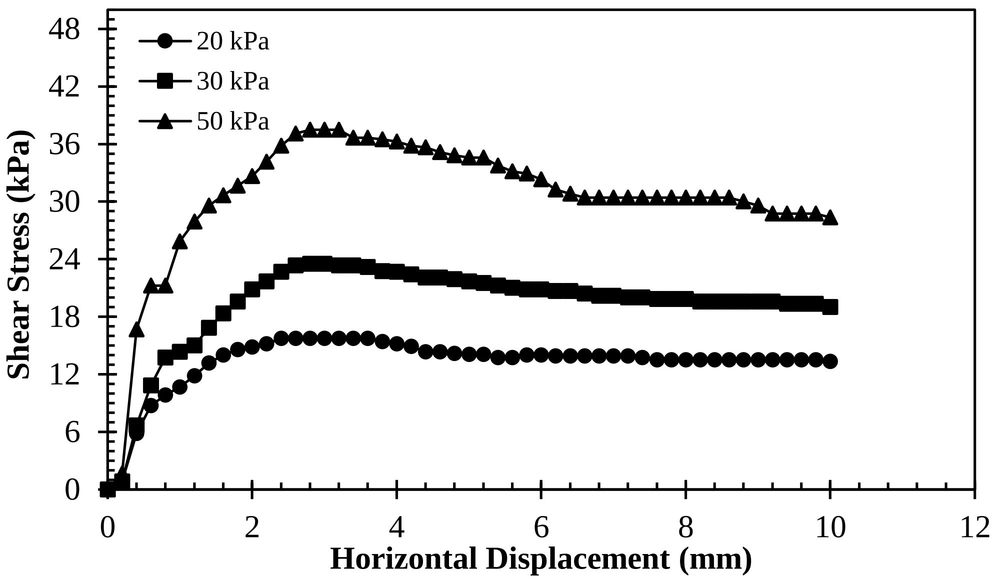


Figure 4.16: Shear stress versus horizontal displacement for reinforced Bedis sand with 0.75% plastic waste at 60% relative density

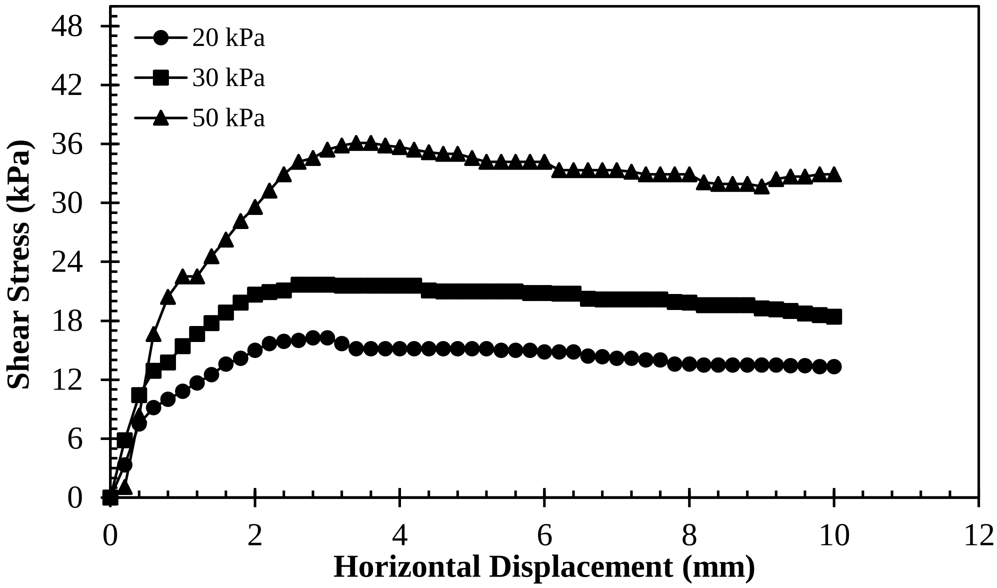


Figure 4.17: Shear stress versus horizontal displacement for reinforced Bedis sand with 1.0% plastic waste at 30% relative density

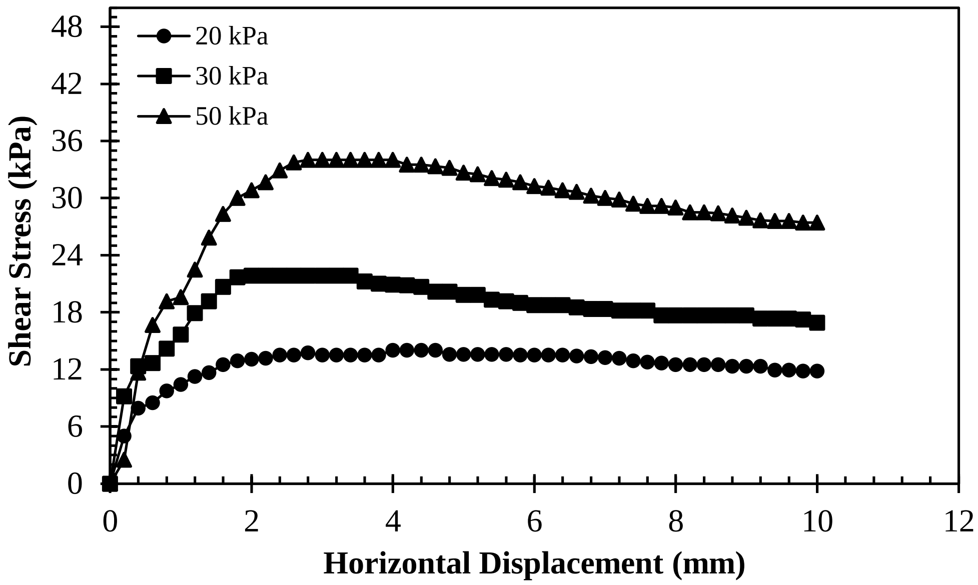


Figure 4.18: Shear stress versus horizontal displacement for reinforced Bedis sand with 1.0% plastic waste at 60% relative density

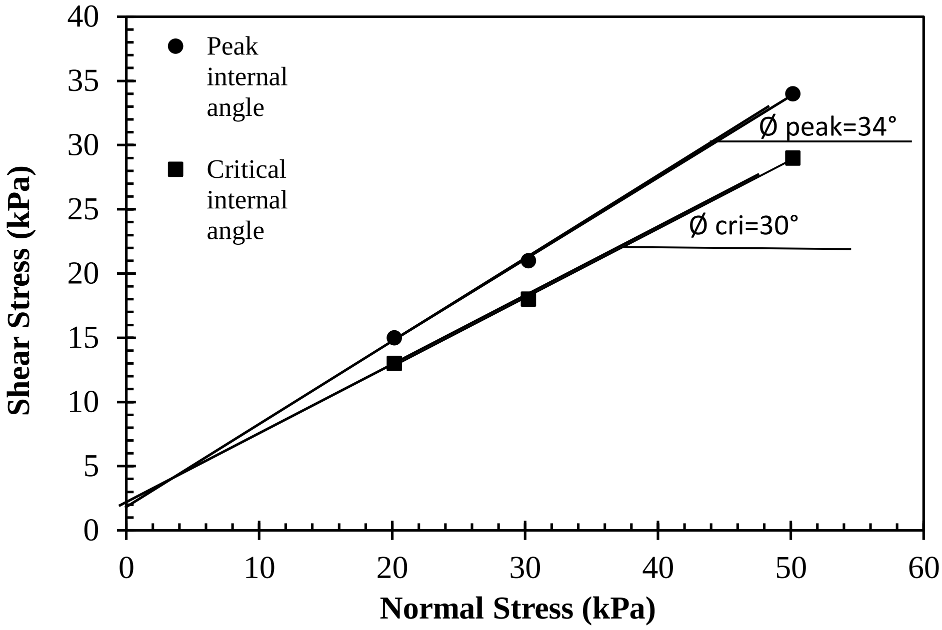


Figure 4.19: Shear stress versus normal stress for sand mixed with 0.5% plastic chips at 30% relative density

From Figure 4.4 and Figure 4.19, the results indicate that the shear strength parameters of the reinforced soil were improved when 0.5% plastic waste was added into the soil. With plastic chips reinforcement, the soil causes a cohesion 2 kPa, which is contributing to the shear strength.

From Figures 4.5 and 4.20, the result of peak internal friction angle of reinforced sand increased 1° and also, there was a very slight improvement (1 kPa) in the cohesion value.

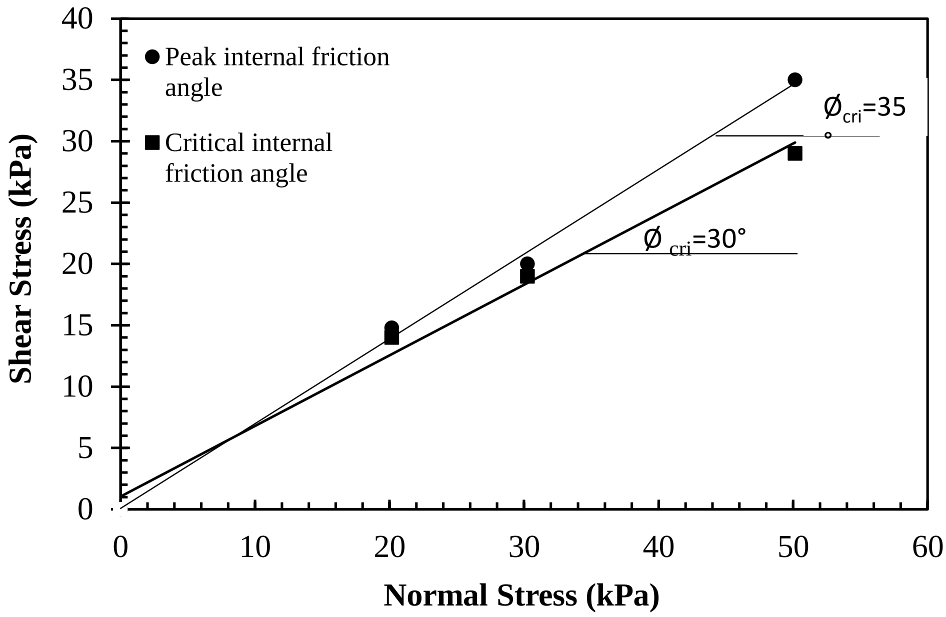


Figure 4.20: Shear stress versus normal stress for 0.5% plastic chips at 60% relative density

From Figure 4.4 and Figure 4.21, the results indicate that, the peak internal friction angle of reinforced sand increased 3° whereas, the critical internal friction angle increased 15°. Also, there is a very small increase (1 kPa) in the cohesion value.

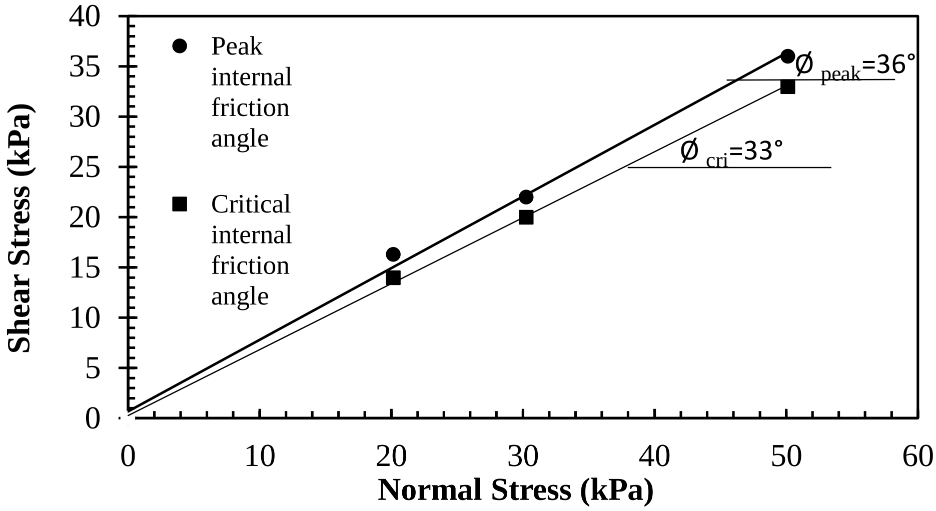


Figure 4.21: Shear stress versus normal stress for sand mixed with 0.75% plastic chips at 30% relative density

From Figures 4.5 and 4.22, it can be seen that, both the peak and the critical internal friction angles of sand reinforced with 0.75% of plastic chips at 60% relative density (dense state) were higher than the internal friction angles of unreinforced sand. The results indicate 3° increment in both the peak and the critical internal friction angles. The reinforced soil has cohesion, which is 1 kPa.

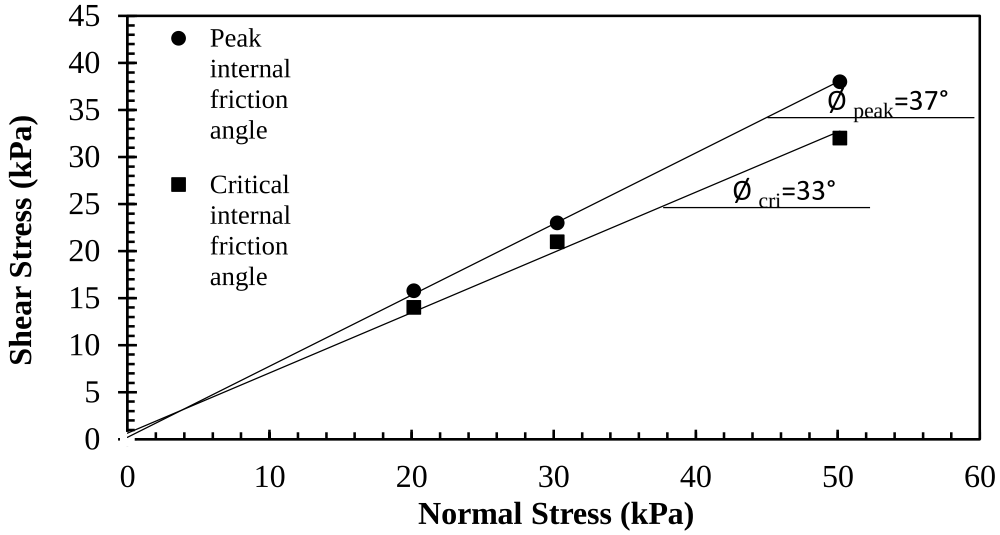


Figure 4.22: Shear stress versus normal stress for sand mixed with 0.75% plastic chips at 60% relative density

From Figure 4.4 and Figure 4.23, the results indicate that, when the sand was reinforced with 1% plastic chips at 30% relative density, the peak internal friction angle increased 1° whereas the critical internal friction angle increased 13°. The reinforced soil has a cohesion value of 1 kPa.

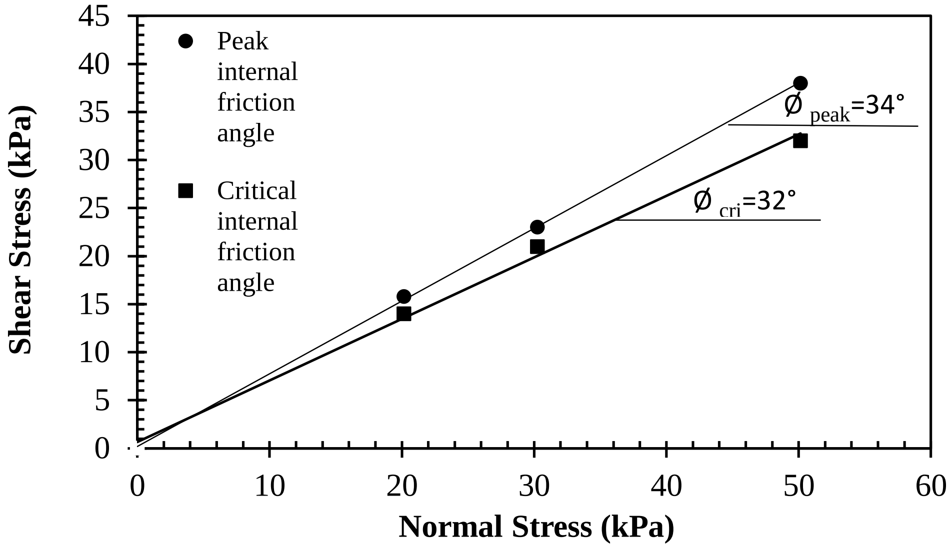


Figure 4.23: Shear stress versus normal stress for sand mixed with 1% plastic chips at 30% relative density

When the values obtained in Figure 4.5 were compared with the values obtained in Figure 4.24, it can be seen that no change in the peak and the critical internal friction angle was obtained for the sand reinforced with 1% plastic chips at 60% relative density. That can be explained due to the excess amount of plastic chip materials in sand which cause an increase in the void space and reduce the friction between the sand particles.Consequently, reduction in the internal friction angle was obtained.

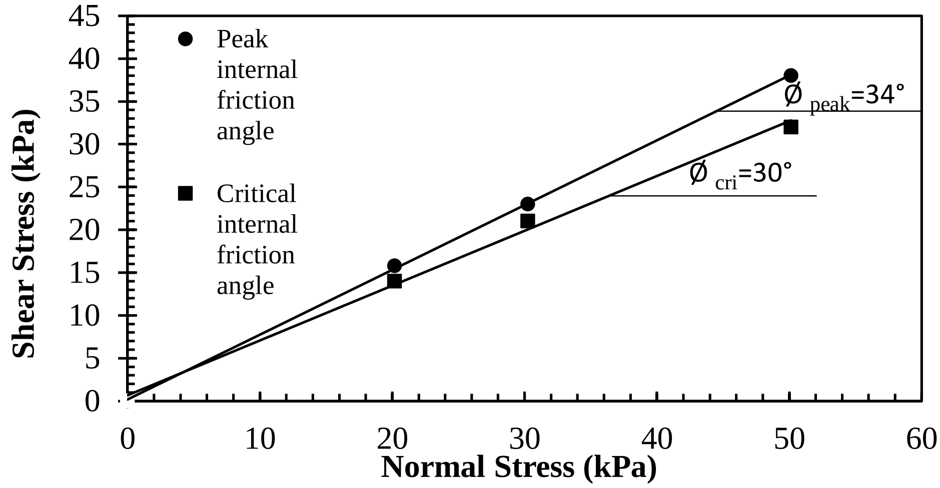


Figure 4.24: Shear stress versus normal stress for sand mixed with 1% plastic chips at 60% relative density

Tables 4.2 and 4.3 give the summary of the internal friction angles and cohesion values for natural and plastic chips reinforced sand in loose (30% relative density) and dense (60% relative density) states.

Table 4.2: The shear strength parameters for natural and reinforced sand in loose state (D_r= 30%)

	Friction Angle			
% Plastic chips	0%	0.50%	0.75%	1.0%
Peak internal friction angle (ϕ_{peak})	33 °	34 °	36 °	34 °
Critical internal friction angle (ϕ_{cri})	18 °	30 °	33 °	32 °
Cohesion(kPa)	0	2	1	1

Table 4.3: The shear strength parameters for natural and reinforced sand in dense state (D_r= 60%)

	Friction Angle			
% Plastic chips	0%	0.50%	0.75%	1.0%
Peak internal friction angle (ϕ_{peak})	34 °	35 °	37 °	34 °
Critical internal friction angle (ϕ_{cri})	30 °	30 °	33 °	30 °
Cohesion(kPa)	0	1	1	1

The above tables indicate that when the soil was reinforced with plastic waste, the soil developed a very small cohesion value contributing to the shear strength.

From Figures 4.25 and 4.26, the results indicate that the values of peak and critical internal friction angles were increasing with the increase in the percent of plastic waste up to 0.75%. After that percentage the values of internal friction angle start to decrease. There is no further increment in the shear strength parameters of the reinforced sand above 0.75%. This means that the optimum percent of plastic waste which should be used for reinforcing the Bedis beach sand is 0.75%. Above this value, the reinforcing effect of plastic waste on the shear strength of sand diminishes due the reduction in the contact surface area among the sand particle. The improvement of sandy soil with plastic waste up to 0.75% agrees with Babu and Chouksey (2011) and contradicts in 1% of plastic waste.

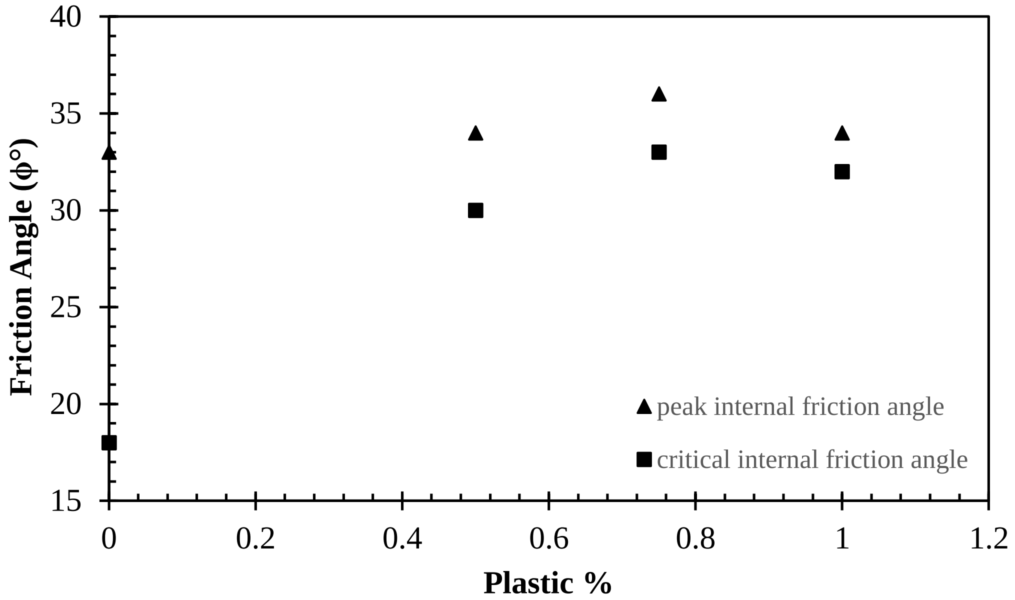


Figure 4.25: The internal friction angles versus percent plastic chips at 30% relative density

From Figure 4.26, the results clearly indicates that addition of 1% plastic waste caused a reduction in the friction angle even at 60% relative density. This result was for both peak and the critical internal friction angles in dense sand above 0.75% plastic waste reinforcement.

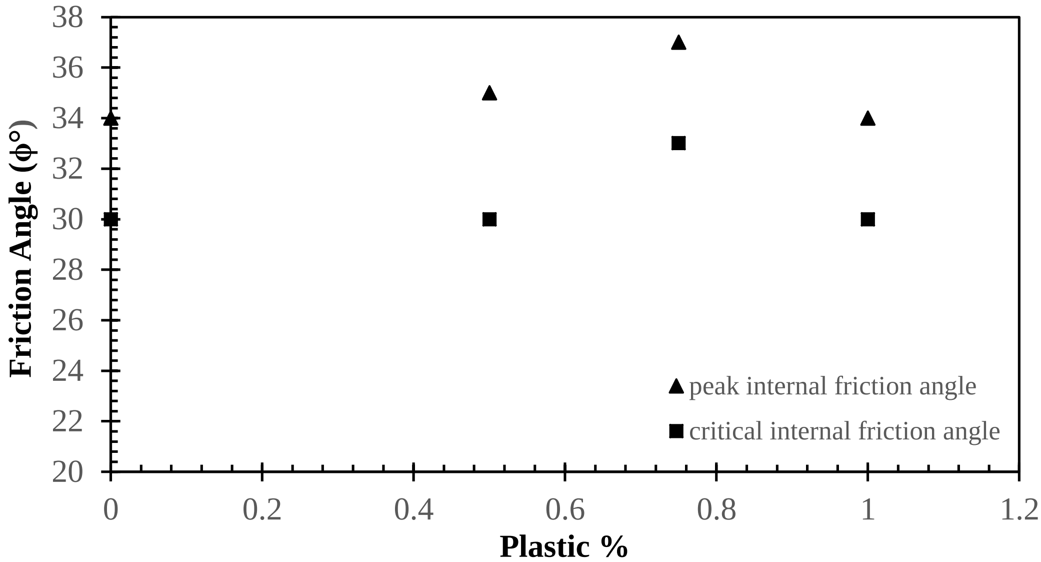


Figure 4.26: The internal friction angles versus percent plastic chips at 60% relative density

4.3.1.2 The impact of plastic chips on the CBR value of sandy soil

Figure 4.27 represents the curves of California bearing ratio of natural and 0.5% and 0.75% reinforced sand at 30% relative density. The results indicate that when the plastic chips were added into the soil, the CBR value increased from 9 to 10. That means that addition of plastic chips into the sand did not cause a significant change in the CBR value. The CBR values for 0.5% and 0.75% reinforced sand were similar. The results shown in Table 4.5 indicate that, the natural and the reinforced sand can be utilized as subbase materials.

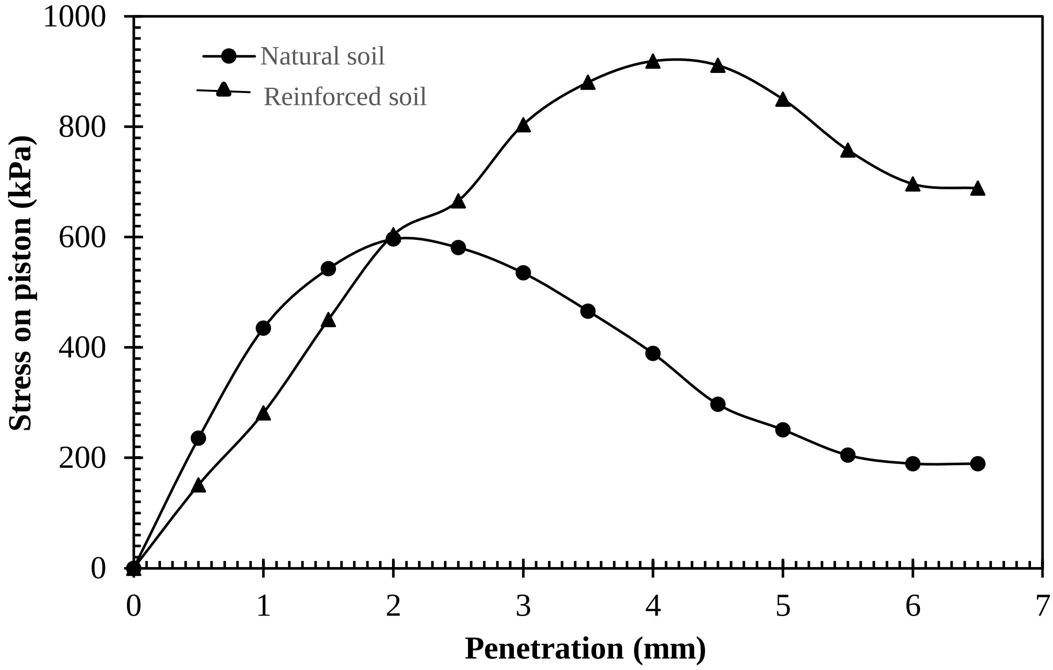


Figure 4.27: California bearing ratio curve for natural and 0.5 %plastic chips reinforced Bedis sand at 30% relative density

The unusual drop in the curve happened for sand because of that the sand has only friction and there is no cohesion. For this reason, after reaching the maximum penetration the sand particles started to move over each other (sand particles try to find new path) this causes the drop in the curve.

Figure 4.28 represents the curves of California bearing ratio of natural and reinforced sand soil with 0.5% plastic chips at 60% relative density. The figure indicates that when the plastic chips were added into the soil, the CBR value increased by 9% comparably with natural soil.

Apparently, as the soil sheared during penetration, strips fixed in the sand by friction elongated as the soil de-formed. The resistance to deformation provided by the strips was the likely cause of the increase in CBR value.

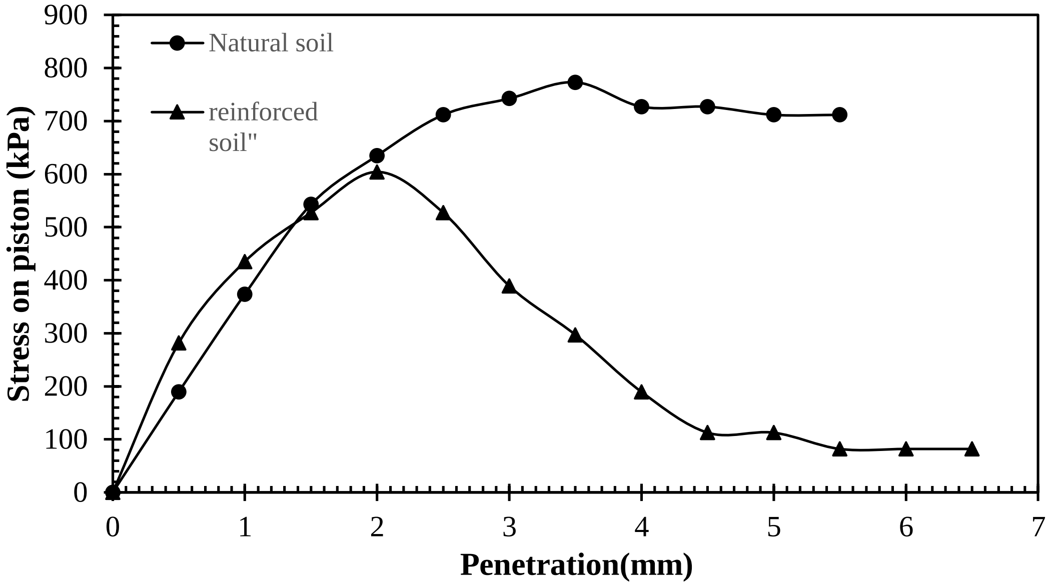


Figure 4.28: California bearing ratio curve for natural and 0.5% of plastic chips reinforced Bedis sand at 60% relative density

The unusual drop in the curve happened for sand because of that the sand has only friction and there is no cohesion. For this reason, after reaching the maximum penetration the sand particles started to move over each other (sand particles try to find new path) this causes the drop in the curve.

4.3.2 Tuzla soil

In this part of the study, the effect of plastic chips on the linear shrinkage, unconfined compressive strength, California bearing ratio, swell and compressibility characteristics of Tuzla soil will be discussed.

4.3.2.1 The influence of plastic chips on the linear shrinkage of reinforced soil

The linear shrinkage test results indicated that 0.75% of plastic chips reduced the linear shrinkage of reinforced soil from 18% to 16%.

4.3.2.2 The influence of plastic chips on the unconfined compressive strength of reinforced soil

Figure 4.29 represents the stress- strain curves, obtained from unconfined compression test. The test results indicate that the plastic chips of 0.1% decreased the unconfined compressive strength of soil about 39%. According to Table 4.4, before reinforcing the Tuzla soil with plastic chips, consistency of the natural compacted soil was hard but when the plastic waste was added into the soil, the consistency of the soil changed and it became very stiff. The presence of plastic chips in the soil increased the void ratio and a consequent reduction in the dry density of the soil was obtained (Table 3.3). Increase in the void ratio caused a softening in the soil and that resulted in a changed in the soil's consistency. The value of unconfined compressive strength of the soil reinforced with 0.5% of plastic waste decreased around 39% from the original compacted soil. The unconfined compressive strength of reinforced soils reduced significantly.

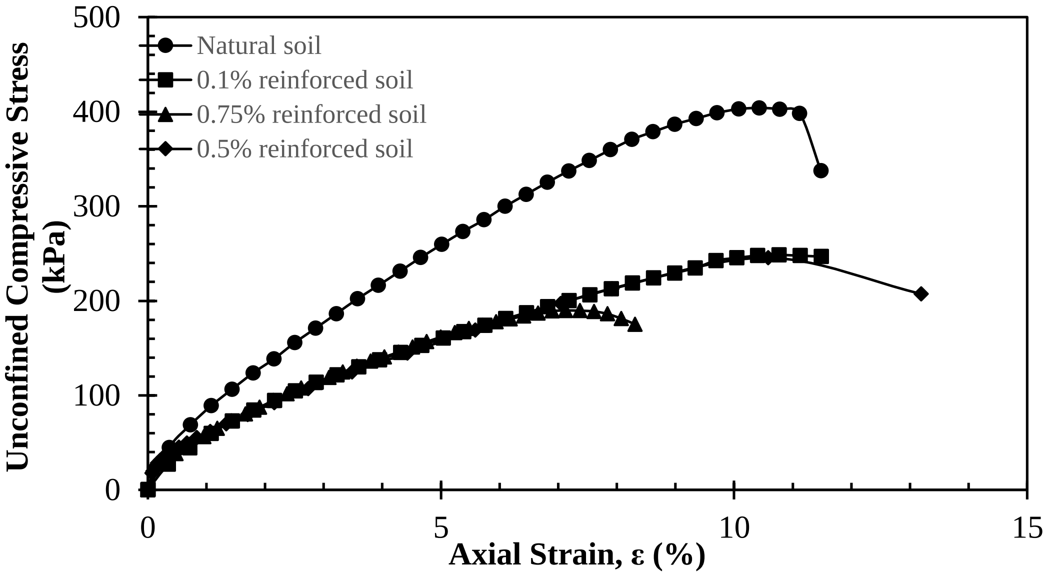


Figure 4.29: Shear stress- shear displacement curves of natural and reinforced soils with various percentages of plastic chips

Table 4.4: Consistency and unconfined compressive strength of clays (Das, 2008)

Consistency	q_u (kN/m^2)
Very soft	0-24
Soft	24-48
Medium	48-96
Stiff	96-192
Very stiff	192-383
Hard	>383

The Figure 4.30 summarises the unconfined compressive strength of natural soil and the reinforced soil with different percentages of plastic chips (0.1%, 0.5%, and 0.75%). The results indicate that at all percentage of plastic chips, the unconfined compressive strength of the soil decreases.

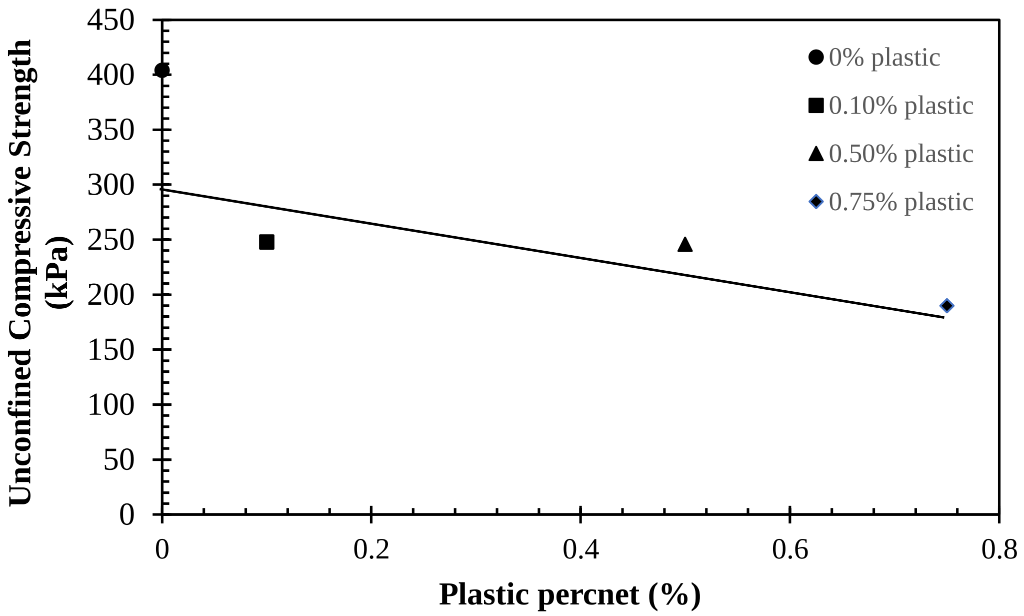


Figure 4.30: The effect of plastic waste on the unconfined compressive strength of Tuzla soil

4.3.2.3 The influence of plastic chips on the California Bearing Ratio Of reinforced soil

Figure 4.31 represents the CBR test results for the natural soil and the reinforced soil with 0.75% of plastic chips. The test sample in this test was compacted at optimum moisture content and dry density value refered to in Table 3.3 and the CBR test on this sample was performed without soaking. The results of the CBR values were 7.2 for natural soil and 8.7 for reinforced soil. Test result indicates that the increase in the CBR value was achieved when the plastic chips were added into the soil. The CBR value of 0.75% plastic chip reinforced soil increased by 1.5%. According to Table 4.5(Joseph Bowles, 1970), which refers to the CBR values for the need of specifying the proper soil required for road and foundation construction, the value of CBR for 0.75% plastic chip reinforced soil is fair as a subbase material for road constructions.

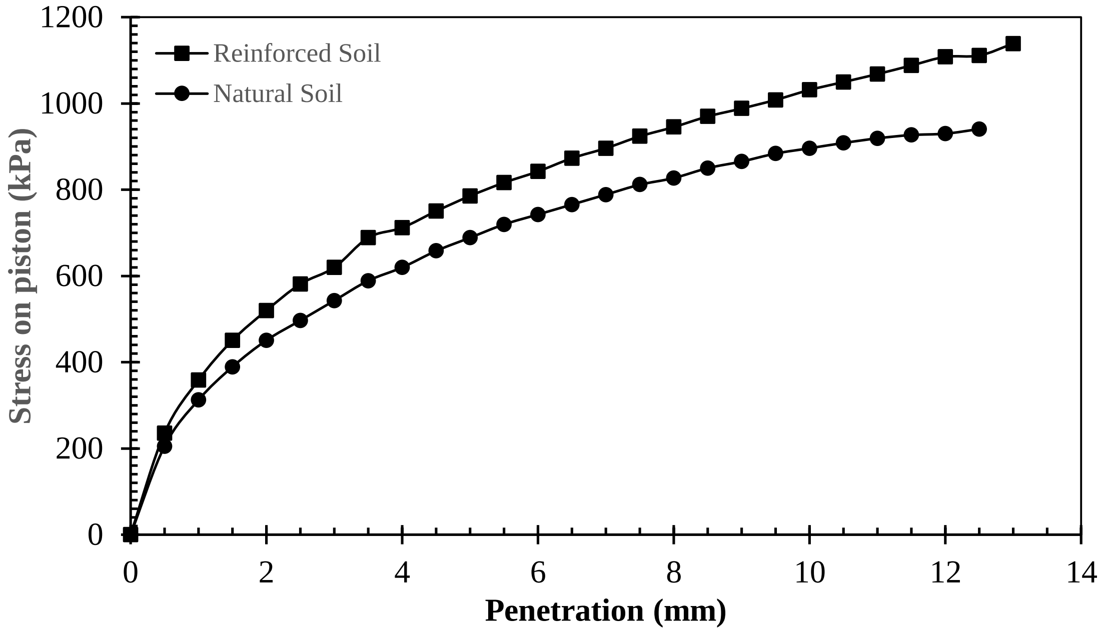


Figure 4.31: Stress on piston and penetration for natural and reinforced soils without soaking

Table 4.5: The USCS for CBR numbers (The Asphalt Institute, 1970)

CBR No. (%)	General rating	Uses	Unified
0 – 3	Very poor	Subgrade	OH, CH, MH, OL
3 – 7	Poor to fair	Subgrade	OH, CH, MH, OL
7 – 20	Fair	Subbase	OL, CL, ML, SC, SM, SP
20 – 50	Good	Base, subbase	GM, GC, SW, SM, SP, GP
> 50	Excellent	Base	GW, GM

Figure 4.32 shows the CBR curves of compacted natural and reinforced soil with plastic chips of 0.75%. In this case, the samples were soaked for 4 days to determine the CBR value under the worst condition. The CBR values for natural and reinforced samples were 2.7, and, 1.8 respectively. Test results indicate that the CBR value of the reinforced soil was less than the natural soil. That can be explained because of the plastic chips resulting in more empty space in the soil and enabling the penetration of water into the reinforced soil during soaking. This means that more water penetrated into the reinforced soil reduced the soil's strength and caused a reduction in the CBR value.

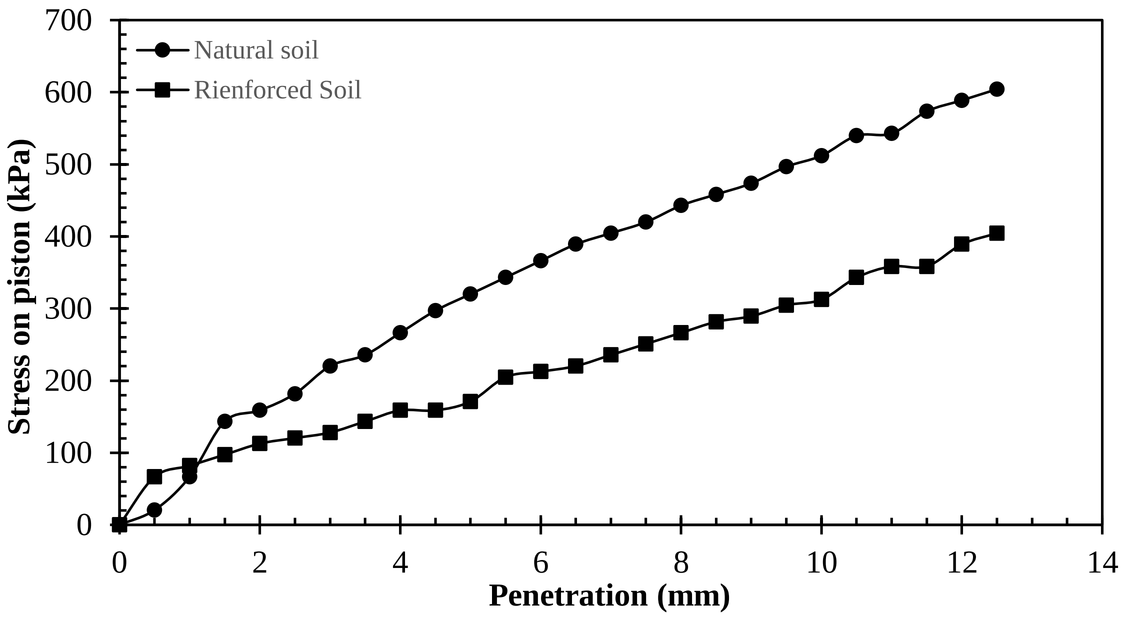


Figure 4.32: Stress on piston and penetration for natural and reinforced soils

Figure 4.10 shows the CBR curves for the natural soil and Figure 4.33 shows the CBR curves for soaked and unsoaked reinforced soils. From Figure 4.10, the results indicate that the CBR value for natural soil decreases from 7.2 (unsoaked) to 2.7 (soaked). This reduction is about 62.5% from the original value. The reduction in the CBR value was from 8.7 (unsoaked) to 1.8% (soaked) in the reinforced soil. This decrease is nearly 80% of the original unsoaked sample. It is obvious that the reduction in the CBR value is high in the case of reinforced soil. As discussed earlier, because of gap produced due to the presence of plastic waste in the soil, more water was able to infiltrate into the soil easily. Consequently, the soil became softer and resulted in reduction in the CBR value.

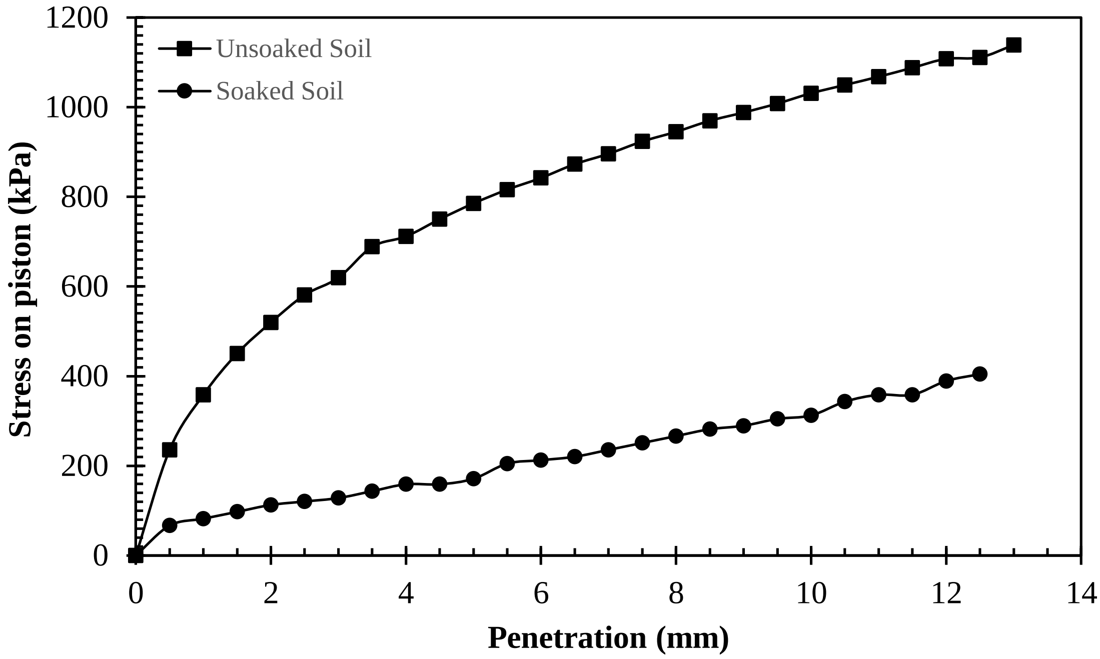


Figure 4.33: Stress on piston and penetration for reinforced soil with plastic chips with and without soaking

4.3.2.4 The influence of plastic chips on one-Dimensional swell of reinforced soil

From Figures 4.34 and 4.35, the change in the values of swelling of the soil with time can be seen. The curve represents the swelling of 0.75% plastic chips reinforced soil. The test results indicate that the primary swelling achieved in 24 days is 4.1%.

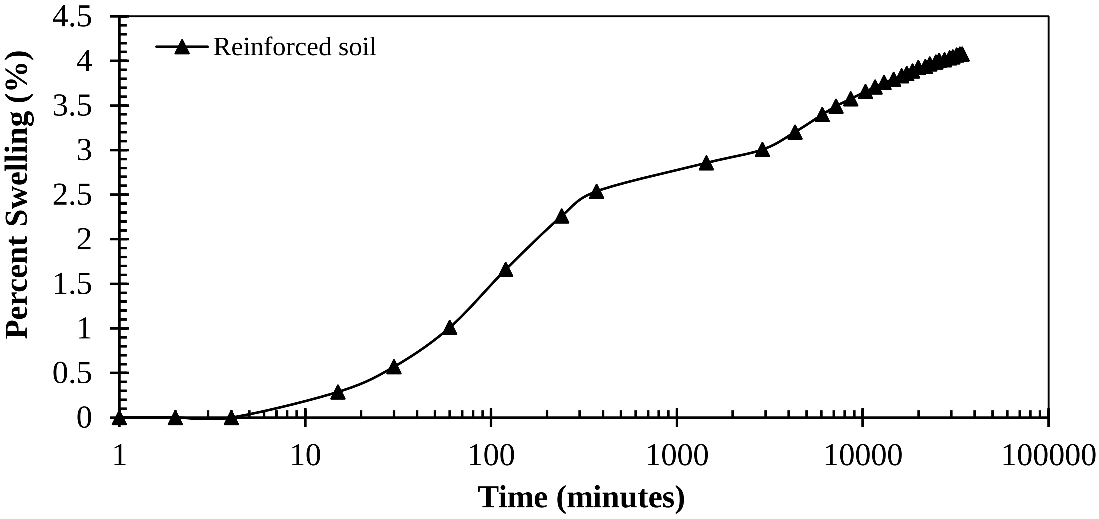


Figure 4.34: Swelling percent versus log time for reinforced soil with 0.75% plastic waste

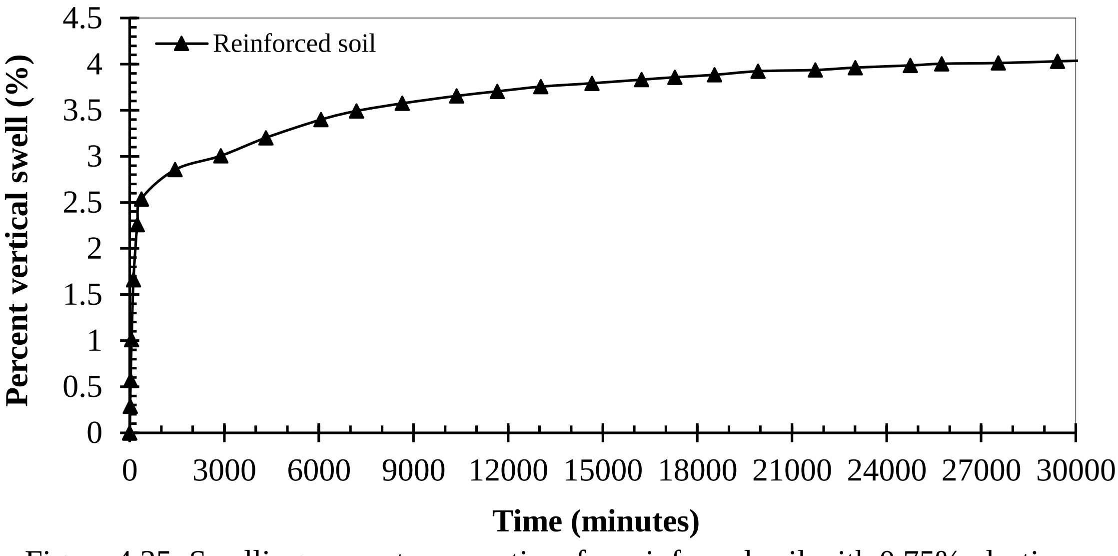


Figure 4.35: Swelling percent versus time for reinforced soil with 0.75% plastic waste

In Figures 4.36 and 4.37, the results indicate that the percentage of primaray swelling was 4.5% and this value decreased to 3.3% with 0.75% plastic chips the reinforcement.

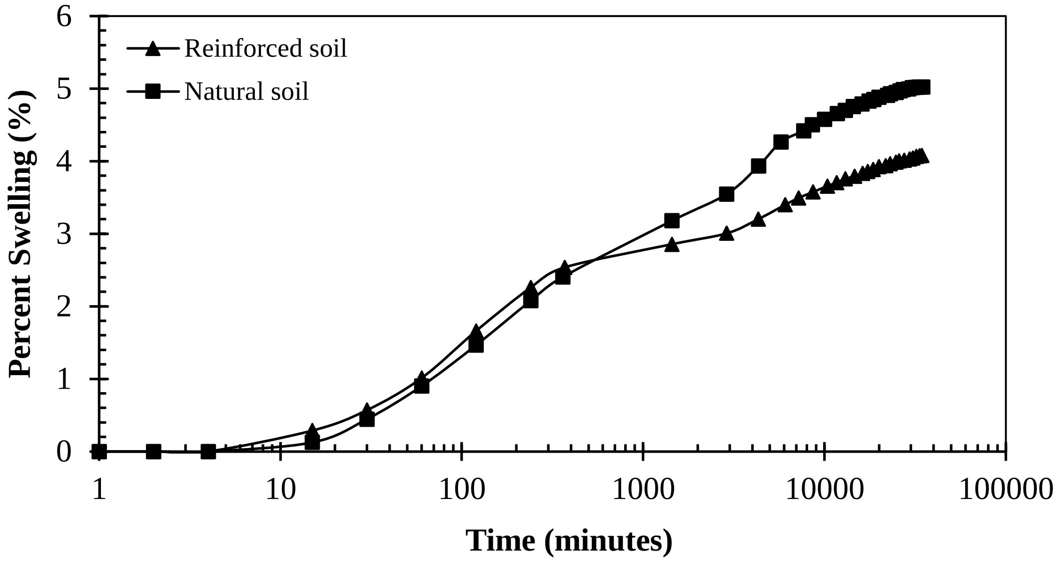


Figure 4.36: Swelling percent versus log time for natural and reinforced soil with 0.75% plastic

In the beginning of the 0.75% reinforced soil swelling test, it was observed that the swelling dial gauge was moving faster than native soil. This behaviour can be explained due to the spaces produced as a result of the presence of the plastic waste, enabling water to penetrate into the soil pore spaces faster and accelareting the swell.

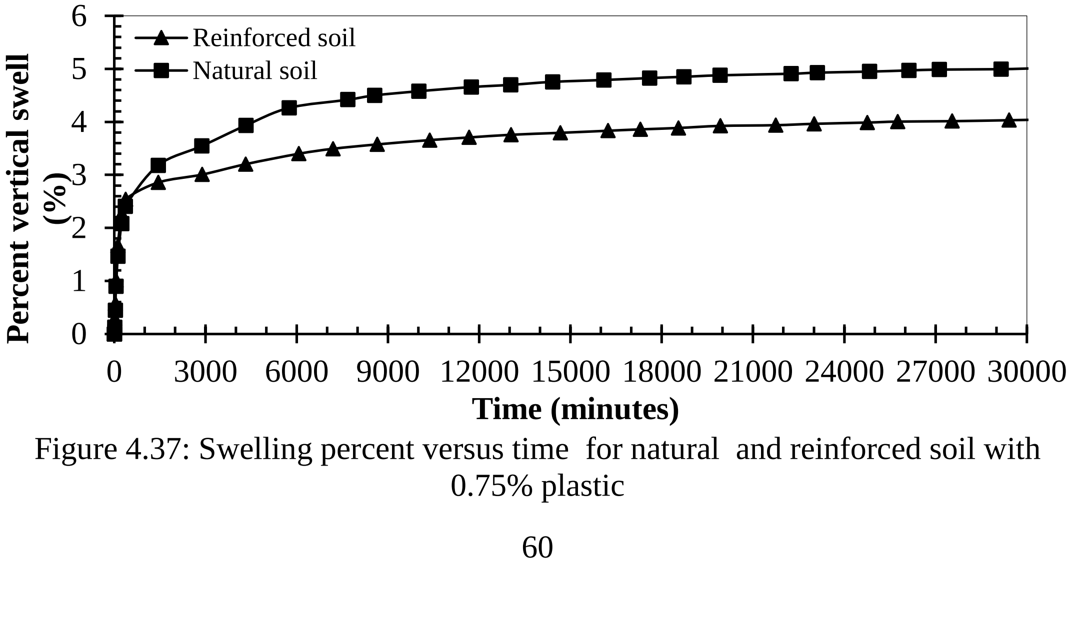


Figure 4.37: Swelling percent versus time for natural and reinforced soil with 0.75% plastic

4.3.2.5 The influence of plastic chips on one-Dimensional consolidation of reinforced soil

From Figure 4.38 the void ratio and the applied pressure were presented for natural and reinforced soil with plastic chips of 0.75%. From Figure 4.39, the compressibility characteristics of the soil, the swelling pressure and the preconsolidation pressure were determined and these values were presented in Tables 4.7 and 4.8.

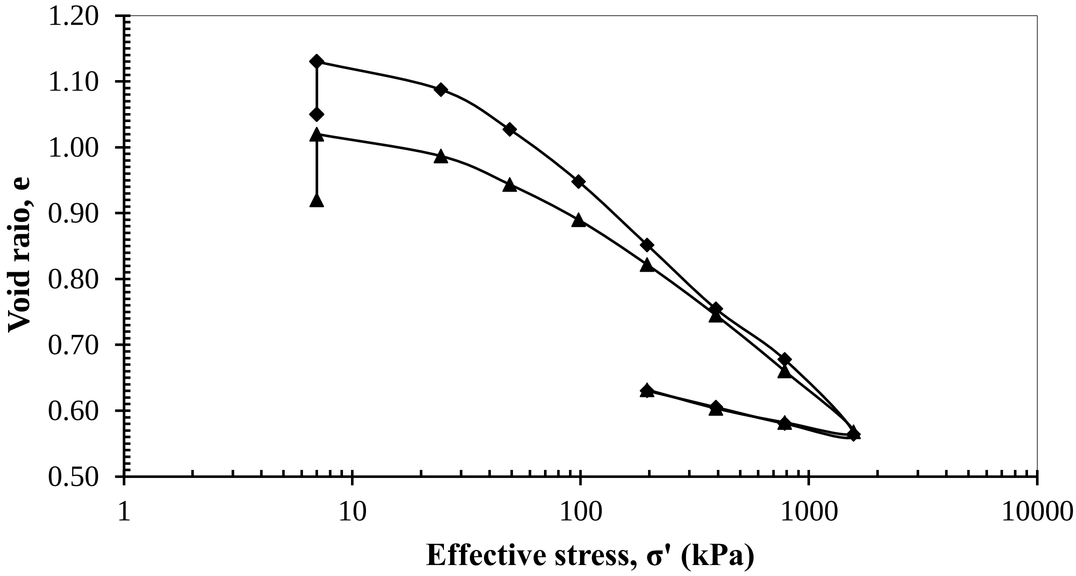


Figure 4.38: Void ratio versus log pressure curves for native and reinforced soil

Table 4.6: Swell and compressibility characteristics of natural and reinforced soil with 0.75% of plastic waste

Soil type	Coefficient of Consolidation C_v (m^2/yr)	Compression index C_c	Swelling index C_s	Time required for 90% consolidation t_{90} (min)
Natural Soil	0.46	0.31	0.09	17.00
Reinforced Soil	0.42	0.28	0.09	20.00

Table 4.7: Swelling pressure and preconsolidation pressure natural and reinforced soil With 0.75% of plastic

Soil type	Swelling Pressure (kPa)	Preconsolidation Pressure (kPa)	Hydraulic conductivity K (m/s)
Natural Soil	80	100	3.27E-11
Reinforced Soil	40	150	3.52E-11

From Table 4.7, the results showed that the addition of plastic chips to the soil decreased the swelling pressure of soil from 80 to 40 kPa, which represents an increase of about 50%. Test results in Table 4.8 indicate that the natural soil had an apparent preconsolidation pressure of 100 kPa and with the addition of 0.75% plastic chips, the apparent preconsolidation pressure increased from 100 to 150 kPa, Increase in the apparent preconsolidation pressure indicates a reduction in the compressibility of the soil.

From Table 4.6, it can be seen that due to the presence of plastic waste the compression index had an insignificant change.

Chapter 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusions

From the results of the experimental work carried out on the engineering properties of natural and reinforced sandy and silty clay soils, the following conclusions can be made:

- For Bedis sand, the inclusion of plastic chips of waste bottles increased the shear strength of the soil. The values of the peak and the critical internal friction angles increased with increase in the percent of plastic chips up to 0.75% by dry weight of sand. The enhancement up to 0.75% plastic chips was due to the development of the friction surfaces between the plastic chip pieces and the soil particles. Addition of plastic chips beyond 0.75% by dry weight of sand decreased the peak internal friction angle slightly. The optimum percentage of plastic waste for reinforcing the Bedis beach sand was found to be 0.75%. Above this value, the reinforcing effect of plastic waste on the shear strength of sand diminishes due to the reduction in the contact surface area among the sand particles.
- For Bedis sand, the inclusion of plastic waste into the soil caused a slight increase in cohesion, which increases the shear strength of soil.
- For Bedis sand, apparently as the soil sheared during penetration, strips fixed in the sand by friction elongated as the soil de-formed. The resistance to

deformation provided by the strips was the likely cause of the increase in CBR value.

- For Tuzla soil, reduction in the unconfined compressive strength of reinforced soil was obtained. Test results indicated that plastic chips did not improve the shear strength parameters of Tuzla soil. The presence of plastic chips in the soil increased the void ratio and a consequent reduction in the dry density of the soil. Increase in the void ratio caused a softening in the soil and that resulted in a changed in the soil's consistency.
- For Tuzla soil, the optimum percentage of plastic waste found for the improvement of the CBR value was 0.75%. However, when the soaked reinforced sample was tested, reduction in the CBR value was obtained.
- Addition of 0.75% of plastic chips in Tuzla soil improved the compressibility and the swelling behavior of Tuzla soil. The linear shrinkage of reinforced soil was reduced by a noticeable amount. The plastic chips also decreased the swelling potential of Tuzla soil. Also, reduction in the swelling pressure was obtained.
- For Tuzla soil, test results indicate an increase in the apparent preconsolidation pressure of Tuzla soil. The preconsolidation pressure increased from 100 to 150 kPa with 0.75% plastic waste. Increase in the preconsolidation pressure indicates an induced overconsoildation in the soil.
- For Tuzla soil, the compression index (C_c) had insignificant reduction when the soil reinforced with plastic chips.

5.2 Recommendations

Literature review indicates that, few investigations have been done on the reinforcing of sand and clay by using waste plastic bottles. Specially, the effect of different percentages of plastic waste on the swelling, consolidation and CBR characteristics of clayey soil needs to be further studied.

In this study, only one percentage of plastic chips was used to study the swelling, consolidation and CBR values. For further studies, different percentages and sizes of plastic waste should be used. In addition, another cementing material can be blended with plastic chips such as lime and cement in order to increase its effect on geotechnical properties of soils.

In this study, the cutting of plastic bottles into small chips was time consuming and very difficult to attain the required size as described in materials and methods section. For further studies, it is better to search for recycling industries, which most probably may help to get the required material in different shapes and sizes.

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