Effect of Fibreglass on Shear Strength and Dilation Characteristics of Sand

Fatemehzahra Ahooee

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Prof. Dr. Serhan Çiftçioğlu
Acting Director

I certify that this thesis satisfies the requirements as a thesis for the degree of Master of Science in Civil Engineering.

Prof. Dr. Özgür Eren
Chair, Department of Civil Engineering

We certify that we have read this thesis and that in our opinion it is fully adequate in scope and quality as a thesis for the degree of Master of Science in Civil Engineering.

Assoc. Prof. Dr. Zalihe Sezai
Supervisor

Examining Committee

1. Assoc. Prof. Dr. Huriye Bilsel
2. Assoc. Prof. Dr. Zalihe Sezai
3. Asst. Prof. Dr. Eriş Uygar

ABSTRACT

In recent years, environmental awareness about the improvement of soil and recycling of waste materials are gaining more interest among the soil scientists and geotechnical engineers. Fibreglass is a type of plastic and lightweight material. This study was conducted to investigate the effect of fibreglass reinforcement content on the strength parameters of a typical beach sand. The shear strength of unreinforced sand and reinforced sand are carried out by direct shear box apparatus and California Bearing Ratio, CBR device. Three different contents of fibreglass by dry unit weight and relative density were used in this thesis. The percentage of fibreglass content was 0.5%, 1% and 1.5% by dry weight. Relative density was at three different states, loose, medium dense and dense. In direct shear test, it was observed from the results of the tests that the shear strength of sand was not influenced significantly by fibreglass content. An increase of the fibre content caused a decrease in the peak friction angle and dilation angle. According to the results, addition of the fibreglass reinforcement more than 0.5% by dry unit weight of sand, did not improve the shear strength of sand. In other words, the optimum fibreglass content was 0.5 percent. The CBR value increased with the addition of fibreglass reinforcement. Addition of 0.5% of fiberglass reinforcement caused an increase in both the CBR values and shear strength of the soil. Therefore, fibreglass reinforcement improved the engineering properties of the reinforced sands.

Keywords: Sand stabilisation, dilation angle, direct shear box test, plastic fibreglass, shear strength parameters.

ÖZ

Son yıllarda, atık maddelerin geri dönüşümü ve zemin iyileştirilmesi konusunda çevre bilinci, zemin bilimleri ve jeoteknik mühendisleri arasında daha fazla ilgi kazanmaktadır. Cam elyafı plastik ve hafif bir malzeme türüdür. Bu çalışmanın amacı, cam elyafı takviye etkisinin tipik bir plaj kumunun kayma dayanımı parametreleri üzerindeki etkisinin araştırılmasıdır. Doğal ve takviyeli kumun kayma dayanımı parametreleri ve penetrasyon direnci, sırası ile, direkt kesme kutusu deneyi ve Kaliforniya taşıma oranı düzeneği kullanılarak belirlendi. Bu çalışmada kuru birim ağırlığın üç farklı yüzdelik değerinde ve farklı yoğunluk değerlerinde deneyler gerçekleştirilmiştir. Bu çalışmada kullanılan cam elyafı yüzdesi kumun kuru ağırlığının % 0.5, % 1 ve % 1.5 değerlerindedir. Kum numuneler üç farklı bağıl yoğunluk değerleri ve sıkışma yüzdelik değerlerinde: gevşek, orta sıklıkta ve yoğun sıkılıkta hazırlanmıştır. Direkt kesme kutusu testlerinin sonuçlarına bakıldığında, cam elyaf takviye yüzdesi artışının kum kayma gerilmesi üzerinde pek fazla bir etkisinin olmadığını göstermiştir. Test sonuçlarına göre, cam elyaf yüzdesinin, kumun kuru birim ağırlık yüzdesinin 0.5 değerinden yüksek olması, kumun kesme mukavemeti üzerinde olumlu bir etkiye sahip değildir. Cam elyaf optimum oranı yüzde 0.5 olarak bulunmuştur. Cam elyaf ilavesi ile KTO artmıştır. Yüzde 0.5 değerine kadar cam elyaf ilavesi KTO ve kayma mukavemetinde artışa neden olmuştur. Böylede, cam elyaf takviyesi takviyeli kumun mühendislik özelliklerini iyileştirmiştir.

Anahtar Kelimeler: Kaliforniya Taşıma Oranı, dilatasyon açısı, direk kesme deneyi, cam elyaf, maksimum sürtünme açısı, kayma mukavemeti parametreleri.

DEDICATION

To My Supportive Father;

My Symbol of Strength

Who Offered Me Full Support in Life...

And My Affectionate Mother;

My Symbol of Patience

Who Taught Me the Alphabets of Life …

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LIST OF SYMBOLS

e	Insitu void Ratio
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
G_s	Specific gravity
ρ_{max}	Maximum dry density
ρ_{min}	Minimum dry density
c'	Cohesion
c_c	Coefficient of curvature
c_u	Coefficient of uniformity
ρ	Fibre concentration
V	Volume of the sand
V_r	Volume of the Fibres
λ	Thermal conductivity coefficient
τ	Shear stress
ψ	Angle of dilation
dh	Horizontal displacement
$d\nu$	Vertical displacement
φ_p	Peak friction angle
φ_c	Critical friction angle
S	Sphericity
R	Roundness

LIST OF ABBREVIATIONS

ASTM	American Society for Testing and Materials
CBR	California bearing ratio
Dr	Relative density
DSBT	Direct shear box test
GRP	Glass reinforced plastic

Chapter 1

INTRODUCTION

1.1 Introduction

The purposes of a geotechnical investigation are to study the characteristics of soil and geological conditions at a site and to provide recommendations for design and construction. Geotechnical investigations are also performed to obtain information on the physical properties of soil. Improvement of soil is a challenge. Soils can be reinforced with natural and artificial materials.

The aim of this study is to investigate the effect of fibreglass on shear strength parameters and dilatancy characteristics of sand. The primary purpose of reinforcing soil is to increase its shear strength thereby increasing its bearing capacity and reducing its compressibility. This research presents data obtained from laboratory tests on sand reinforced with varying fibreglass content. The sand samples used in the laboratory tests are also prepared in variation relative densities.

1.2 Objective of Research

The objective of this research is to investigate the effect of fibreglass content on shear strength of sand. The purpose of mixing sand with fibreglass is to increase the shear strength and bearing capacity and to reduce compressibility.

In this study, artificial fibreglass is used with varying percentage: 0%, 0.5%, 1% and 1.5%. Some physical tests were carried out on natural sand without any reinforcement. The artificial fibreglass used in this study is glass-reinforced plastic, GRP obtained from Iran. The length of fibre is approximately in the range 10mm-20mm. The sand used in this study was taken from the coastline of Palm Beach, Gazimağusa in North Cyprus.

1.3 Outline

This thesis is based on five chapters:

- The first chapter includes the explanation of the aim and objective of the research. In this section, basic information about the previous research, methods of reinforcements and testing are explained.
- The second chapter (literature review), gives as summary of the previous researches on this subject and discusses the effect of relative density, the particle size distribution and particle shape on the dilatancy of sand and methods used for increasing the shearing resistance of dense and loose sands.
- The third chapter (materials and methodology), gives the details of physical properties of sand and GRP used in the present study and introduces the testing methods. The site and the method of sampling and all the tests which were performed within this study are discussed in this chapter. In the present study, all the tests were performed in accordance with American Society Testing and Materials (ASTM) standards.
- The fourth chapter (result and discussion), includes the results and discussion of the experimental findings.
- Finally, in chapter five, conclusion and recommendation for further research on this subject are presented.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

In recent years, environmental awareness and high dumping cost of glass wastes have encouraged the use of recycled glass fibres (GRP) in construction industry such as building materials, concrete and ground improvements. This chapter discusses the effect of relative density and the particle shape on the dilatancy of sand and remediation methods used for increasing the shearing resistance of loose sands.

2.2 Particle Size

According to Blyth (1972), the shape and size distribution of sand particles depend on the formation history of the grains where it results from the disintegration of rocks due to weathering. As Goktepe (2010) noted, the fabric of sand particles are the most determining factors in controlling the soil behaviour when it is used for different purposes.

Koerner (1970) studied the influence of the effective grain size D_{10} for saturated sandy soils, with effective grain size varying from fine gravel (2.6 mm) to clay size. The results indicated that friction angle increases with decreasing effective grain size. Zelasko et al. (1975) tested three types of sand and also observed that an increase in the mean grain size causes a slight decrease in the friction angle. However, they reported that the overall effect of grain size on the shear strength is not significant.

Similarly, Bishop (1948) came to the same conclusion in his study. Also, Kirkpatrick (1965) studied the effects of particle size by examining two cohesionless materials. The results showed that an increase in particle size reduced the angle of internal friction. In addition, Hough (1957) emphasized that particle size influences the improvement of strength through controlling the amount of shearing displacement needed for both eliminating interlocking and bringing the solids to a free sliding position.

Particle size distribution test is carried out to study physical properties of soils. The conventional method of characterizing particle sizes in soils is to divide the array of possible particle sizes into three arbitrary separable size ranges, namely sand, silt, and clay (Lin-Sien Lum, 2001).

2.3 Particle Shape

One of the early studies conducted by Terzaghi (1925) was influencial in understanding the particle shape characteristics of sand soils. Gilboy (1928) indicated that any kind of analysis or classification of soil, which neglects the presence, and effect of the shape, will be incomplete and erroneous. Holubec and D'Appolonia (1973) stated that the results of dynamic penetration tests in sands are affected by the particle shape. In addition, some studies (e.g., Confort, 1973; Holtz & Kovacs; 1981) emphasized on the effect of particle shape on the friction angle. Cedergen (1989) added that particle shape impacts the permeability. Kramer (1996) also reported that particle shape can have a substantial role in liquefaction potential.

Several studies (Wadell 1932; Krumbein 1941; Powers 1953; Holubec and D'Appolonia 1973; You'd 1973; Mandelbrot 1977; Hyslip and Vallejo 1997 Cho et al.,

2006) have introduced a detailed explanation of particle shape. Generally, three independent properties are employed to analyse the shape of a soil particle. These properties are roundness, sphericity, and smoothness, form which the overall shape of a particle can be defined.

Particles shape plays an important role in the shear strength properties of the sand. Figure 2.1 shows that the roundness of the particles plays a significant role on the critical friction angle compared the sphericity. Smooth and round particles move around each other easily, indicating low dilation angles. Therefore, both shape and size distribution characteristics of sand particles play a significant role in the shear strength behaviour of sand. The shape of sand particles are dependent on the mechanical and chemical weathering processes on the parent rock (Rahaman, 1995; Krinsley 1974). The transition region from chemical to mechanical shape control occurs for a particle size between d~50 and 400 μm. In this respect, there are generally three significant scales in a particle’s shape (Rahaman, 1995; Krinsley 1974). Rahaman (1995) and Krinsley (1974) provided definitions and their conventional evaluation in the form of dimensionless parameters as follows:

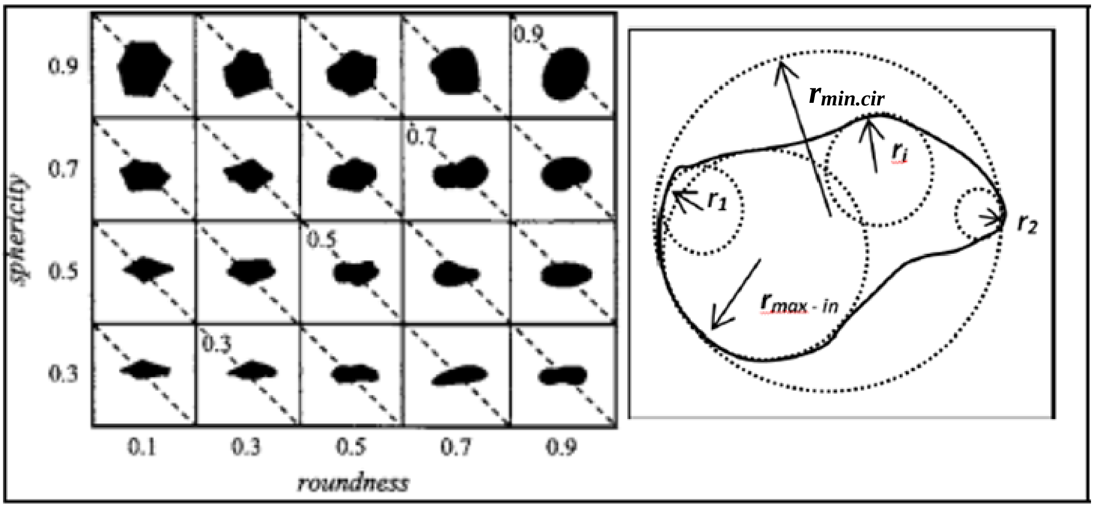

Figure 2.1. Particle Shape (Krumbein and Sloss 1963; Cho et al. 2006)

- Sphericity (S): It refers to the global form of the particle and indicates the similarity between the particle's length, height, and width. Sphericity can be measured as the diameter of the largest inscribed sphere relative to the diameter of the smallest circumscribed sphere. The sphericity can be given by the following equation:

$$S = \frac{r_{max-in}}{r_{min-cri}} \quad (2.1)$$

where,

$r_{max\text{-}in}$ = the radius of the largest inscribed sphere

$r_{min\text{-}in}$ = the radius of the smallest circumscribed sphere

- Roundness (R): It explains the scale of major surface features which is typically one order of magnitude smaller than the particle size. The roundness is quantified as the average radius of curvature of surface features in relation to the radius of the maximum sphere that can be inscribed in the particle (Rahaman 1995). The roundness can be given by the following equation:

$$R = \frac{\Sigma r_i / N}{r_{max-in}} \quad (2.2)$$

where,

$\Sigma r_i /N$ = the average radius of curvature of features

$r_{max\text{-}in}$ = the radius of the smallest circumscribed sphere

- Smoothness: Deals with the smoothness of the particle surface texture in relation to the radius of the particle (Rahaman, 1995).

Consequently, the particle size distribution as well as the shapes of the coarse grained sand particles are required to be studied, as they have significant impact on the shear strength behaviour.

2.4 Specific Gravity

Specific gravity, Gs is defined as the ratio of the mass of a unit volume of soil at a given temperature to the mass of the same volume of gas-free distilled water at a given temperature. Typical values for Gs are given in Table 2.1:

Table 2.1. Typical Gs values for soil (ASTM D854)

Type of Soil	Gs
Sand	2.65- 2.67
Silty Sand	2.67- 2.70
Inorganic Clay	2.70- 2.80
Organic Soil	< 2.00

2.5 Relative Density

Relative density and percent compaction are generally utilized for gauging the state of compatibility of a given soil mass. In this respect, some engineering properties such as shear strength, compressibility, and permeability of a given soil depend on the rate of compaction. Relative density data can be correlated to liquefaction resistance and to penetration testing. Therefore, penetration tests are the main method of examining the liquefaction resistance.

Relative density has been defined by geotechnical engineers as the compatibility of sand and gravel soils with few fines. These tests have been standardized by The American Society for Testing and Materials (ASTM, 1973) as density is the most basic way to improve soil quality (ASTM D4253).

Since 1922, the concept of relative density has widely been employed to describe the state of compactness of cohesionless granular soils. It involves comparing the natural or compacted density of soil to the minimum and maximum densities, which can be determined in the laboratory.

2.6 Friction Angle

The friction angle is the shear strength parameter used in the analysis of the response of sand to shearing. The variation of friction angle with respect to stress level, fabric, as well as particle damage have been debated in the research to date. Terzaghi et al. (1996) concluded that, surface roughness is directly correlated to the strength, texture, and hardness of the particles, which in turn are determined by the crystal structure of the constituent minerals as well as the inter crystalline bonds.

The peak strength is apparently accompanied with a reduction of shear stress. The theoretical state at which the shear stress and density remain constant while the shear strain increases is called the critical state, or residual strength (Roscoe, Schofield & Wroth 1958). The contractive and dilative specimens reached a critical state in the shear tests, and therefore critical state friction angles are defined from both dense and loose specimens.

The peak friction angle in sands is a result of the combined effects of relative density, mean effective stress, loading path, and basic frictional shear strength as reflected in the value of the critical state friction angle.

2.7 Direct Shear Box Test

The direct shear box test equipment mainly includes a metal shear box into which the soil specimen is placed. The specimen can be square or circular in plan, about 19–25cm^2 in the area, and about 25 mm in height. The box is split horizontally into two halves. Then, shear force is applied to the side of the top half of the box to cause failure in the soil specimen. During the test, the shear displacement of the top half of the box and the change in specimen thickness is recorded by the use of horizontal and vertical dial measuring devices. In the direct shear box test, Das (1983) reported the following behaviour:

- In both dense and medium sand, shear stress expands with shear displacement to a maximum or peak value and subsequently decreases to a roughly permanent value at large shear displacements. This permanent stress is known as the ultimate shear stress.
- The shear stress for loose sand increases with shear displacement to an ultimate value and then remains constant.
- For both dense and medium sand the volume of the specimen initially decreases and then increases with shear displacement. At larger amonuts of shear displacement, the amount of the specimen remains approximately constant.
- For loose sand the volume of the specimen steadily decreases to a certain degree and remains approximately constant subsequently.

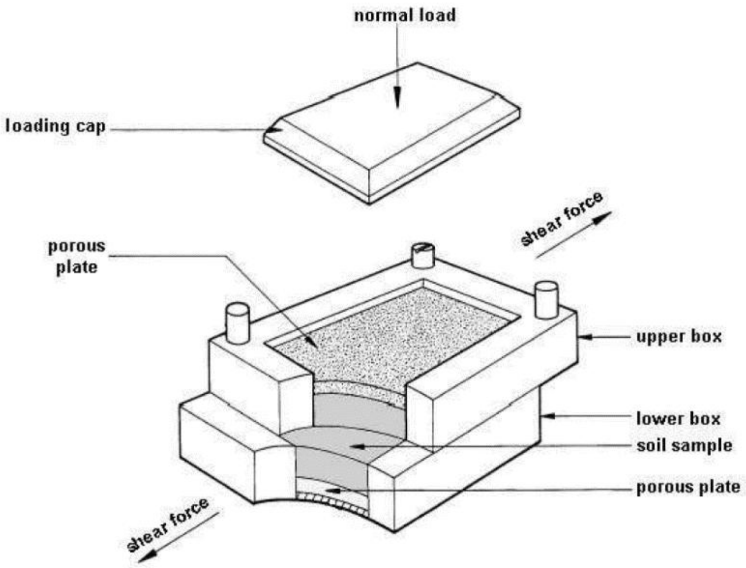

Figure 2.2. Direct shear box test apparatus

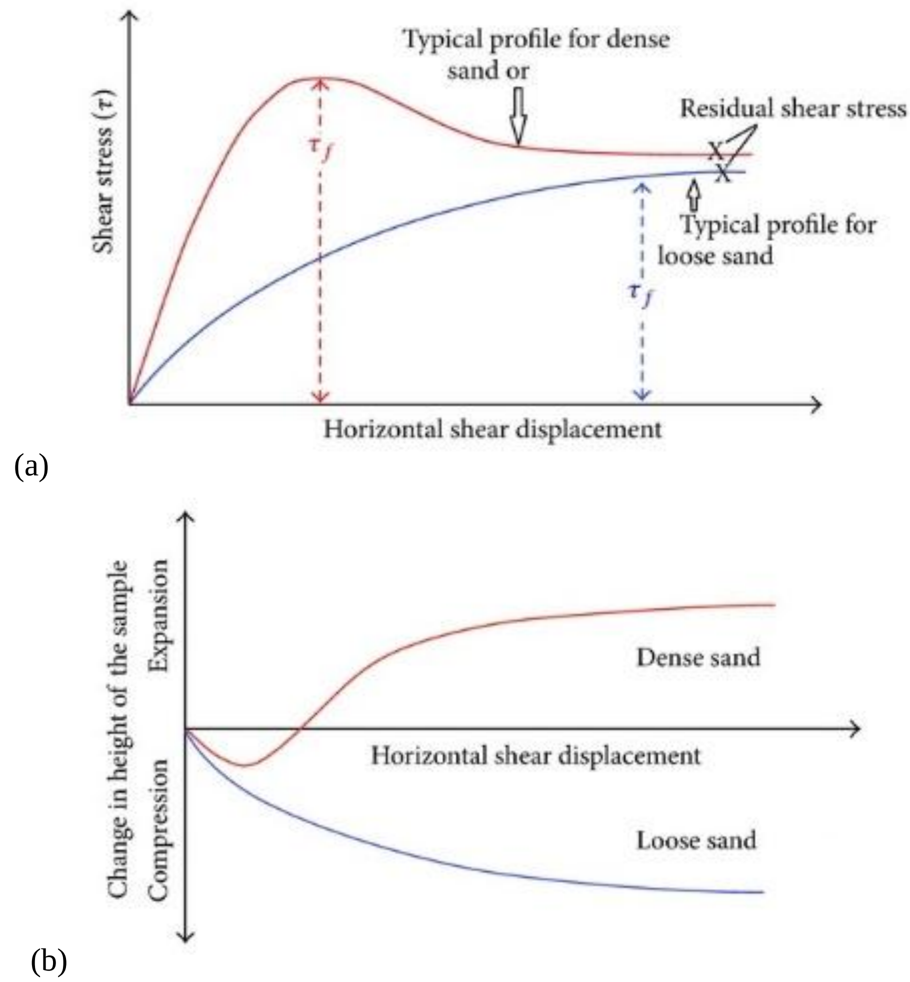

Figure 2.3. Shear Characteristics of Dense and Loose Sand; (a) shear stress, (b) volume change (Capper and Cassie, 1969)

Therefore, if dry sand is utilized for the test, the pore water pressure, u is equal to zero, and thus the total normal stress is equal to the effective stress. In addition, the test may be repeated for a number of normal stresses. It should be noted that the angle of friction of the sand can be gauged by plotting a graph of the maximum or peak shear stresses versus the corresponding normal stresses. However, the Mohr Coulomb failure envelope can be determined by drawing a straight line through the origin and the points representing the experimental outcomes. If the direct shear test is performed on a saturated granular soil, enough time should be allotted between the application of the normal load and the shearing force for soil drainage through the porous stones. In addition, the shearing force should be applied cautiously at a slow rate for complete drainage to occur.

2.8 Dense Sand

The behavior of packing grains in a dense sand with low voids ratio are shown in Figure 2.4. If the sand is sheared along a plane XX, and if it is presumed that distortion and crushing of single grains does not take place, those grains lying just above the surface XX will be forced to ride up and over those lying right below while relative movement takes place. Then, expansion occurs which can be evaluated by observing the upward shift of the top surface of the sand. It is worth mentioning that the resulting raise in volume is called dilatancy.

The shear stress versus displacement curve is indicated in Figure 2.3 (a), and the corresponding volume change relationship with displacement is presented in Figure 2.3 (b) (Capper & Cassie, 1969).

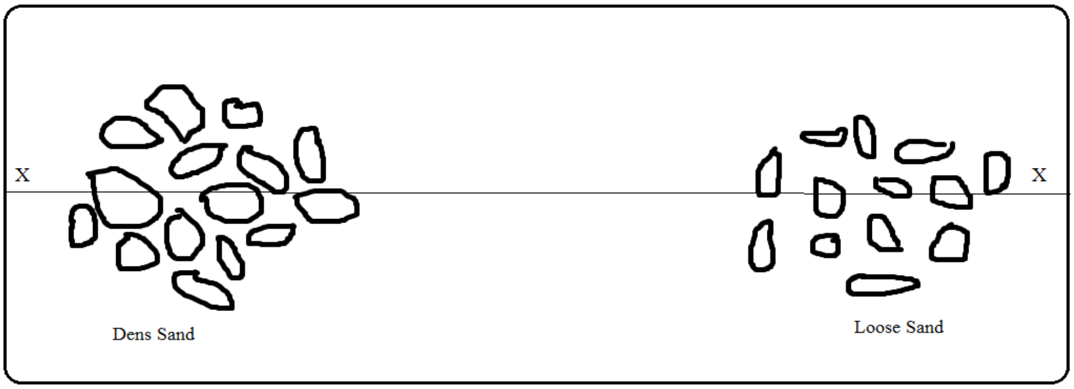

Figure 2.4. Effect of shear on grain structure in sands (Capper & Cassie, 1969)

The trivial initial contraction is as a result of certain bedding down of grains when shearing begins. It is evident that the stress curve rises quite sharply to a peak value and then falls off to a lower value than the peaks. The excess of the peak over the final value, indicates the extra work which needs to be applied to produce the vertical movement due to dilantancy. After the shearing stage, the grains close to the shear surface are in a less dense state of packing than they were at the beginning.

2.9 Loose Sand

A loose condition of packed grains is represented in Figure 2.4 (Capper & Cassie, 1969). If the sand is sheared in conjunction with a plane XX, the grains will be forced to move downwards into void spaces, leading to a collapse of the relatively open structure. According to Capper and Cassie (1969), this may cause a volume of decreases and/or contractions which can be examined as a downward trend of the top surface. Moreover, in free-draining submerged sand this will result in water being ejected from the soil structure (Capper & Cassie, 1969).

In this regard, Figure 2.3 (a) represents the resulting shear stress versus displacement curve which is less steep than the dense curve and does not include a marked peak. Further, the loose curve indicated in Figure 2.3 (b) represents the corresponding volume change relationship with displacement. After the shearing stage, the grains close to the shear surface are in a denser packing state compared to how they were initially.

2.10 Factors that affect the shear strength of sands

According to Holtz and Kovacs (1981), sand is a frictional material and the following factors affect the frictional resistance of sand;

1. Void ratio or relative density,
2. Particle shape,
3. Particle size,
4. Grain size distribution,

5. Particle surface roughness,
6. Pore water
7. Effective stress
8. Particle strength
9. Crushability

It should be noted that void ratio, correlated to the density of sand, is the most important factor which affects the strength of sand.

2.11 Dilatancy

Several studies (Bolton & Simoni, 1986; Hamidi et al., 2009; Houlsby, 2006) calculated the angle of dilation (ψ) by relating the horizontal displacement (h) and vertical displacement (v) to measure the rate of dilation (dev/dh) through the equation below:

$$\tan \psi = \frac{dV}{dh} \qquad (2.3)$$

As several studies (Casagrande, 1936; 1975; Ishihara, 1993; Núñez 1991) reported, critical state is reached as dilatancy disappears, either because of a volume change in drained shear or an effective pressure change in undrained shear. Therefore, equalizing Bolton's dilatancy term to zero will result in, specifically from a theoretical perspective, an implicit correlation between mean pressures as well as the critical state void ratio of sand. Sand dilates with shearing at a rate that rises with confining relative density Dr and goes down with increasing effective confining stress. The peak friction angle of sand will depend on its critical-state friction angle as well as its dilatancy.

In addition, sand exhibits dilatancy which causes a reduction in the decreasing relative density (Dr) and increases the confining stress. Bolton (1986), on the basis of a precise analysis of a large number of plane strains and triaxial tests, proposed an equation to envisage this relationship. Moreover, Bolton (1986) determined the vertical movement of the lid of the shear box. He showed that if the lid moves upward during the test, the volume of the soil increases (dilation); and if the lid moves downward during the test, the volume of the soil decreases (contraction). Hence, the term dilatancy is chosen to explain the increase in volume of dense sand during the shearing process.

Coulomb below illustrates the relationship between the shear strength and resistance to shear of soil and its cohesion as well as the shearing resistance angle (friction angle).

$$\tau = c' + \sigma_n \tan\varphi \qquad (2.4)$$

where,

τ = shear strength

c' = cohesion

σ_n = normal stress

ϕ = angle of shearing resistance

It is evident that the shear strength of non-cohesive soil is influenced only by friction and interlocking of particles, whereas the shear strength in cohesive materials will depend on both cohesion and the internal friction.

2.12 Fibreglass: Recycle Material used for Sand Reinforcement

As Harding and Welsh (1983) claimed, an individual structural glass fibre is both rigid and strong in tension and compression along its axis. Although it might be considered that the fibre is fragile in compression, it is only the length aspect ratio of the fibre which makes it weak since a typical fibre is both long and narrow which warps smoothly. On the other hand, the glass fibre is weak in the shear process along its axis. Hence, as Gordon (1991) concluded, if a collection of fibres can be ordered permanently in a desirable direction within a material, and if they can be prevented from buckling in compression, the material will be ideally strong in that direction.

Further, fibres can be an efficient means of soil reinforcement. The efficiency of fibre reinforcement will be dependent on the deformation characteristics of the host soil as well as the fibre properties. The interaction between fibres and soil occurs at the particle level, yet the reinforced soil is to be used on much larger scales.

Some recent studies (e.g. Maeda & Ibraim, 2008) have benefited from the discrete element method to model the soil and fibres; however, for the practical use of the aforementioned method the behaviour of the soil reinforced with fibres needs to be characterized in terms of parameters for continuum mechanics. However, there have been inconsistent results in the research to date, e.g. Gray and Ohashi (1983) reported that the fibre content has a significant impact on the composite strength up to a certain degree, however, no further effect was noticed. In a similar way, an increase in the fiber length results in a resistance gain of the reinforced material (Gray & Ohashi, 1983; Santoni et al., 2001). In terms of peak resistance, there is a consensus that the inclusion of fibres into the soil reduces the loss in post-peak strength (Consoli et al. 1997; Gray et al., 1986; Ranjan, 1996; 1999; 2003; 2007; Casagrande et al., 2006). This means that it increases the amount of volumetric compression at rupture (Bueno et al., 1996; Stauffer & Holtz, 1996). In addition, some studies showed that the higher the fibre content, the larger the volumetric deformation (e.g. Shewbridge & Sitar, 1989; Nataraj et al., 1996).

Considering direct shear analysis, Gray and Ohashi (1983) reported that the inclusion of discrete fibres for reinforcing cohesionless soil will reduce the loss of post-peak tension. This means that including fibres will cause less conspicuous strain-softening, and prevents any catastrophic failure. Thus, the optimum fibre efficiency of the reinforced soil is heavier than that of the unreinforced soil. Ola (1989) in his direct shear tests found that reinforcing fibre increases the tip intensity level of sand and improves the stress deformation behavior substantially by setting the quantity of peak reduction in shear resistance (Lovisa, Shukla, & Sivakugan, 2010). Interestingly, the direct shear test outcomes suggested that the inclusion of fibre in a sandy land in a dry state may cause an apparent cohesion intercept which will continue to be almost unaltered by an addition of moisture content. The top friction angle was found to be a deciding criterion for the relative density of sand in both reinforced and unreinforced states.

2.13 California Bearing Ratio, CBR

The measurement of the shear strength of a material at a known density is called CBR. As Croney (1977) pointed out, the shear strength of soil can generally be determined in terms of Coulomb's Law. However, according to Rosenal (1963), soil failure occurs as individual grains move relatively to one another which is represented in underlying soil mechanics. As Kin Mak Wai (2006) noted, California bearing ratio (CBR) is one of the moste frequent utilized index tests to evaluate the stiffness modulus as well as the shear strength of the subgrade which is explained in underlying soil mechanics.

Generally, geofibres have been utilized extensively, for soil stabilization and improvement purposes, due to their cost-effective price, light weight, and beneficial contribution to strength gain. In this regard, there have been many research studies on employing geofibres to improve soil quality, especially sandy soil. As several studies (e.g. Arteaga, 1989; Freitag, 1986; Maher & Ho, 1994) revealed, adding geofibre increases the load bearing capacity of sand, and enhances the shear modulus as well as liquefaction resistance.

Early studies revealed that the improvement of sand properties will depend on the type, length, content, and orientation of the geofibre (Arteaga, 1989; Gray & Al-Refeai, 1986). With respect to the proper utilization of geofibre with fine-grained soil, Fletcher and Humphries (1991) investigated the effect of mixing discrete polypropylene geofibres with MH-type silt through California Bearing Ratio (CBR) values and found that adding geofibre enhanced the bearing capacity of the soil by as much as 133% increase in CBR values. Also, Grogan and Johnson (1994) studied the use of geofibre with lime modified clay, cement modified sand, and a silty sand to test its performance under applied traffic load. Road sections with and without geofibre reinforcement were constructed and subjected to truck traffic tests. The results revealed that the inclusion of geofibre causesd up to 90% more traffic passes until failure in the clay, 60% passes until failure in the modified sand, and some improved traffic performance was observed for the silty sand. In another study, Ahlrich and Tidwell (1994) tried to stabilize the plastic clay and a uniform clean sand with the addition of monofilament and fibrillated geofibres. They found that the plastic clay could not be efficiently stabilized by either of the geofibre types investigated although both geofibre types appropriately improved the strength properties of the sand.

Chapter 3

MATERIALS AND METHODS

3.1 Introduction

In this chapter, the site and the method of sampling and all the tests which were performed within this study will be explained and discussed. In this study, all the tests were performed in accordance with ASTM (American Standard of Testing Materials) standards.

3.2 Materials

3.2.1 Palm Beach Sand

The sand used in this study was taken from the coastline of Palm Beach, Gazimağusa in North Cyprus. The disturbed sand sample was collected from 20-40 cm below the surface by using shovels. Then the obtained sample was taken to the Soil Mechanics Laboratory, Civil Engineering Department, Eastern Mediterranean University, and dried in the oven at 105±5°C for 24 hours. In order to represent the actual field conditions in soil testing, in all the tests, unwashed sand samples were used in the study.

3.2.2 Fibreglass

Fibreglass is a type of plastic material which is impregnated together with small glass fibres for reinforcement. It is also referred to as GRP: glass-reinforced plastic material (Mayer, 1993). Fibreglass (or fiberglass) is type of fibre reinforced plastic where the reinforcement fibre will be particularly glass fibre.

Figure 3.1. Site Plan of Palm Beach Coastline (Google Earth, 2015)

Figure 3.2. Fibreglass (Ariyana Pars Co.)

The plastic matrix is typically a thermosetting plastic, most often epoxy, a thermoplastic, polyester resin or vinyl ester. Fibreglass is lightweight, sturdy against compression and tension, and easy to mould into complex shapes (Nawy and Edward, 2001).

In this study, fibre concentration in the reinforced sand is defined as follows:

$$\rho = \frac{V_r}{V} \tag{3.1}$$

where,

ρ = the fibre concentration

V_r = volume of the fibres

V = volume of the sand

The fibreglass used in this study was prepared by Ariana Pars Company in Iran. Thermal conductivity coefficient λ= 0.04 is an important parameter for construction materials. Lower thermal conductivity of insulations turns them to be better materials, so the smaller the λ, the better the insulation. The density of fibreglass is 10 kg/m^3. The dimensions of the fibres that were used in all tests in this study were 0.010 mm in diameter and 20 to 25 mm long. The properties of fibreglass are presented in Table 3.1.

Table 3.1. The properties of the fibreglass (Ariana Pars Co.)

Properties	Values
Thermal conductivity, λ *(w/mK)	0.04
Density of fibreglass, kg/m^3	10
Silica, (%)	44
Sodium carbonate, (%)	18
Dolomite, (%)	11
Feldspar, (%)	12
Borax, (%)	7
Limestone, (%)	4
Barium carbonate, (%)	3
Sodium sulfate, (%)	1

*watts per meter Kelvin

Fibreglass is produced by utilizing less energy compared to other soil stabilization products, as it is formed of recycled materials. Fibreglass is a light weight material, nevertheless features substantial mechanical strength, impact resistance, fire resistance, and durability. It is also a good thermal and electrical insulator.

3.3 Methodology

3.3.1 Particle Size Analysis of Palm Beach Sand

The sieve analysis test is conducted on Palm Beach sand based on ASTM D6911-04 (2009) standard. The test is for the determination of the particle size distribution of sand by shaking oven dried soil through a set of sieves (Figure 3.3) and recording the mass of soil retained on each sieve. The particle size distribution of Palm Beach sand is depicted in Figure 3.4 given below.

Figure 3.3. Sieve Analysis Apparatus

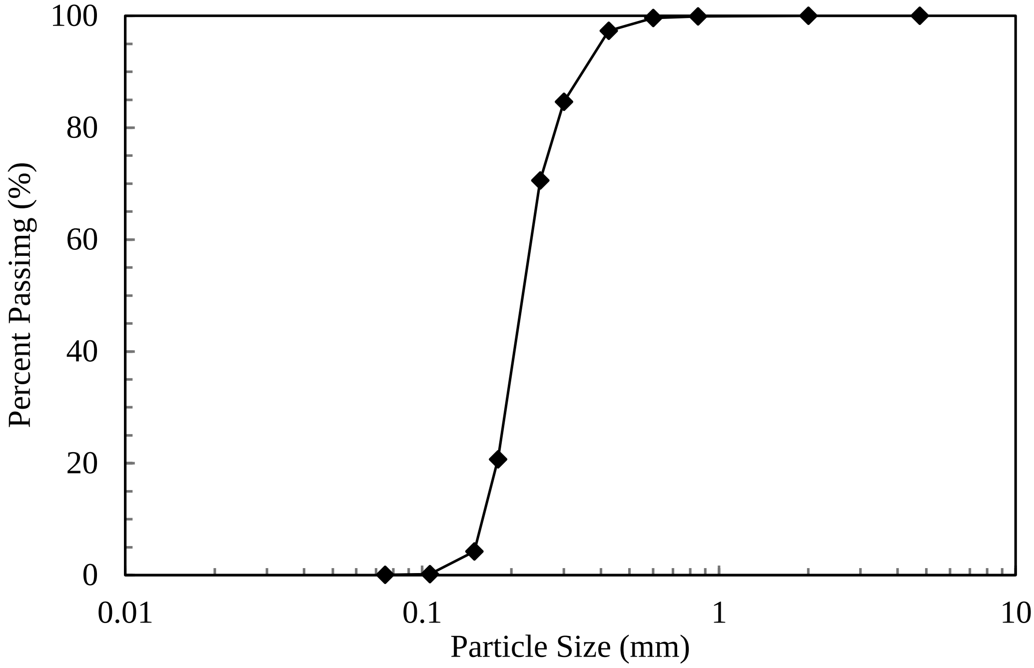

Figure 3.4. The particle size distribution of Palm Beach Sand

The characteristics of Palm Beach sand and some coefficients acquired from the sieve analysis are shown in Table 3.2.

Table 3.2. The physical properties of Palm Beach Sand

Soil Properties	Values
Effective size, D_{10} (mm)	0.17
Coefficient of uniformity, C_u	1.41
Coefficient of curvature, C_c	0.88
Specific gravity, G_s	2.67

According to the Unified Soil Classification System, ASTM D2487 (2011) and the gradation curve obtained for Palm Beach sand shown in Figure 3.4, the sand is classified as poorly graded sand, SP. The optical microscope image of Palm Beach Sand with magnification of 10^x is given in Figure 3.6.

Figure 3.5. Palm Beach sand

Figure 3.7 indicates different shapes of Portway sand. According to the electron micrographs (SEM) of Portway sand (Wang, 2005) the sand particles of Palm Beach are categorized into subrounded and subangular shapes. The geology of Cyprus was greatly influenced by the collision of the Euro-Asian and African tectonic plates, which took place 80 million years ago (Dreghorn, 1978). The coastline in Famagusta region was formed in Pleistocene era. This terrace deposit consists of calcarenites, sands and gravels.

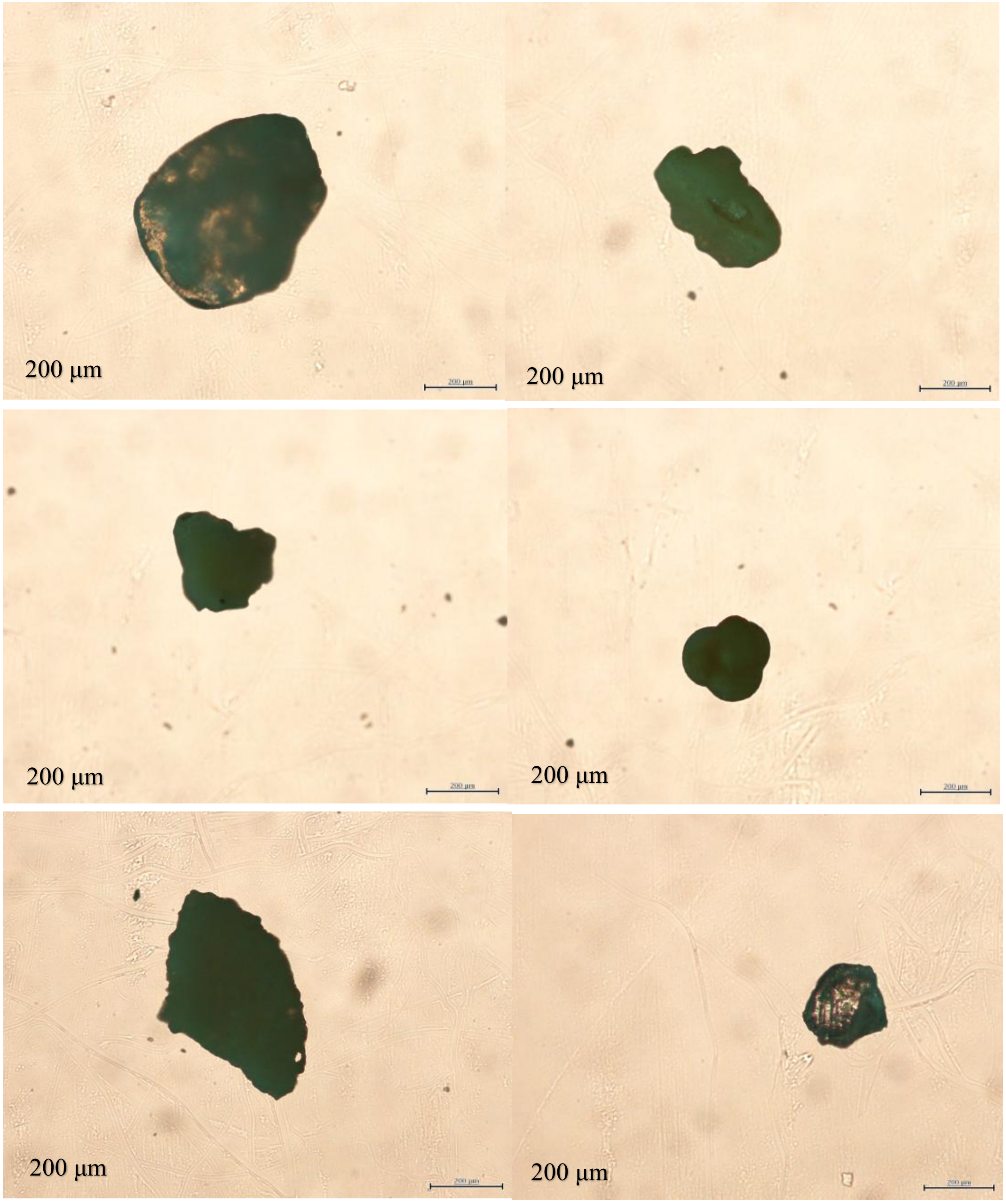

Figure 3.6. Palm Beach sand image with magnification of 10^x

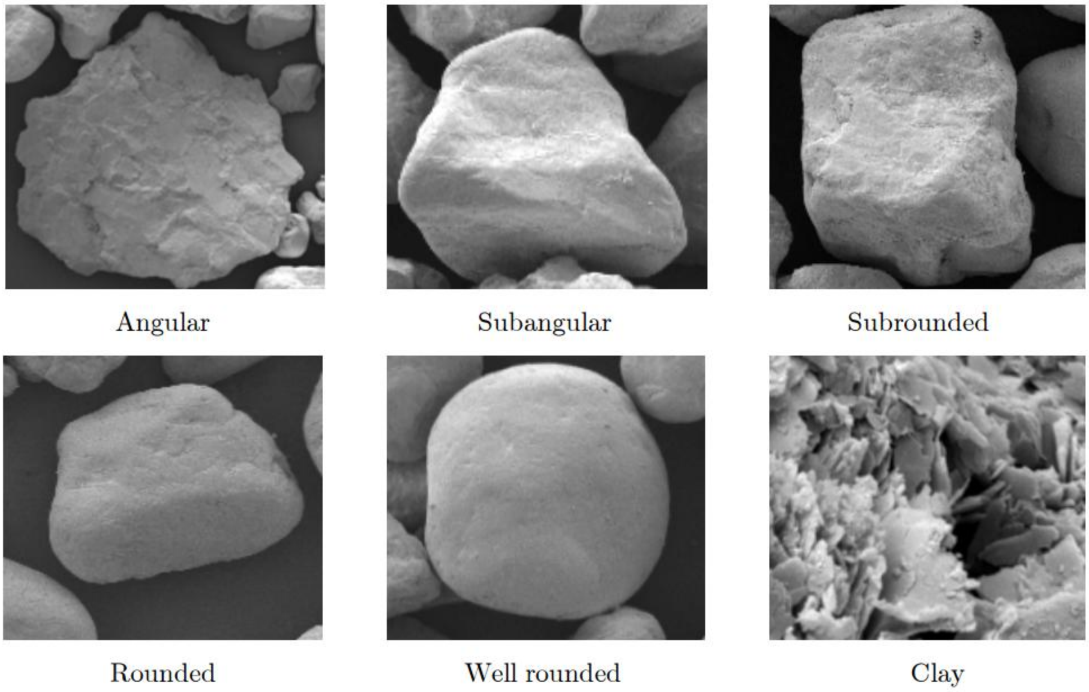

Figure 3.7. Characterization of particle shapes of Portway sand (Wang, 2005)

3.3.2 Relative Density Determination

To determine the state of density of a cohesionless soil with respect to its minimum and maximum densities, the following equation is used:

$$Dr = \frac{e_{max} - e}{e_{max} - e_{min}} \tag{3.2}$$

where,

Dr = relative density expressed as a percentage

e = in-situ void ratio

e_{max} = maximum void ratio (void ratio in loosest state)

e_{min} = minimum void ratio (void ratio in dense state).

3.3.3 Minimum Void Ratio of Palm Beach Sand

In the literature, there are different methods applied for the determination of the state of density of a cohesionless soil with respect to its maximum and minimum densities. Some researchers obtain a control density in the laboratory for the cohesionless material by filling a standard compaction mould in several layers (5 layers), the falling height of the sand about 0.5 in. to 1 in. (ASTM D4254), confining each layer in some manner (between 15-25 rapping), and vibrating the mould by rapping it sharply in the sides with a rubber mallet (Joseph E. Bowles 1992). The largest density value obtained from several trials is then taken as the control criterion for the job. A somewhat better criterion might be obtained by expressing the relative density, Dr of the soil. This has been defined by Terzaghi (1925) as a fraction equation of void ratios of the soil in its loosest state e_{max}, in the natural soil state e, and in the densest possible state e_{min} as given in equation:

$$e_{min} = \frac{\rho_w G_s}{\rho_{dmax}} - 1 \qquad (3.3)$$

where,

e_{min} = minimum void ratio

ρ_w = water density

G_s = specific gravity

ρ_{max} = maximum density

3.3.4 Maximum Void Ratio of Palm Beach Sand

To obtain the maximum void ratio of the sand, a cylindrical compaction mould was used and the sand was carefully poured into the mould, distributing the soil in circular motion over the mould. The mould was slightly overfilled, and then with a straightedge, the excess was struck with as little vibration as possible. The test weight was obtained and repeated for at least 5 times. Equation 3 is used to find the maximum void ratio of thee sand. Figure 3.8 shows the mould and the accessories used in the relative density test.

$$e_{max} = \frac{\rho_w \cdot G_s}{\rho_{dmin}} - 1 \qquad (3.4)$$

where,

e_{max} = maximum void ratio

ρ_w = water density

G_s = specific gravity

ρ_{min} = minimum density

Figure 3.8. Relative Density Test Kits

3.3.5 Standard Proctor Compaction Test

Proctor's (1933), created a solution for determining the maximum density of soils. Ghayttha (1930) found that in a controlled environment (or within a control volume), the soil could be compacted to the point where the air could be completely removed, simulating the effects of a soil in situ conditions. From this, the dry density could be determined by simply measuring the weight of the soil before and after compaction, calculating the moisture content, and furthermore calculating the dry density (Proctor, 1930). The original Proctor test, ASTM D698 (AASHTO T99), uses a 4-inch-diameter (100 mm) mould which holds 1/30 cubic foot of soil, and calls for compaction of three separate lifts of soil using 25 blows by a 2.5 kg (ASTM Standard D1557, 2009). The hammer falls 0.304 m inches, for a compactive effort of 600 kN-m/m^3.

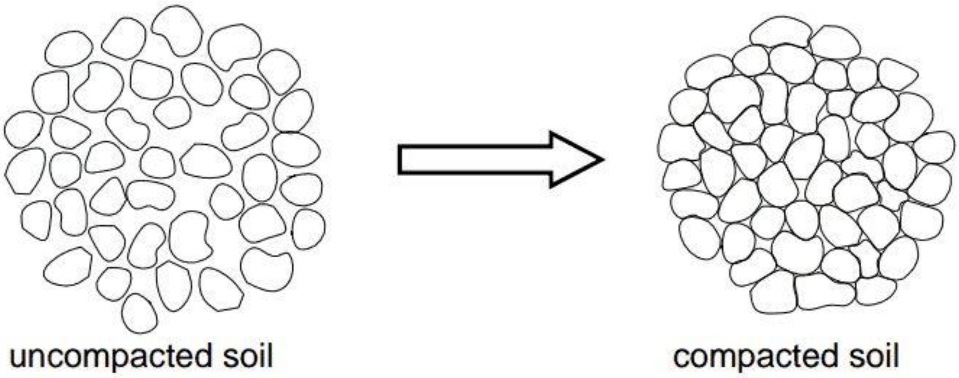

Figure 3.9. Compacted and uncompacted Soils

Compaction is one kind of densification that is realized by rearrangement of soil particles without outflow of water. It is realized by the application of mechanical energy. Figure 3.9 shows the loose and dense soil after compaction. The standard Proctor compaction test on Palm Beach Sand was performed but since the sand was a poorly graded soil, it was difficult to compact the sand and no consistent results were obtained. Figure 3.10 shows the apparatus for the compaction test and the compacted soil in the mold. Figure 3.11 shows the compacted sand using proctor method.

Figure 3.11 presents the results from standard proctor compaction tests. It is observed from the results that, the dry unit weight has a general tendency first to increase as moisture content increases, following by a decrease with further increase in the moisture content.

Figure 3.10. Compaction test apparatus and the compacted soil in the mould

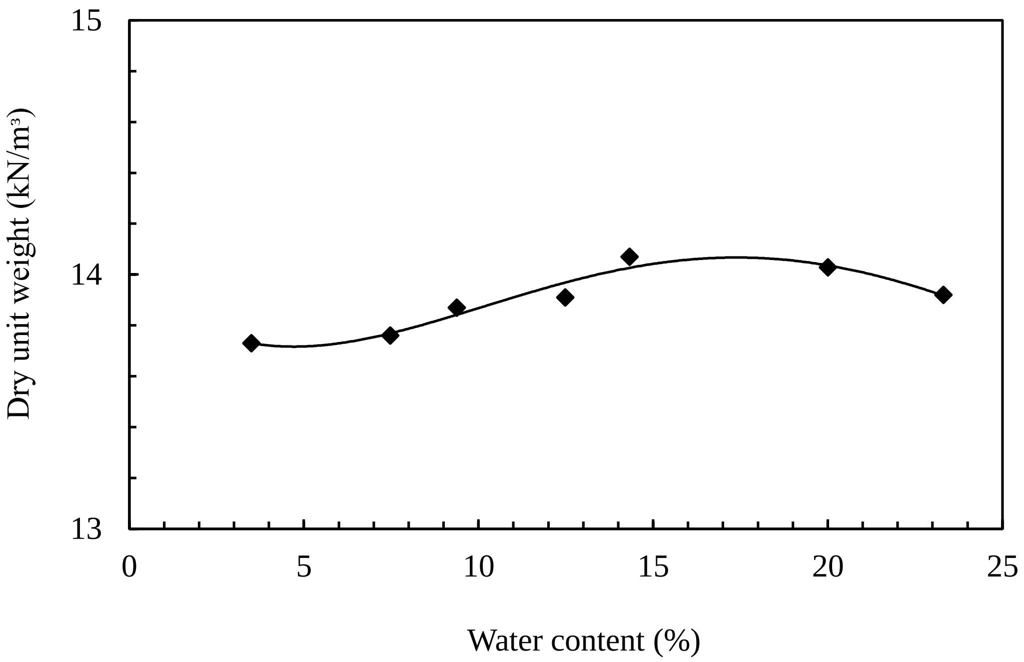

Figure 3.11. Standard Proctor compaction test results for the Palm Beach Sand

3.3.6 Summary of the Results of Density Tests

Table 3.3 presents maximum and minimum void ratio and dry density at various relative density.

Table 3.3. Maximum and minimum void ratio and dry density at various relative density

Density of Palm Beach Sand	Values
Maximum void ratio, e_{max}	0.934
Minimum void ratio, e_{min}	0.733
Void ratio at 30% relative density	0.873
Void ratio at 60% relative density	0.813
Void ratio at 80% relative density	0.773
Maximum dry density ρ_{dmax} (g/cm^3)	1.541
Minimum dry density ρ_{dmin} (g/cm^3)	1.380
Dry density at 30% relative density (g/cm^3)	1.425
Dry density at 60% relative density (g/cm^3)	1.472
Dry density at 80% relative density (g/cm^3)	1.505

3.4 Direct Shear Box Test

The direct shear box test is a conceptually simple test that apparently was used for soil testing as early as 1776 by Coulomb (Lambe & Whitman, 1969) and was featured prominently by French engineer Alexander Collin in 1846 (Skempton, 1984). In the present study laboratory, direct shear tests were performed on pure sand, and sand reinforced with fibres.

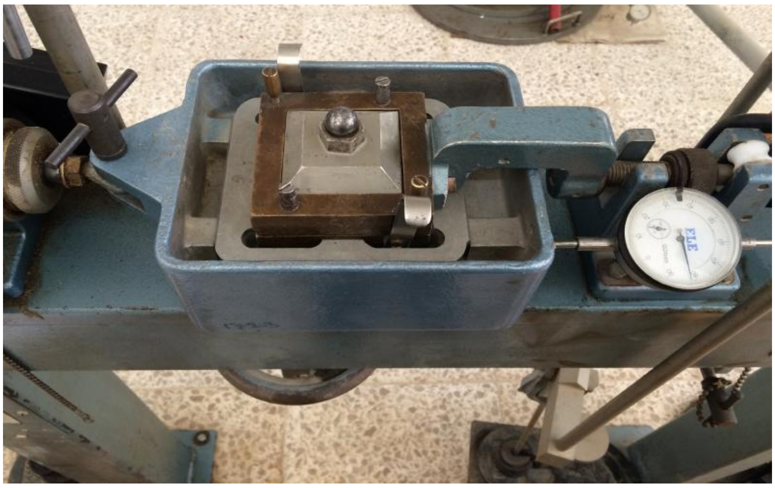

Figure 3.12. Direct Shear Box test apparatus

Figure 3.12 shows the direct shear box apparatus used in the present study. Basically, the test equipment consists of a metal shear box into which the soil specimen is placed. The box is split horizontally into two halves. The normal force on the specimen is applied from the top of the shear box by dead weights. The normal stress on the specimens obtained by the application of dead weights. Shear force is applied to the side of the top half of the box to cause failure in the soil specimen. During the test, the shear displacement of the top half of the box and the change in specimen thickness is recorded by the use of horizontal and vertical dial gauges. The shear box test is the oldest and simplest form of shear test arrangement and the dimension of the box was 6 x 6 x 2.5 cm basically, the testing procedure is very straightforward. The test has been used for measuring the ‘immediate’ or short-term shear strength of soils in terms of total stresses. In the present study, the rate of position (shear deformation) 0.86 mm/min was used in the tests. Figure 3.13 shows the wheels for adjusting shear displacement in the direct shear box test.


Figure 3.13. The wheels in direct shear box test

In the study, tests were conducted on sand and reinforced sand with different percentage of fibreglass content (0, 0.5%, 1% and 1.5% by dry weight of the sand).

Dry sand was used for the tests. The pore water pressure being equal to zero, the total normal stress is equal to the effective stress. In the present study, the direct shear box tests were performed on completely dry sand. Direct shear box test was repeated for three different normal stresses values. In the present study, the test was repeated under the normal stress values of 20 kN/m^2, 30 kN/m^2 and 50 kN/m^2. The angle of friction of the sand can be determined by plotting a graph of the maximum or peak shear stresses versus the corresponding normal stresses. The Mohr Coulomb failure envelope can be determined by drawing a straight line through the points representing the experimental results.

Figure 3.14. Sample preparation for Direct Shear Box test

Three fibre contents 0.5%, 1%, and 1.5% by dry weight of sand were used and different fibre length which were 15 mm to 25 mm, along with three different relative densities 30%, 60% and 80% of the sand. Fibreglass has been cut to the desired length by using scissors. For preparing the test specimens, first the required small amounts of sand and fibres were mixed together in a dry state. To prevent the segregation of fibres before mixing, a small amount of alcohol was added into the fibres. The sand and fibres are hand mixed to prepare a homogeneous sample. In medium dense and dense state, the sample is poured into the box in 5 layers and each layer compacted 20 times using a rubber bung. The compacted specimen dimensions were, 25 mm in height and 60 x 60 mm in plan area. Figure 3.14 shows the direct shear box and the sample preparation in direct shear box test and Figure 3.15 shows the compacted sand specimens.

(a)

(b)

(c)

Figure 3.15. Samples with (a) 0.5%, (b) 1% and (c) 1.5% of fibreglass

3.5 California Bearing Ratio Test (CBR)

In this part of the study, tests were conducted in EMU Soil Mechanics laboratory for determining the effect of fibreglass on the California Bearing Ratio, CBR test of the Palm Beach sand. The CBR tests were performed for reinforced unreinforced sand and the sand specimens were prepared at three different relative densities, Dr: in loose state (Dr = 30%), medium state (Dr = 60%) and dense state (Dr = 80%). In fibreglass reinforced sand due to insufficient material, CBR test performed only with 0.5% fibreglass by the dry weight of sand. In the CBR test, the sand particles were smaller than 19mm were used in the test.

The specimens were compacted according the ASTM D698 by 56 bowls per layers with rammer (24.47 N). In CBR test, the surcharged weight applied on the sand was 4.54 kg with the metal penetration piston (49.63 ± 0.13 mm) diameter and minimum length (101.6 mm). California bearing ratio is defined as the ratio of force per unit area required to penetrate in to a soil mass with a circular plunger of 50 mm diameter. In the test, the rate of penetration should be 1.27 mm /min. Figure 3.16 shows the CBR test apparatus.

During the test, according to ASTM D698, the load readings at various penetrations of 0.64 mm, 1.27 mm, 1.91 mm, 2.54 mm, 3.18 mm, 3.81 mm, 4.45 mm, 5.08 mm, 7.62 mm, 10.16 mm and 12.70 mm were recorded.

For determining the CBR number:

For stress penetration curve: 2.54 mm and 5.08 mm, the following equations were used:

For stress penetration curve: 2.54 mm.

$$\text{CBR Number} = \frac{\text{Stress on piston}}{6.9\ \text{MPa}} \times 100 \tag{3.5}$$

For stress penetration curve: 5.08 mm

$$\text{CBR Number} = \frac{\text{Stress on piston}}{10.3\ \text{MPa}} \times 100 \tag{3.6}$$

Figure 3.16. CBR test apparatus

Chapter 4

RESULT AND DISCUSSION

4.1 Introduction

In this chapter, the results of the test, which have been explained in the previous chapter, will be discussed. The aim of these tests is to investigate the effect of fibreglass on shear strength parameters of Palm Beach sand and to evaluate the effect of fibre reinforcement on the dilatancy behaviour of Palm Beach sand. The primary purpose of reinforcing soil is to increase the shear strength and reduce the compressibility and lateral deformation.

4.2 Direct Shear Box Tests (DSBT)

In this part of the study, direct shear tests were performed under low and high values of normal loads, in order to determine the its effect on shear resistance and displacement, as well as shear strength properties.

As mentioned in Chapter 3, the direct shear tests were conducted at different states of packing, such as loose, medium and dense state. For each state, the specimens were prepared under 3 normal stress values: 20 kN/m^2, 30 kN/m^2 and 50 kN/m^2. The shear stress versus horizontal deformation diagrams will be given and discussed in this part and they will be compared with other diagrams in their categories. All the specimens in direct shear tests were prepared at dry condition.

4.2.1 Direct Shear Test on Natural Sand (Unreinforced)

The shear stress–horizontal displacement curves obtained from the natural (unreinforced) sand with 30% (loose state), 60% (medium state) and 80% (dense state) relative densities and at three different normal stresses values of 20 kPa, 30 kPa and 50 kPa are shown in Figures. 4.2 to 4.4.

Figure 4.1. Sample preparation (natural sand)

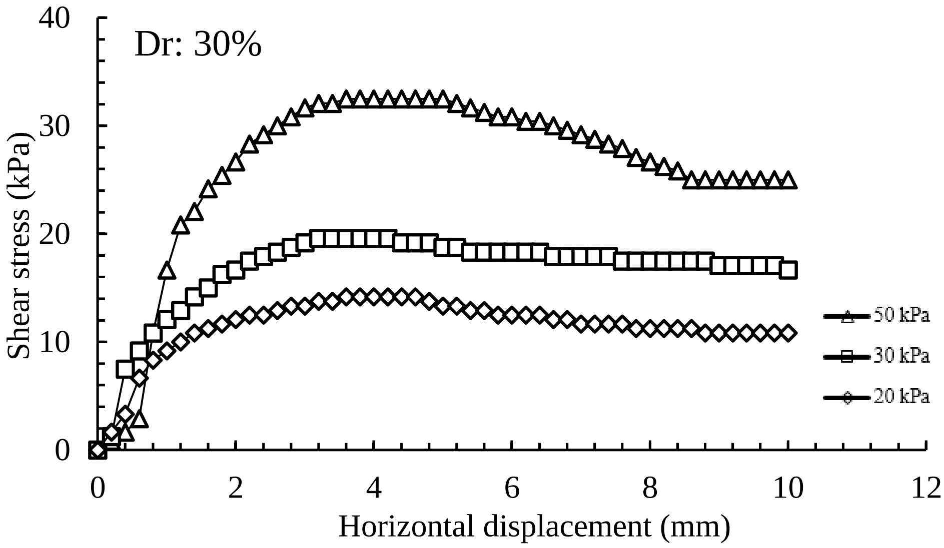

Figure 4.2. Shear stress- horizontal displacement graph for natural sand at 30% relative density

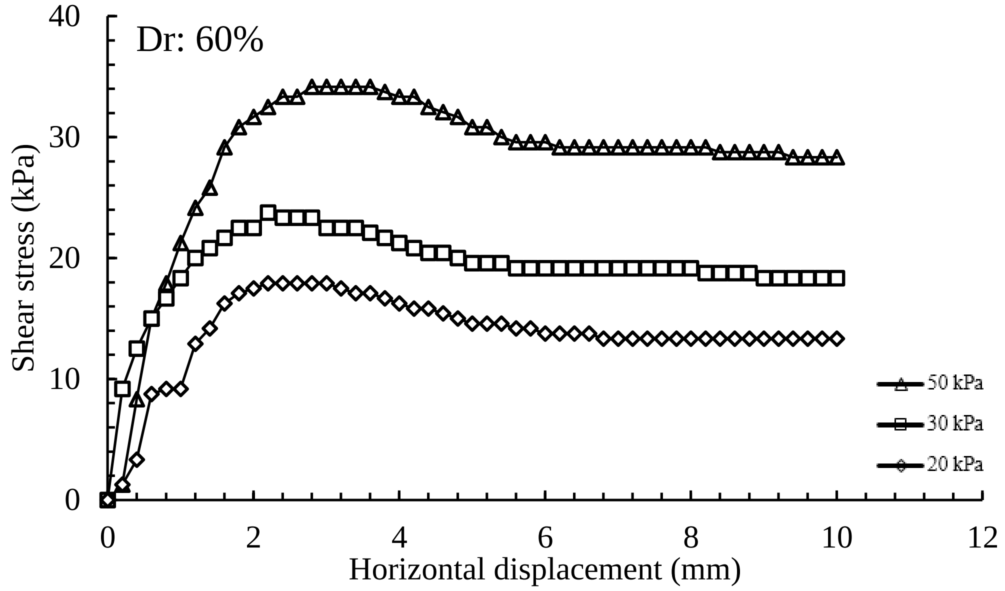

Figure 4.3. Shear stress- horizontal displacement graph for natural sand at 60% relative density

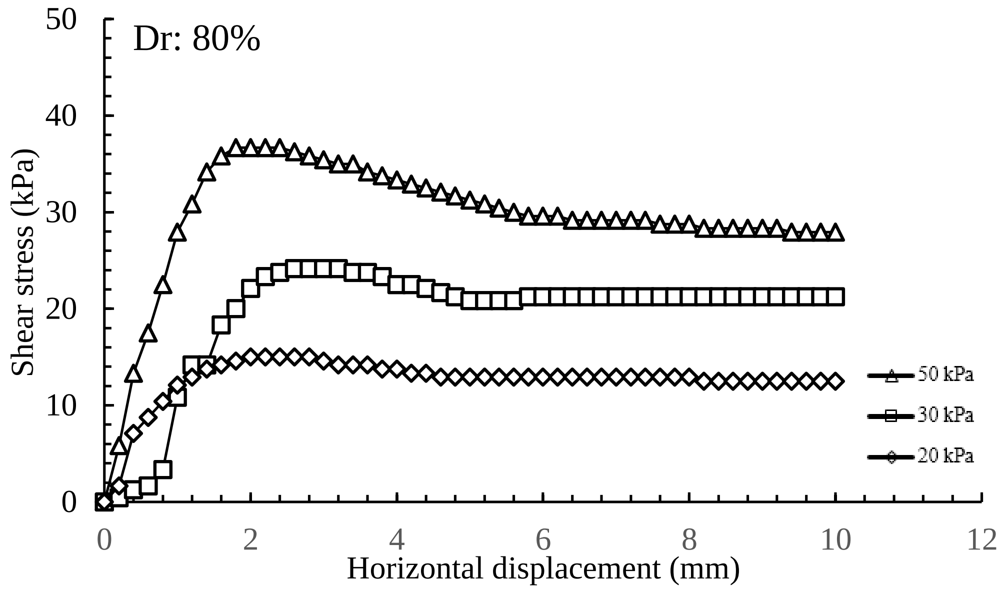

Figure 4.4. Shear stress - horizontal displacement graph for natural sand at 80% relative density

The curves indicate that as the normal stress increases, the slope of the initial loading curve increases. The shear stress of sand under higher normal stress values also increases with increasing relative density.

4.2.2 Direct Shear Test on Fibreglass Reinforced Sand

As aforementioned in Chapter 3, the samples were prepared in the shear box apparatus and tested at various relative densities. The specimens were, loaded at a strain rate of 0.86 mm/minute. Figure 4.6 to 4.8, 4.10 to 12 and 4.14 to 4.16 show the shear stress versus horizontal displacement plots of the reinforced sand prepared at various relative densities. The results show that the shear strength parameters vary with increasing fibre content and relative density.

4.2.2.1 Sand Reinforced with 0.5% Fiberglass

Figures 4.6 to 4.8 indicate the shear stress versus horizontal displacement graphs for the soils mixed with 0.5% fibreglass at three various relative density values.

Figure 4.5. Sand reinforced with 0.5% fibreglass

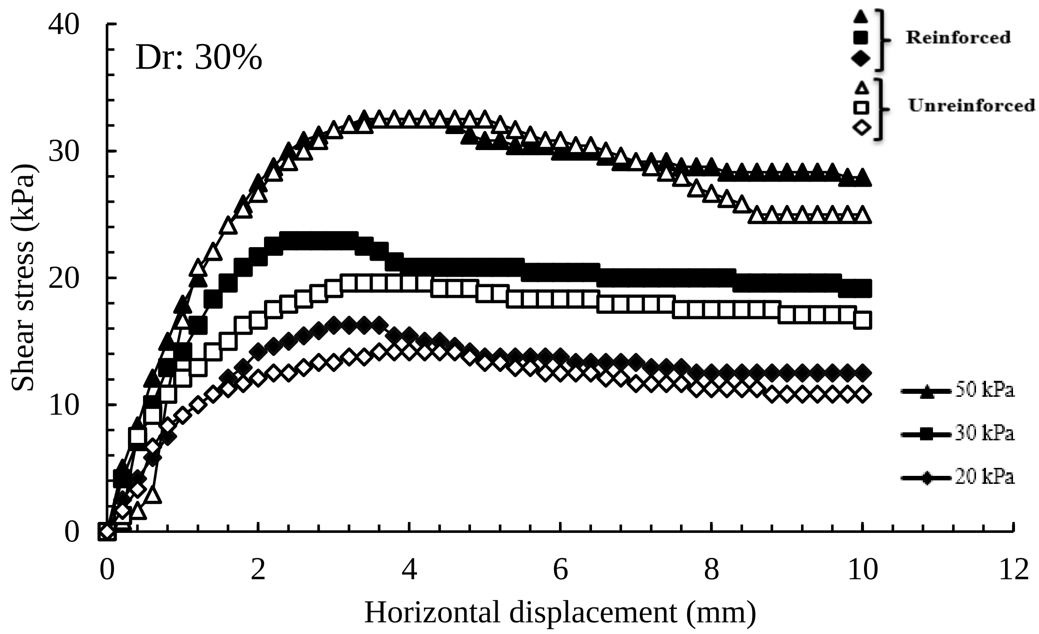

Figure 4.6. Shear stress - horizontal displacement graph for natural sand and sand with 0.5% fibreglass at 30% relative density

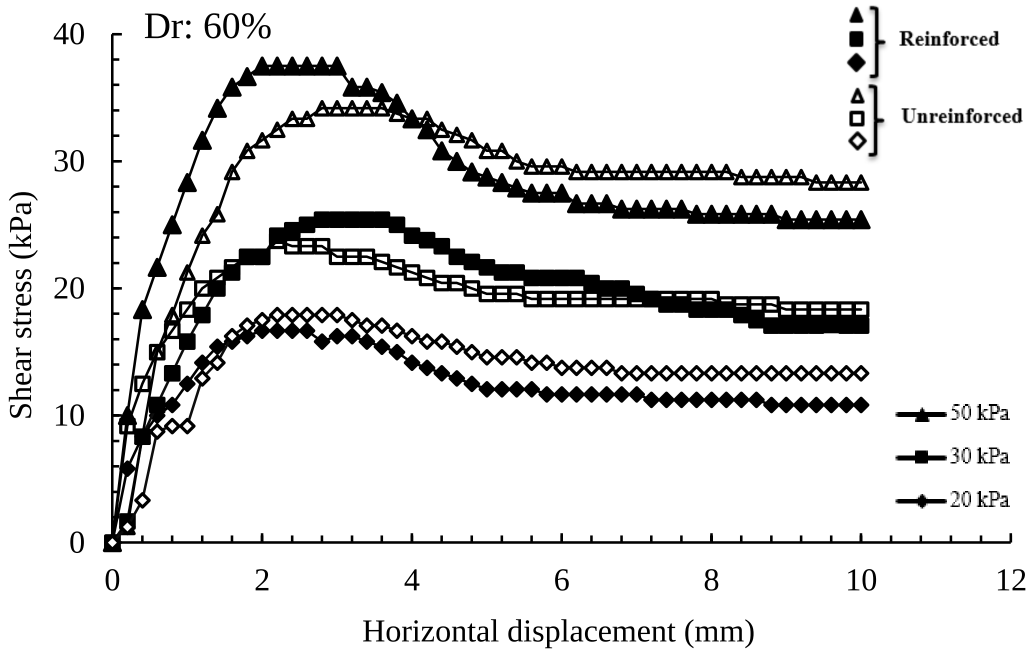

Figure 4.7. Shear stress - horizontal displacement graph for natural sand and sand with 0.5% fibreglass at 60% relative density

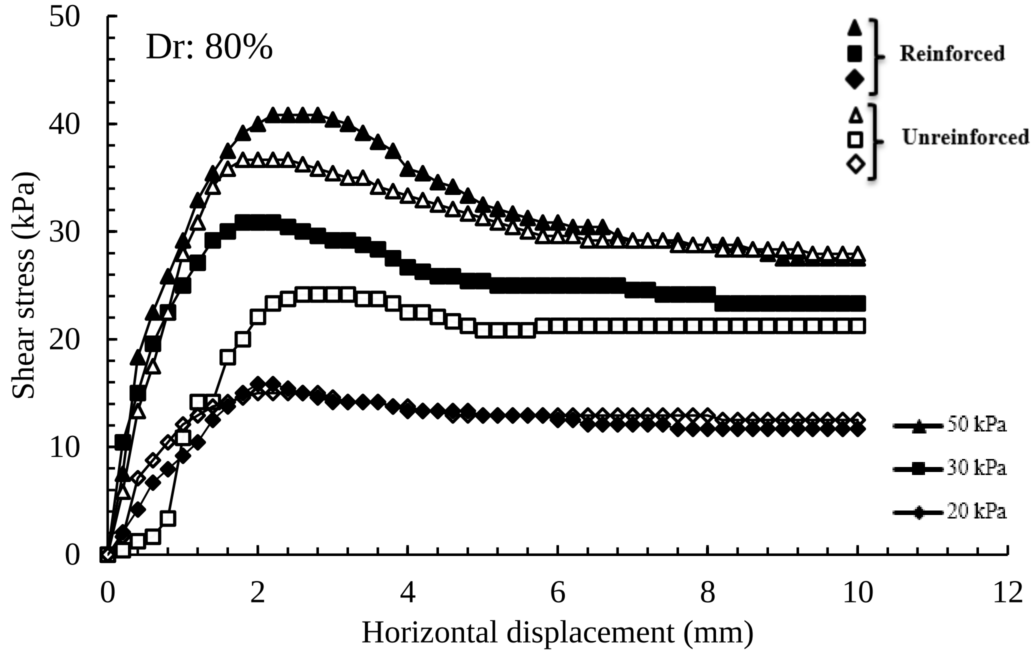

Figure 4.8. Shear stress - horizontal displacement graph for natural sand and sand with 0.5% fibreglass at 80% relative density

In all figures, it can be seen that the shear stress for reinforced sand increase with increasing normal stress. The shear strength is observed to increase with increasing relative density and normal stress. Due to the arrangement of particles in a denser packing, further increase in strength of reinforced sand was obtained.

The tests results also show that with increase in relative density, the shear strength of the reinforced sand have increased which is attributed to more dense packing and higher sand fibre interface friction.

4.2.2.2 Sand Reinforced with 1% Fiberglass

Figure 4.9 shows the sand reinforced with 1% fibreglass. Figure 4.10 to 4.12 shows the results of tests for sand reinforced with 1% fibreglass.

Figure 4.9. Sand reinforced with 1% fibreglass

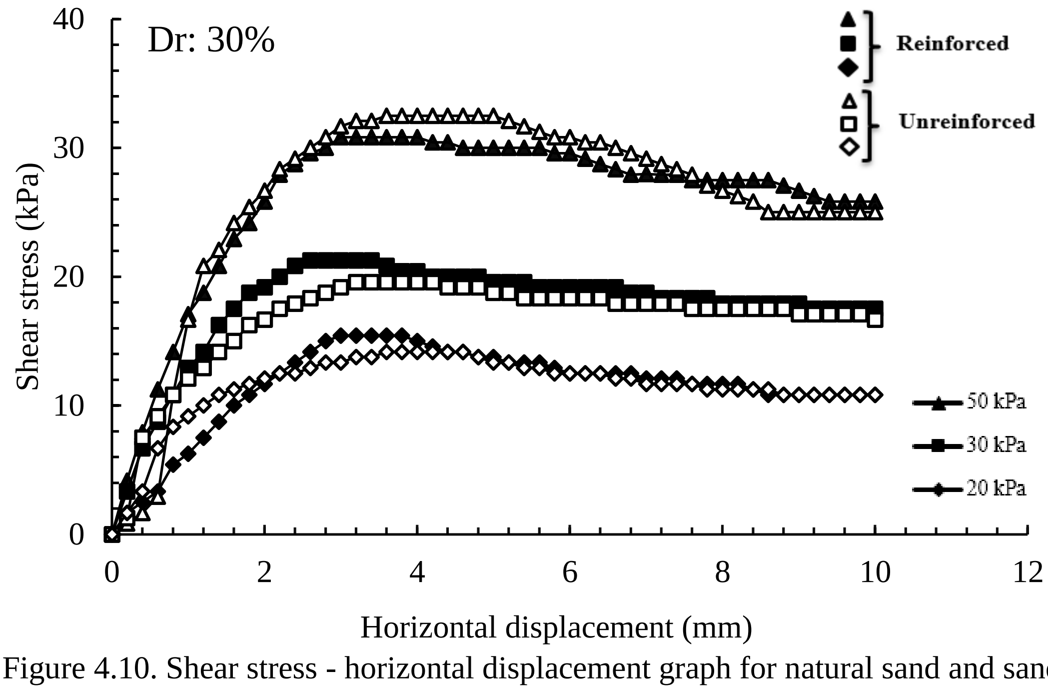

Figure 4.10. Shear stress - horizontal displacement graph for natural sand and sand with 1% fibreglass at 30% relative density

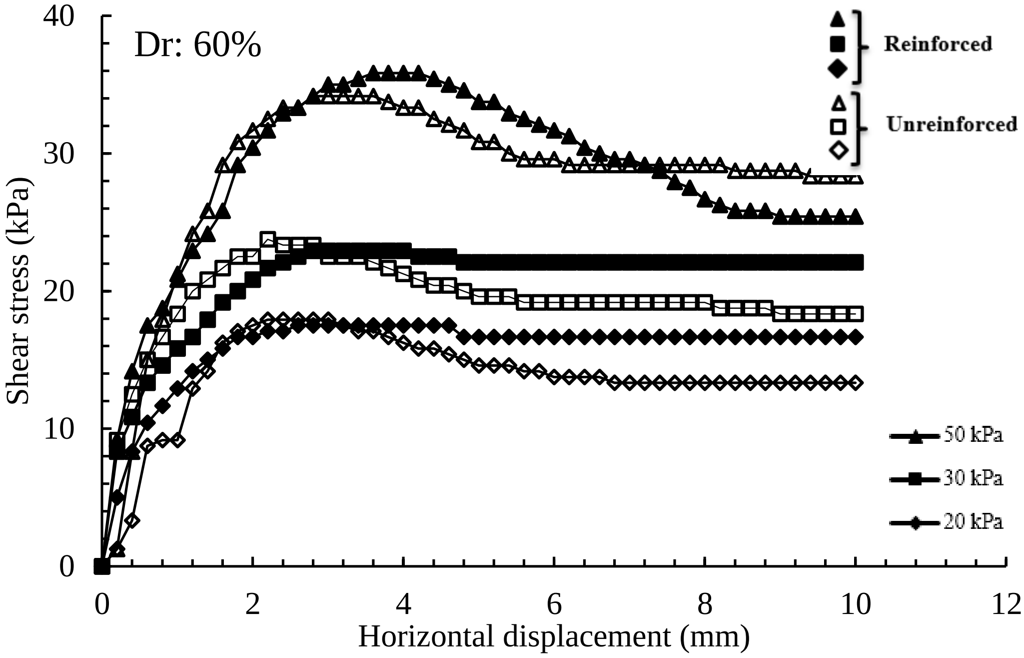

Figure 4.11. Shear stress - horizontal displacement graph for natural sand and sand with 1% fibreglass at 60% relative density

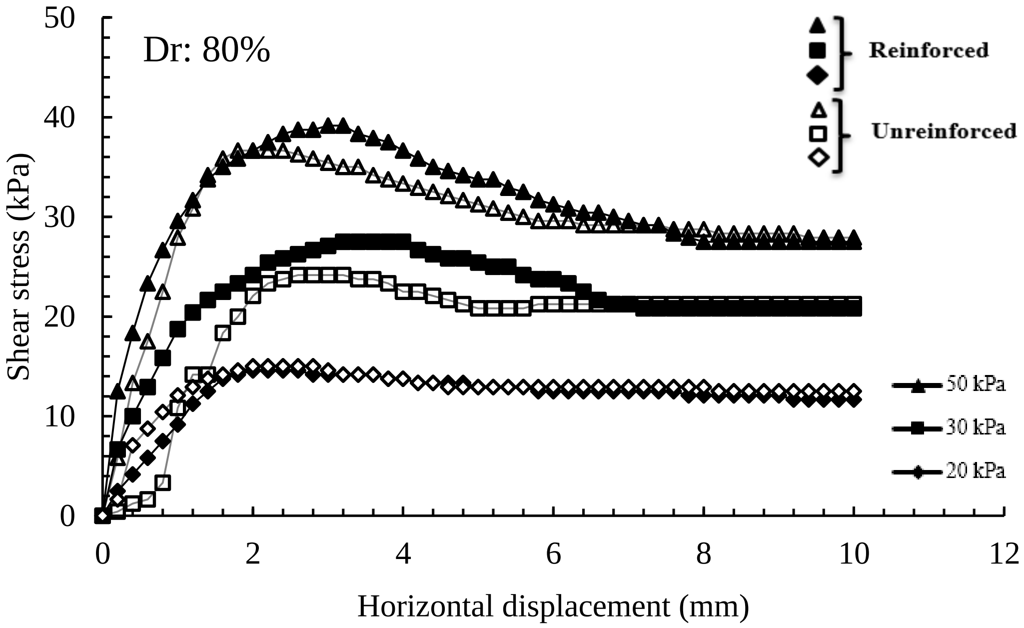

Figure 4.12. Shear stress - horizontal displacement graph for natural sand and sand with 1% fibreglass at 80% relative density

The general form of the shear stress versus horizontal displacement curves of fibre reinforced specimens is similar to that of unreinforced specimens. The figures indicate that fibre reinforcement did not change the shear strength behaviour of sand significantly apart from the increase in the slope of the shear stress versus horizontal displacement curve at small strains.

4.2.2.3 Sand Reinforced with 1.5% Fibreglass

Figure 4.13 shows the Palm Beach sand mixed with 1.5% fibreglass. Figures 4.14 to 4.16 indicate the shear stress versus shear displacement graphs for soils mixed with 1.5% fibreglass at three different relative density values.

Figure 4.13. Sand reinforced with 1.5% fibreglass

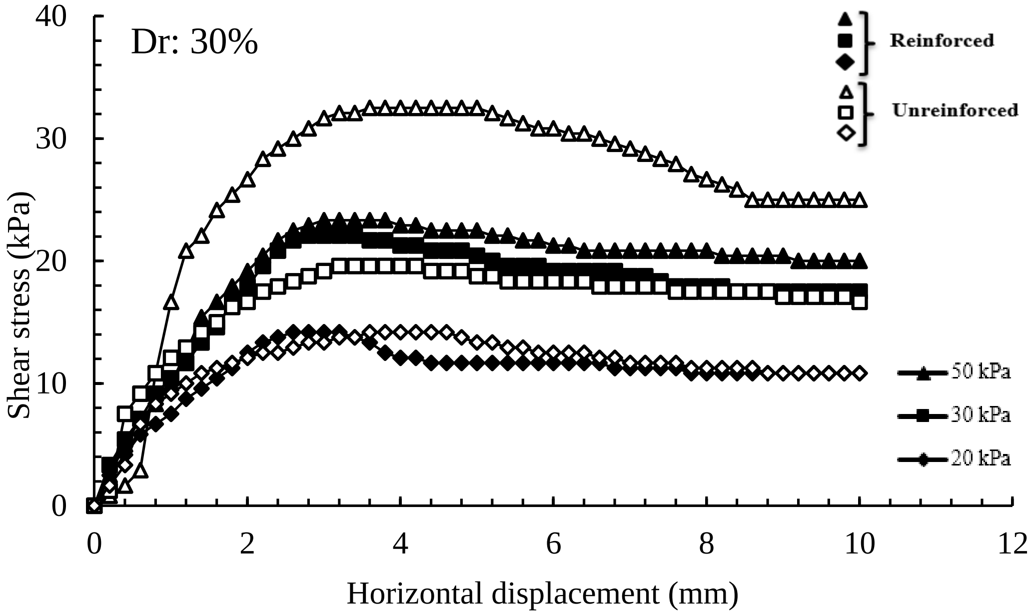

Figure 4.14. Shear stress – horizontal displacement graph for natural sand and sand with 1.5% fibreglass at 30% relative density

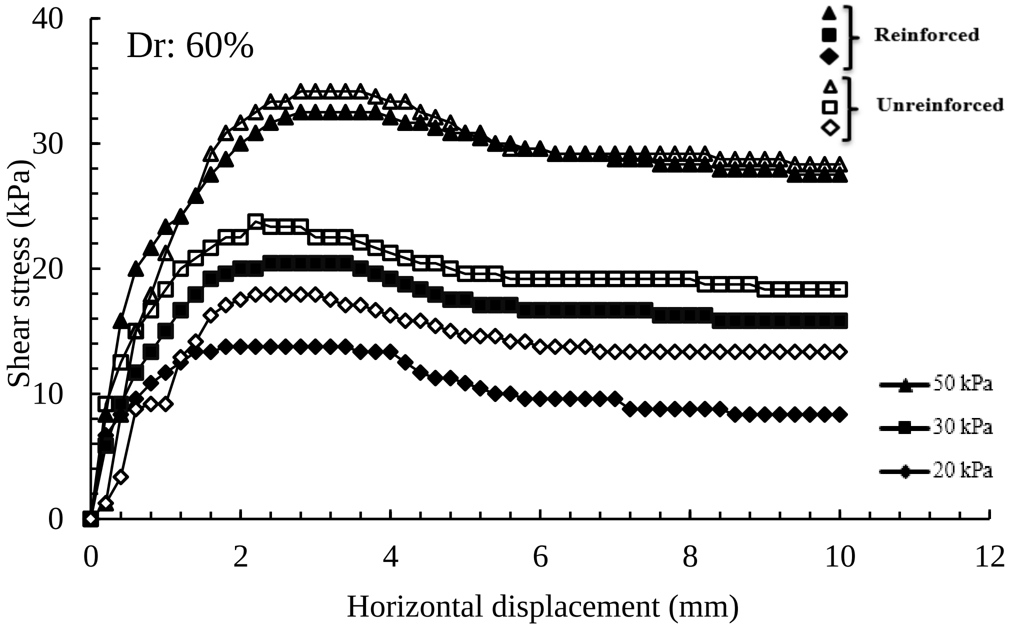

Figure 4.15. Shear stress- horizontal displacement graph for natural sand and sand with 1.5% fibreglass at 60% relative density

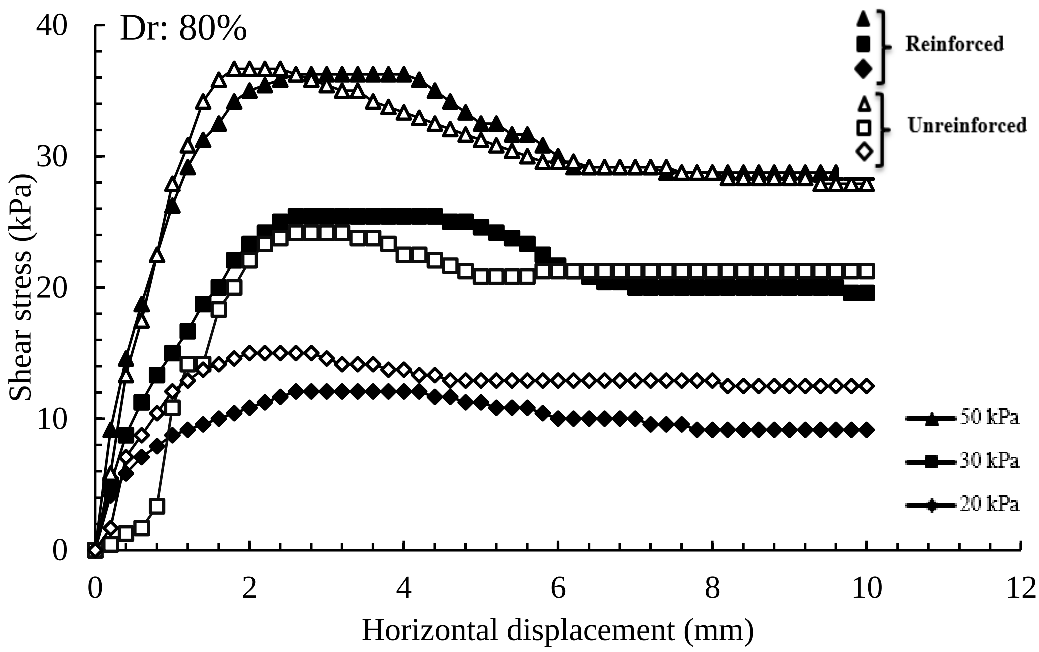

Figure 4.16. Shear stress - horizontal displacement graph for natural sand and sand with 1.5% fibreglass at 80% relative density

According to the results of these tests, it is obvious that increasing the fibreglass more than 0.5% by dry weight of sand does not improve the shear strength of sand. The results indicate that under the same normal stress values, the shear stress at failure decreases with increasing fibre content and this reduction becomes less with increasing relative density. The results indicate that as the fibre content increases, the contribution from sand- fibreglass interface friction to shear strength becomes inefficient.

Increasing the percentage of fibre from 0.5% to 1.5% did not improved the overall shear stress versus horizontal displacement behaviour; there is a decrease in the peak strength with increasing reinforcement. Similar behaviour is observed at all three normal stress levels.

Since the horizontal displacements at failure were small for both natural and reinforced sands (ΔL< 4mm at failure), the strains developed in the fibre reinforcements were likely to be very small as well. Most likely, due to the limited strain mobilisation in fibres at peak shear stress and at failure shear stress, the overall displacement was not significant, because of the additional fibreglass into the sand decreases the interlocking between the sand particles and that caused the reduction in peak shear strength and residual parameters.

4.2.3 Shear Stress at different relative densities under 50 kPa

Figures 4.17, 4.18, 4.19 and 4.20 indicate typical graphs of direct shear tests on sand mixtures reinforced with various fibreglass percentages (0%, 0.5%, 1% and 1.5%) at relative densities of 30, 60 and 80 percent under normal pressure of 50 kN/m^2.

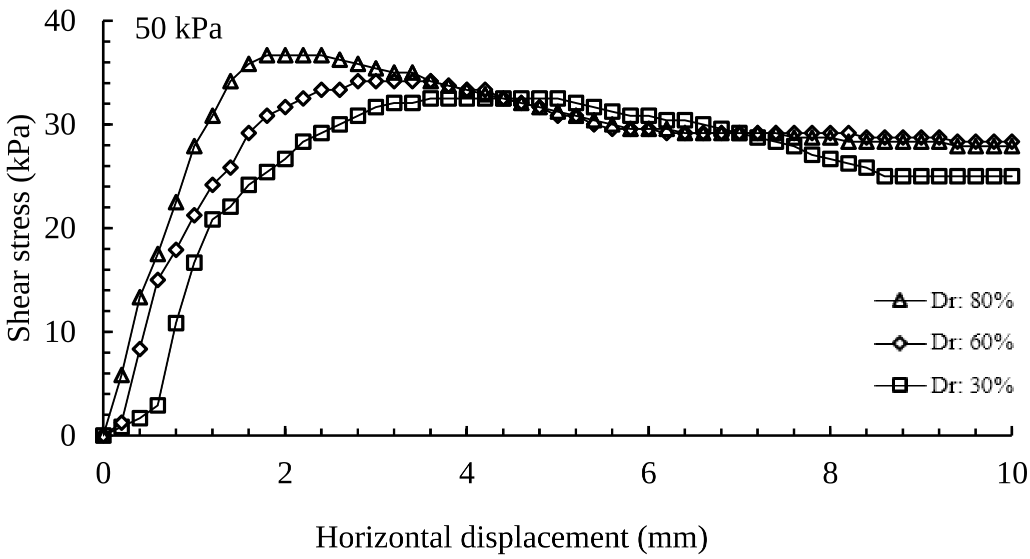

Figure 4.17. Shear stress- horizontal displacement in different density under 50 kPa (natural sand)

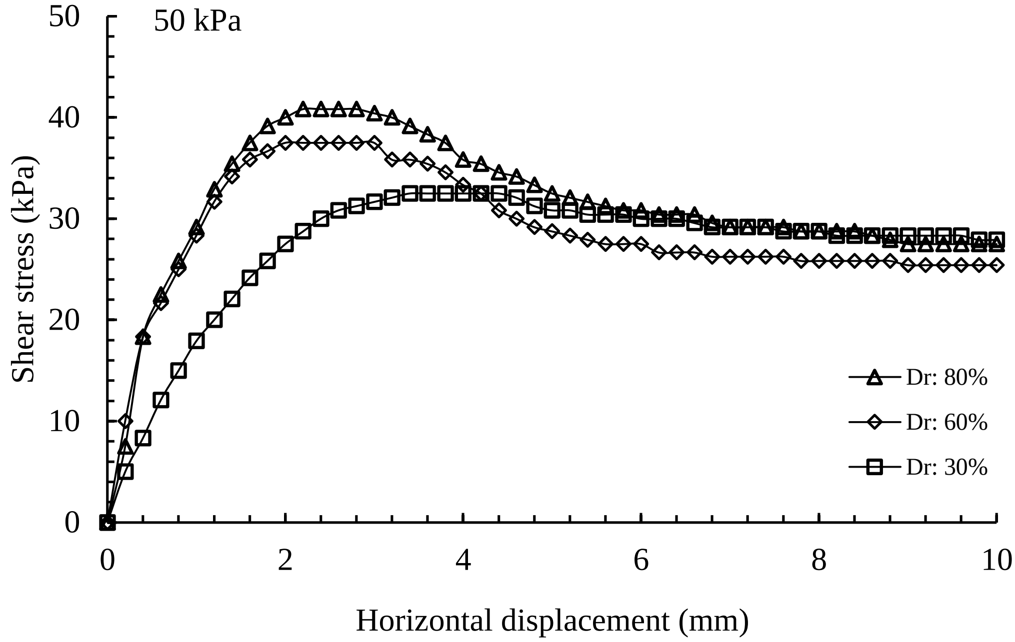

Figure 4.18. Shear stress- horizontal displacement in different density under 50 kPa with 0.5% fibreglass

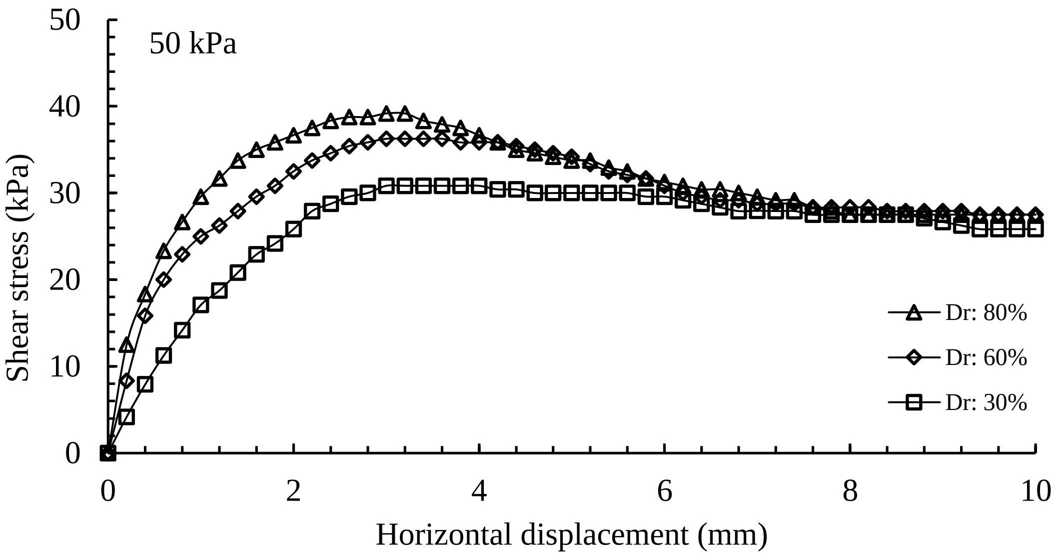

Figure 4.19. Shear stress- horizontal displacement in different density under 50 kPa with 1% fibreglass

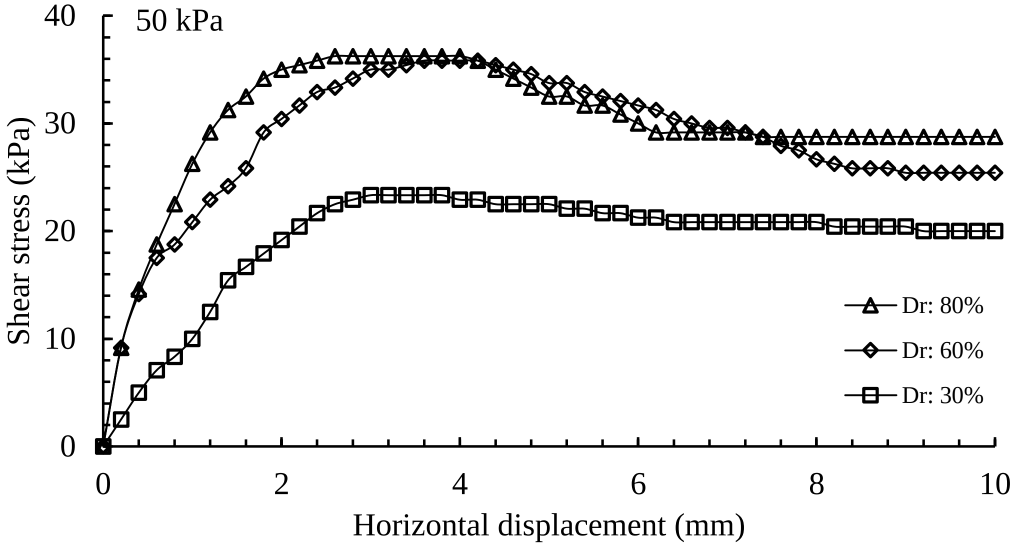

Figure 4.20. Shear stress- horizontal displacement in different density under 50 kPa with 1.5% fibreglass

The results show that, the shear strength increases with an increase in relative density. The graphs show the shear stress increase with increasing normal stress values. The results also indicate that, under the same normal stress values, the peak shear stress decreases with increase in fibre content.

4.2.4 Vertical displacement versus horizontal displacement of sand with different percentage of fibreglass

During the direct shear testing, for the natural and the fibre reinforced sand, the shear stress, horizontal displacement and vertical displacement values were recorded under different relative density and normal stress values. The aim of this part of the study was to show the relationship between the horizontal and vertical displacement. Respective direct shear tests results for various relative densities with three percentages fibreglass are shown in the following, Figure 4.21 to Figure 4.29;

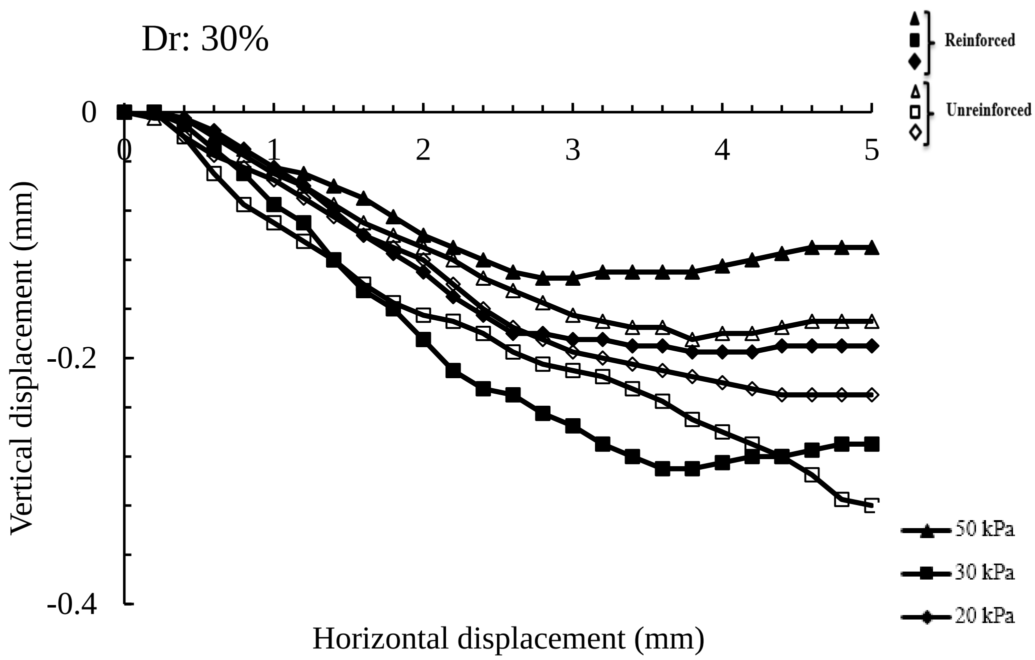

Figure 4.21. Vertical displacement - horizontal displacement graph for natural sand and sand with 0.5% fibreglass at 30% relative density

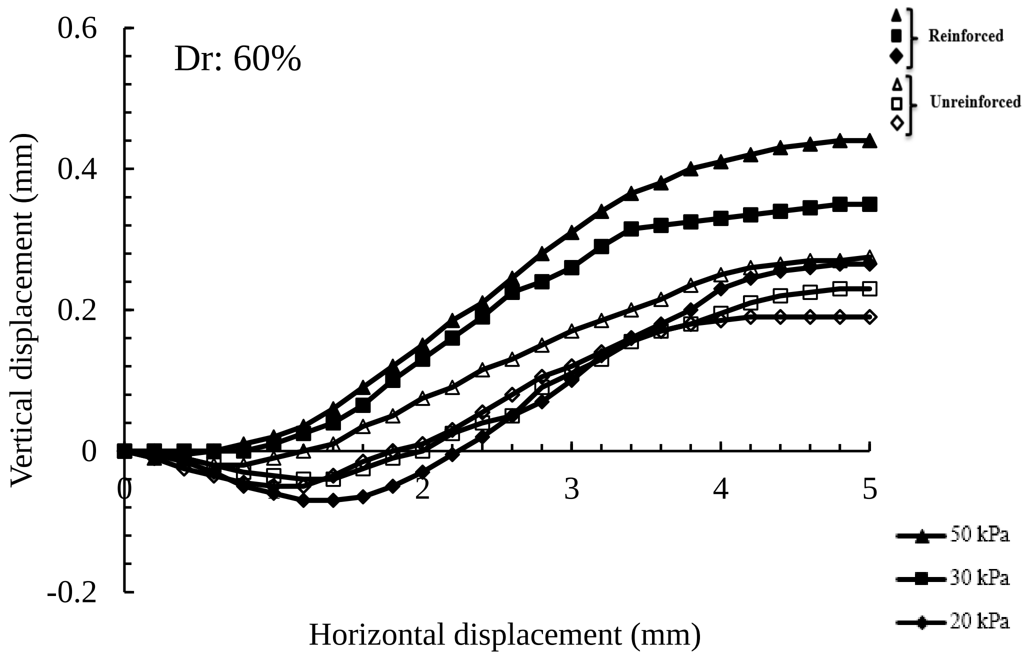

Figure 4.22. Vertical displacement - horizontal displacement graph for natural sand and sand with 0.5% fibreglass at 60% relative density

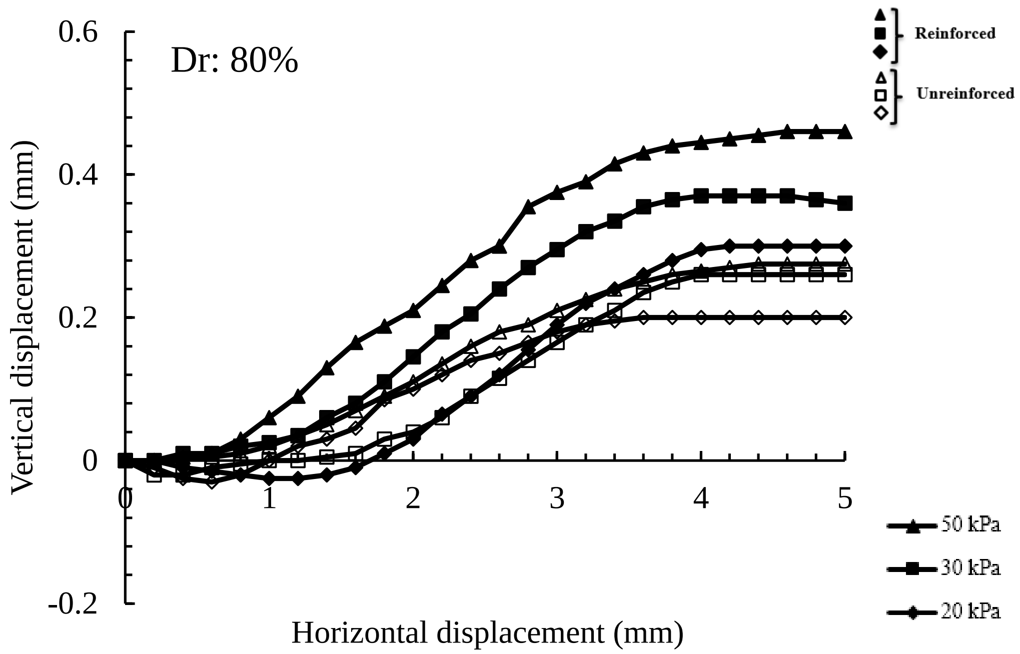

Figure 4.23. Vertical displacement - horizontal displacement graph for natural sand and sand with 0.5% fibreglass 80% relative density

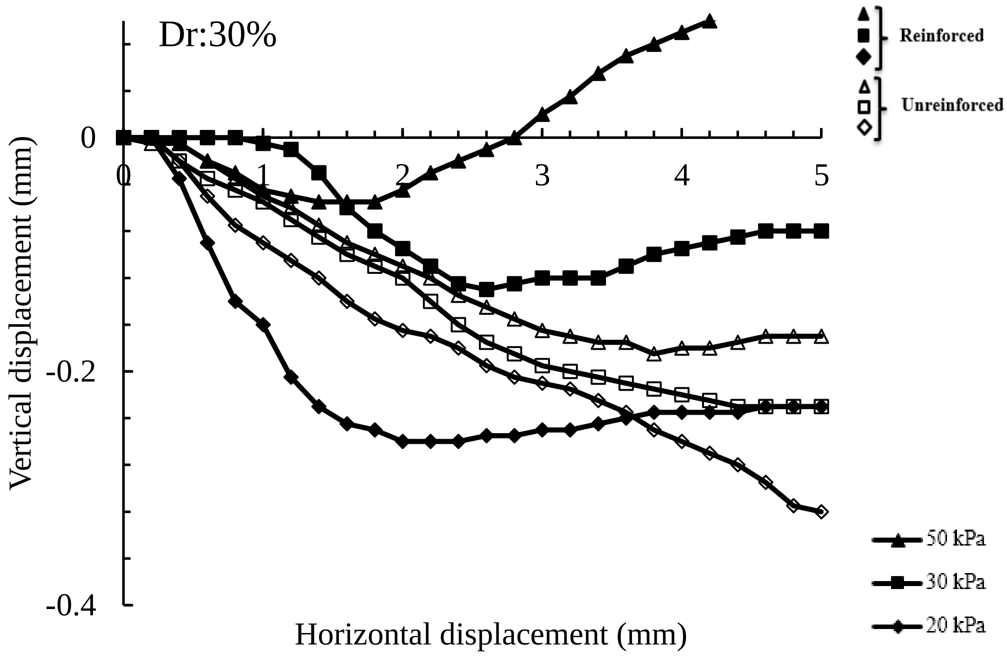

Figure 4.24. Vertical displacement - horizontal displacement graph for natural sand and sand with 1% fibreglass at 30% relative density

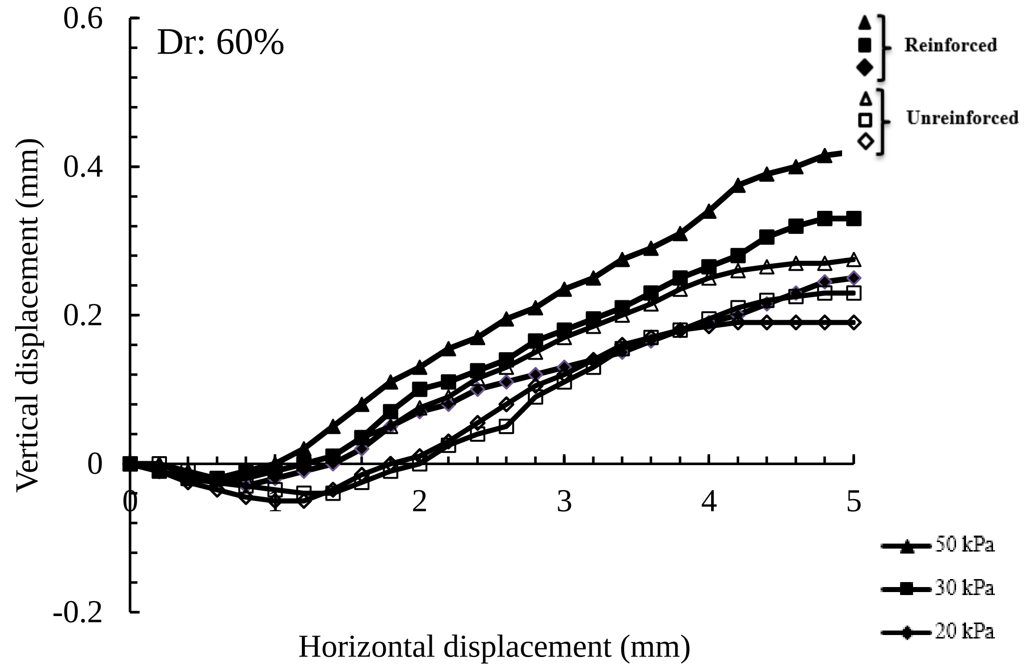

Figure 4.25. Vertical displacement - horizontal displacement graph for natural sand and sand with 1% fibreglass at 60% relative density

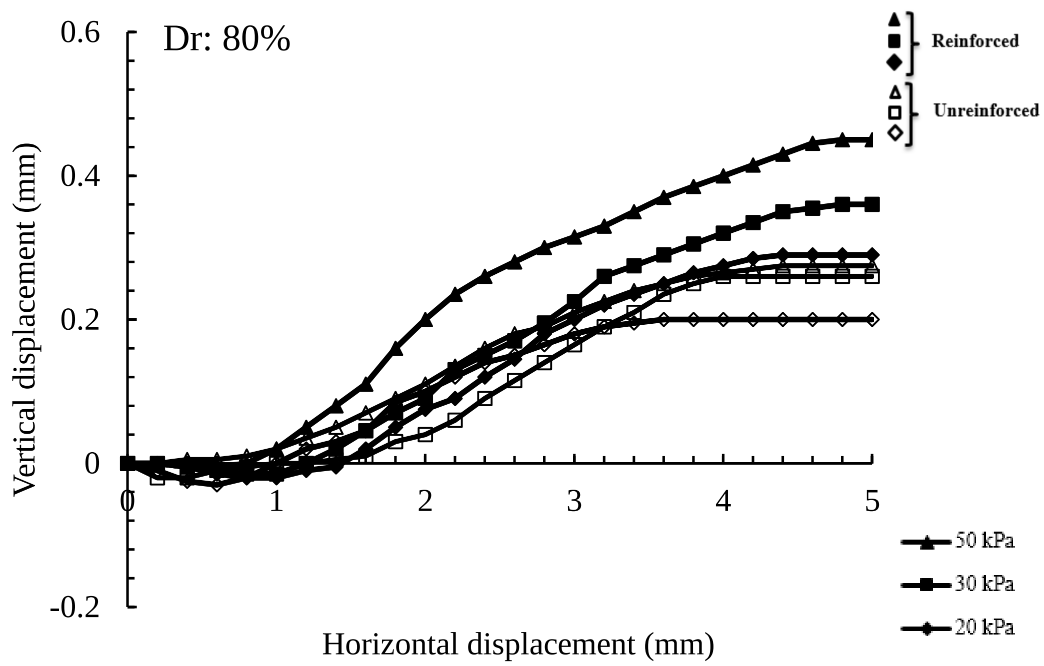

Figure 4.26. Vertical displacement - horizontal displacement graph for natural sand and sand reinforced with 1% fibreglass at 80% relative density

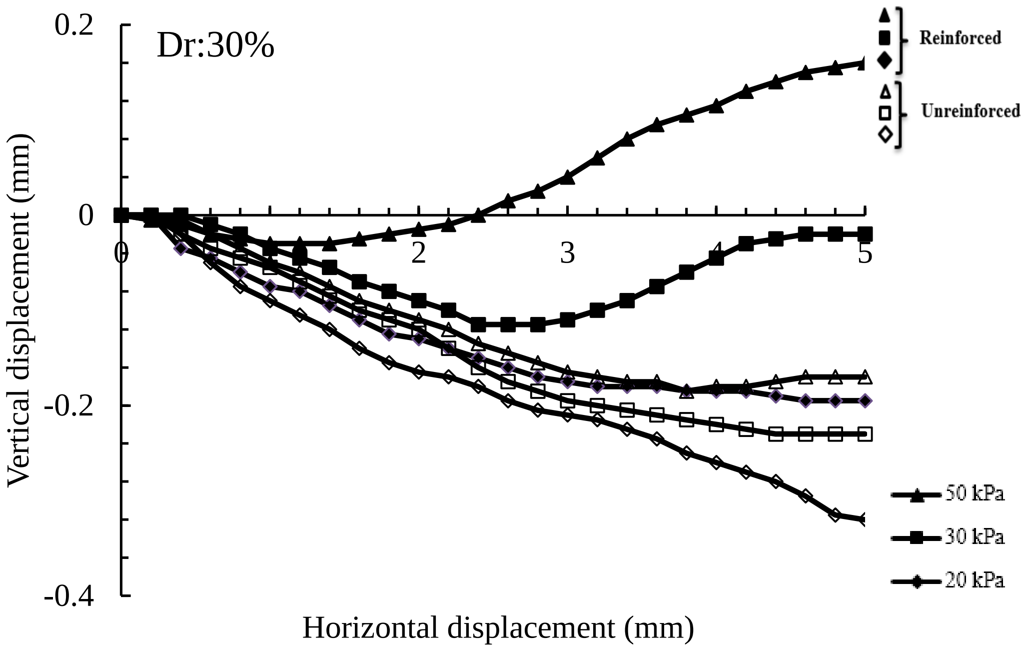

Figure 4.27. Vertical displacement - horizontal displacement graph for natural sand and sand with 1.5% fibreglass at 30% relative density

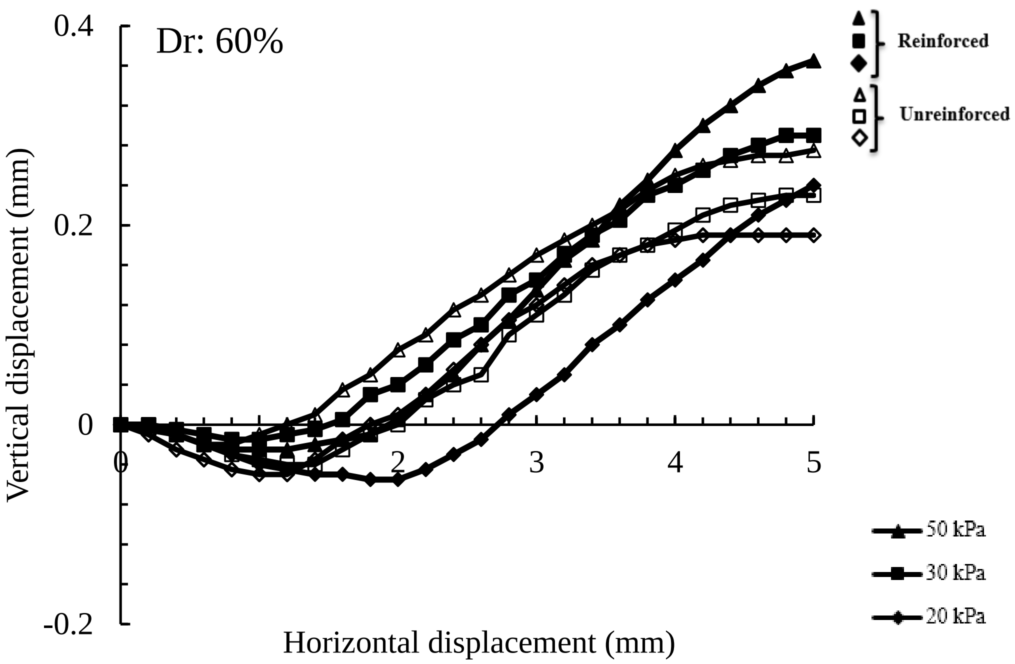

Figure 4.28. Vertical displacement -horizontal displacement graph for natural sand and sand with 1.5% fibreglass at 60% relative density

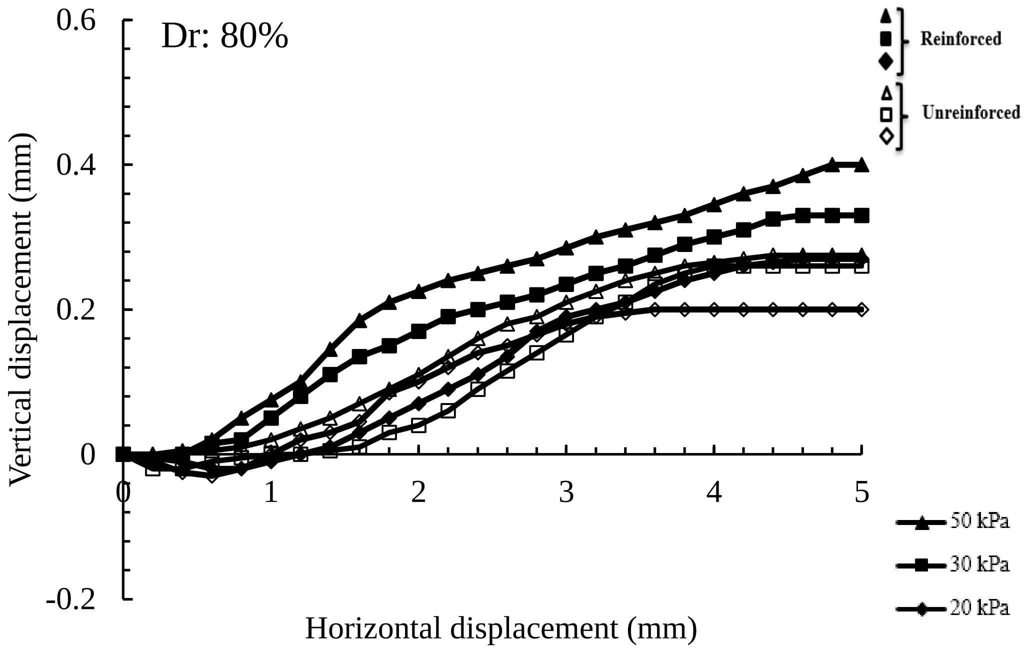

Figure 4.29. Vertical displacement -shear displacement graph for natural sand and sand with 1.5% fibreglass at 80% relative density

All the reinforced and unreinforced samples prepared above 30% relative density reflected a dilative character increasing in volume as shear displacement occurs. In general, all samples in the loose state reflected a contractive behavior. At the same relative density, all the samples illustrate that, with increase in normal stress and fibreglass reinforcement, dilation also increases.

The results show that, dilation usually starts after a small initial compression, which is due to the presence of some void spaces between the particles. Dense samples dilated at lower horizontal displacement than medium dense samples. The increase in shear strength with density is characterized by an increased tendency of the sample to dilate, and the work done in overcoming frictional forces. As the figures show, the peak shear strength increases with increase in relative density.

4.2.5 Shear stress versus normal stress

Figures 4.30 to 4.32 show the shear stress versus normal stress plots of the natural and reinforced sand with different relative density values for peak friction angle. Whereas Figures 4.33 to 4.35 indicate shear stress versus normal stress plots of unreinforced and reinforced sand for critical friction angle.

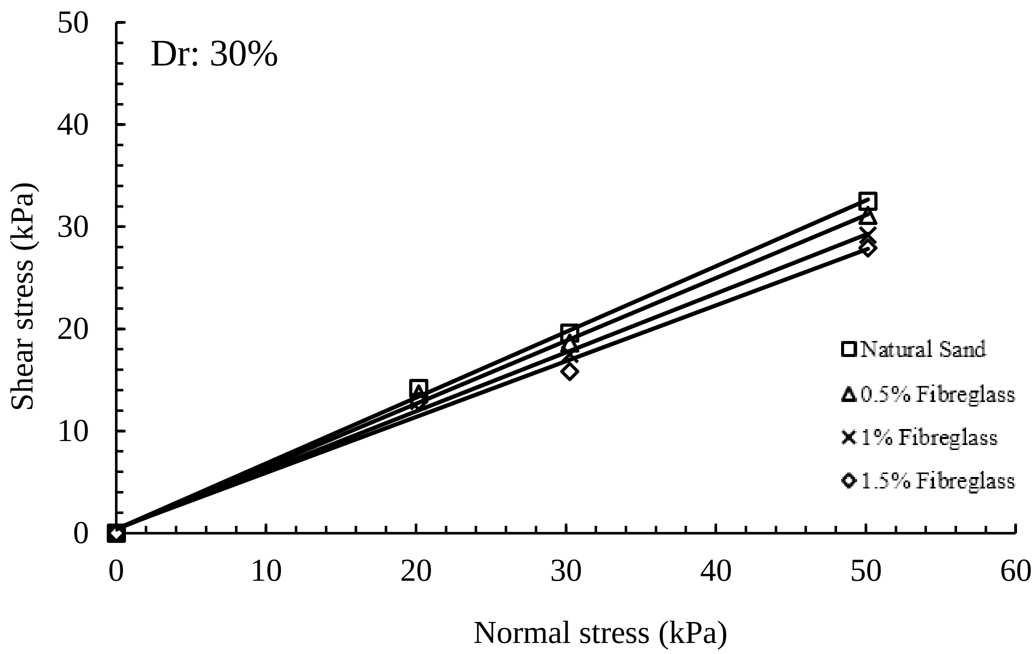

Figure 4.30. Shear stress - normal stress plots for peak friction angle for reinforced and unreinforced sand at loose state

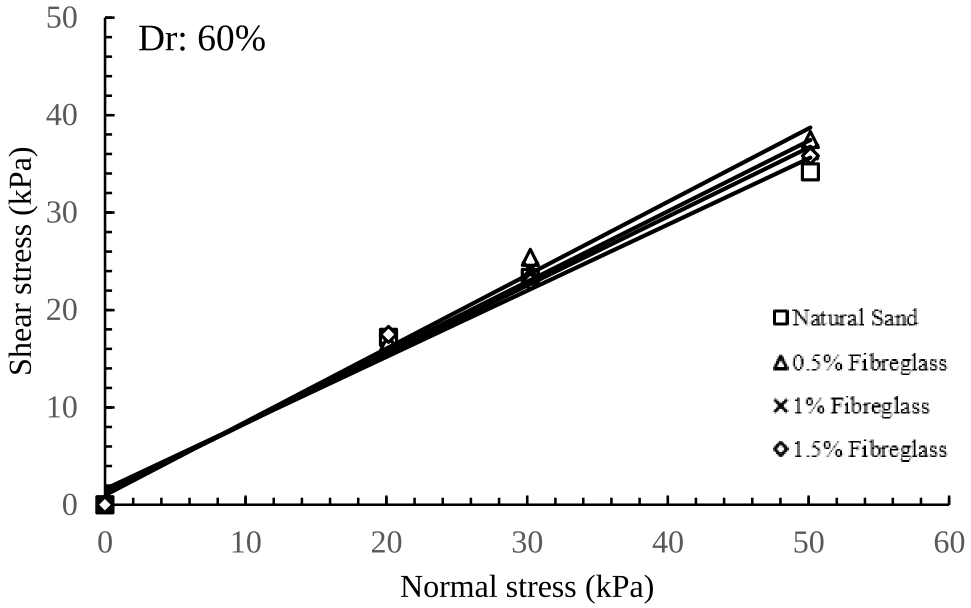

Figure 4.31. Shear stress - normal stress plots for peak friction angle for reinforced and unreinforced sand at medium state

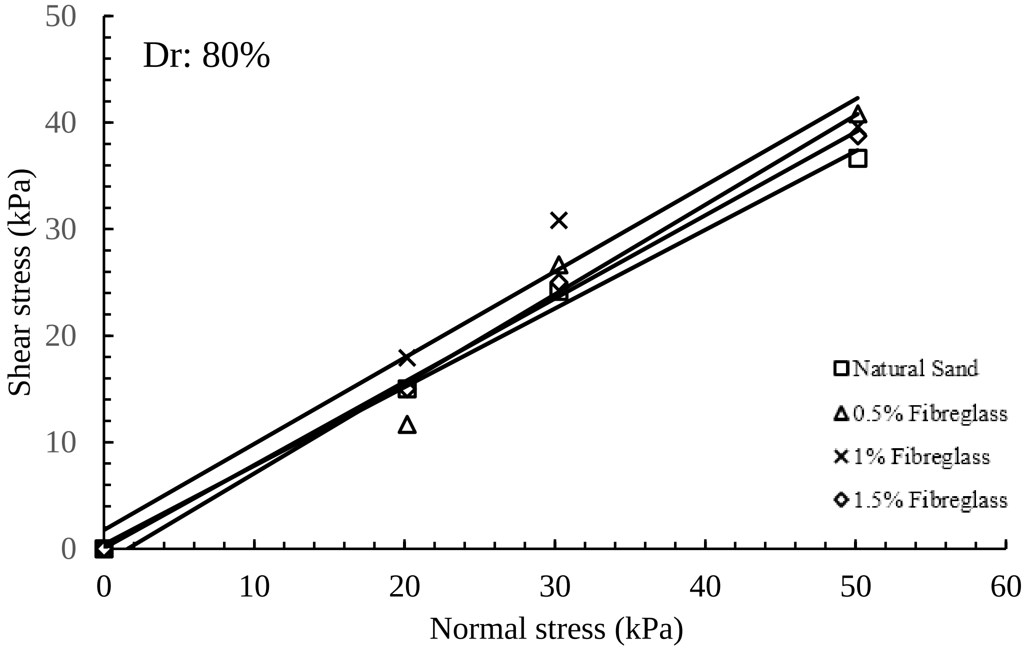

Figure 4.32. Shear stress - normal stress plots for peak friction angle for reinforced and unreinforced sand at dense state

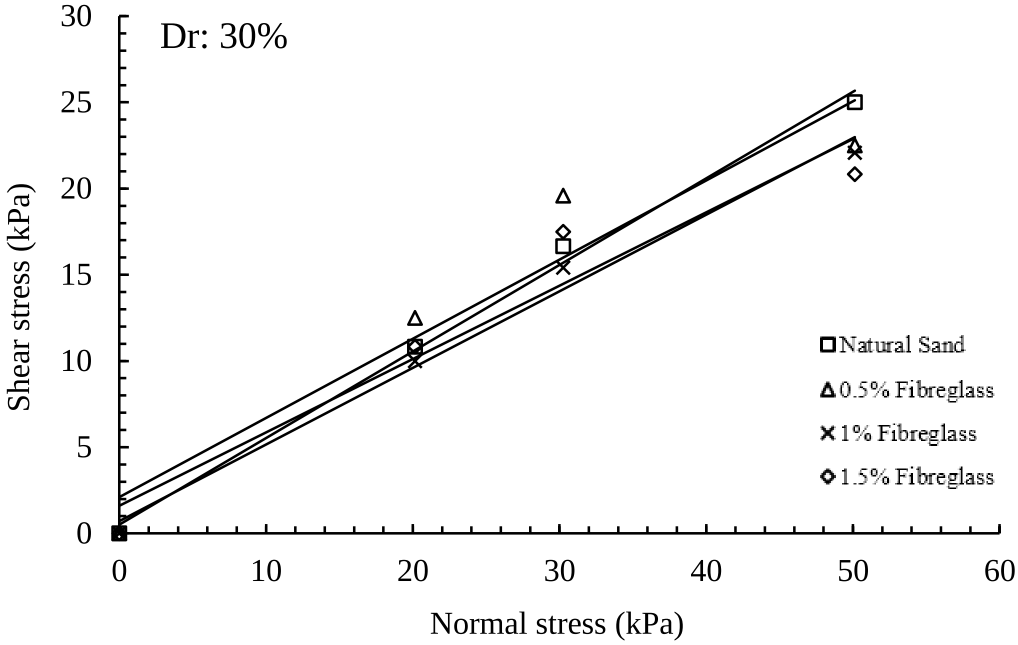

Figure 4.33. Shear stress - normal stress plots for critical friction angle for reinforced and unreinforced sand at dense state

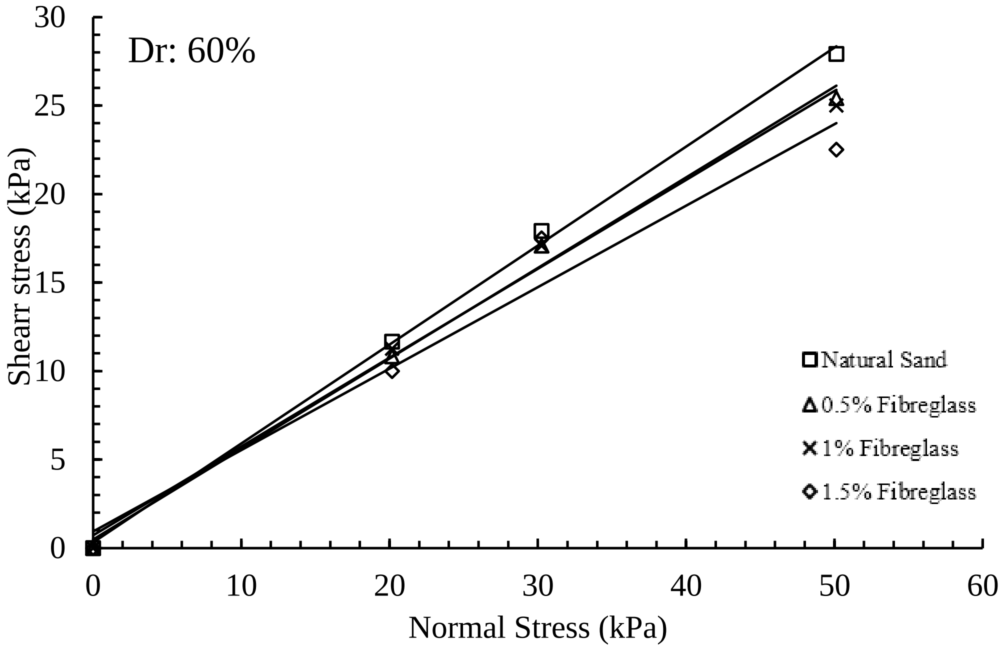

Figure 4.34. Shear stress - normal stress plots for critical friction angle for reinforced and unreinforced sand at dense state

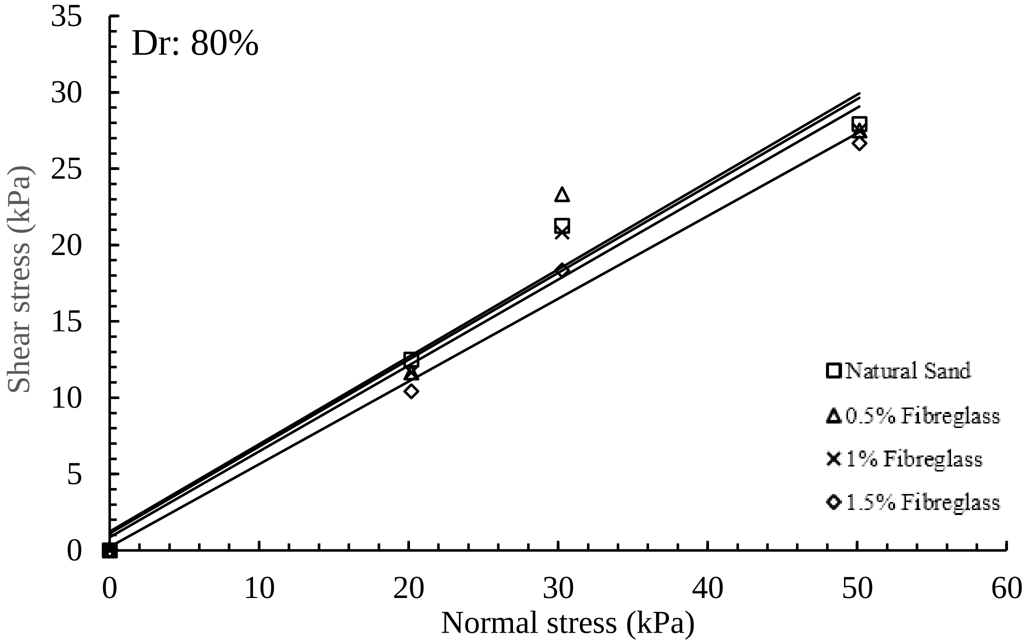

Figure 4.35. Shear stress - normal stress plots for critical friction angle for reinforced and unreinforced sand at dense state

Figures 4.30 to 4.32 and Figures 4.33 to 4.35 were used to determine the peak friction angles and the critical friction angles for the sand, respectively.

4.2.6 Friction Angle

The values of the peak friction angle and critical friction angle for both reinforced and unreinforced sand are presented in Tables 4.1 and 4.2 respectively.

The peak friction angle, ϕ for the natural sand in dense state (Dr= 80%) was found to be 36°. The peak friction angle, ϕ for the reinforced sand with 0.5% fibreglass content in dense state (Dr=80%) was found to be ϕ= 40°. The results are also plotted in Figure 4.36 and 4.37, which show the variation of peak friction angle and critical friction angle with increasing fibre content and relative density.

Table 4.1. Peak friction angle of sand

Fibre Content%	30%	60%	80%
0 (natural sand)	32.61	33.82	36.12
0.5	31.38	36.86	40.03
1	29.86	35.37	38.65
1.5	27.9	34.99	36.5

The results in Table 4.1 indicate that the maximum value of the peak friction angle is obtained for the sand reinforced with 0.5% of fibreglass at 80% relative density. Fibreglass reinforcement above 0.5% caused a reduction in the peak friction angle. Increase in the fibreglass content and relative density caused a further reduction the peak friction angle, because of the fact that there were no more space for the particles to move into, at higher relative density values and high percentage of fibreglass reduce the friction between the particles and reduction in the peak friction angle of sand was obtained. Therefore, the optimum fibre content is obtained as 0.5%.

Table 4.2. Critical friction angle of sand

Fibre content%	Dr: 30%	Dr: 60%	Dr:80%
0	26.56	28.81	29.24
0.5	24.65	27.02	29.68
1	23.74	26.56	29.24
1.5	22.78	24.70	28.36

The results in Table 4.2 indicate the critical friction angle, ϕ_c for the natural sand in dense state (Dr=80%) was found to be 29.24° and for the critical friction angle, ϕ_c for the reinforced sand with 0.5% fibreglass content in dense state (Dr=80%) was found to be almost the same as natural sand.

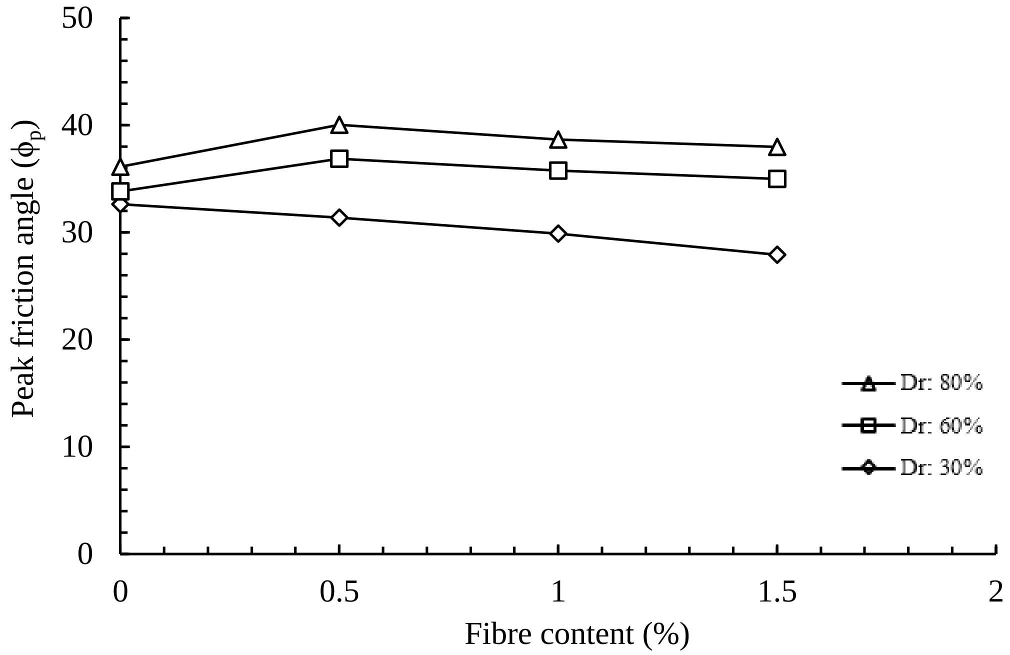

Figure 4.36. Peak friction angle -fibre content

Figure 4.36 and 4.37 shows the summary of the effect of fibreglass content on the peak and the critical friction angle of sand.

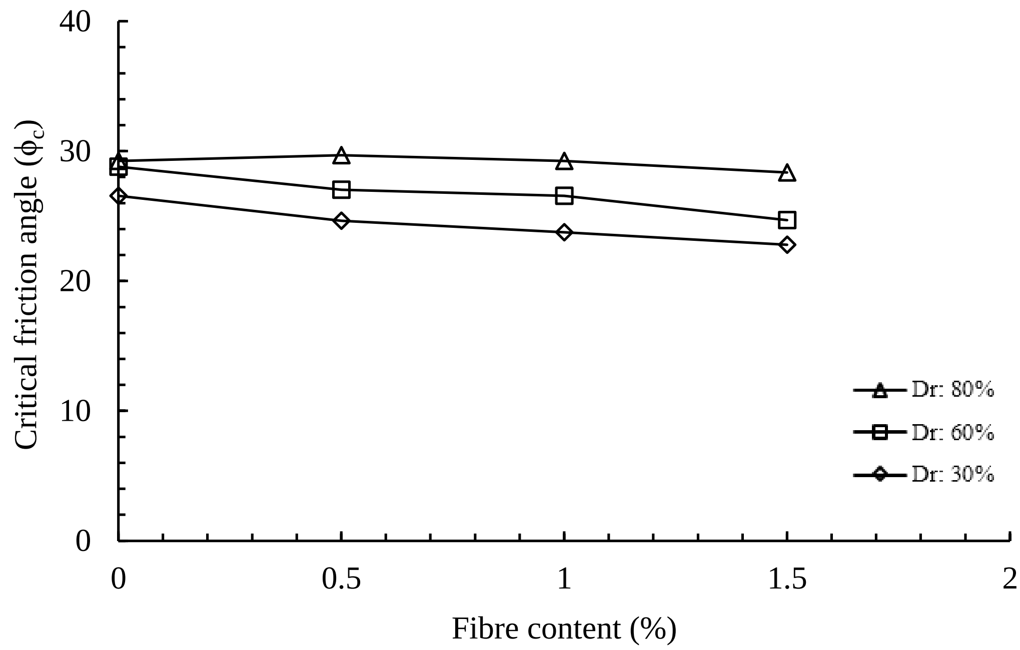

Figure 4.37. Critical friction angle –fibre content

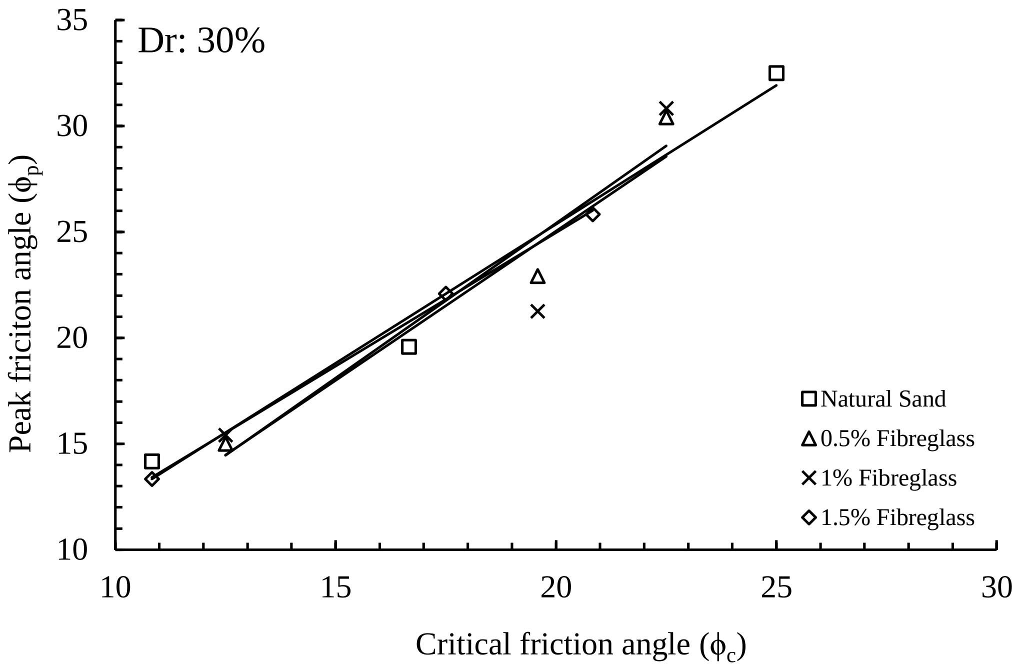

Figure 4.38. Peak friction angle- critical friction angle at 30% relative density

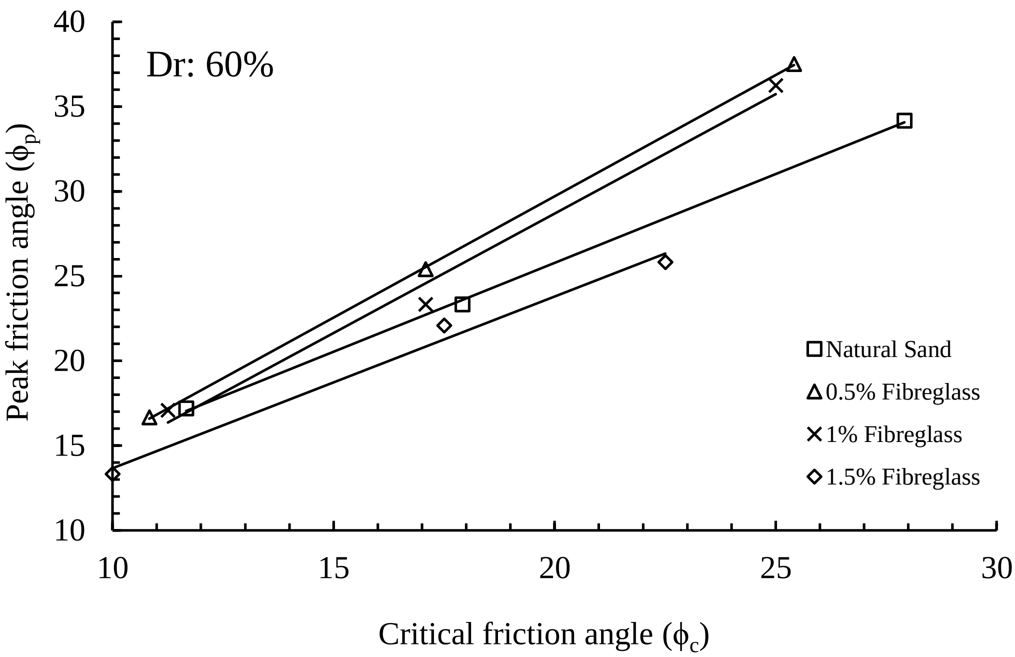

Figure 4.39. Peak friction angle- critical friction angle at 60% relative density

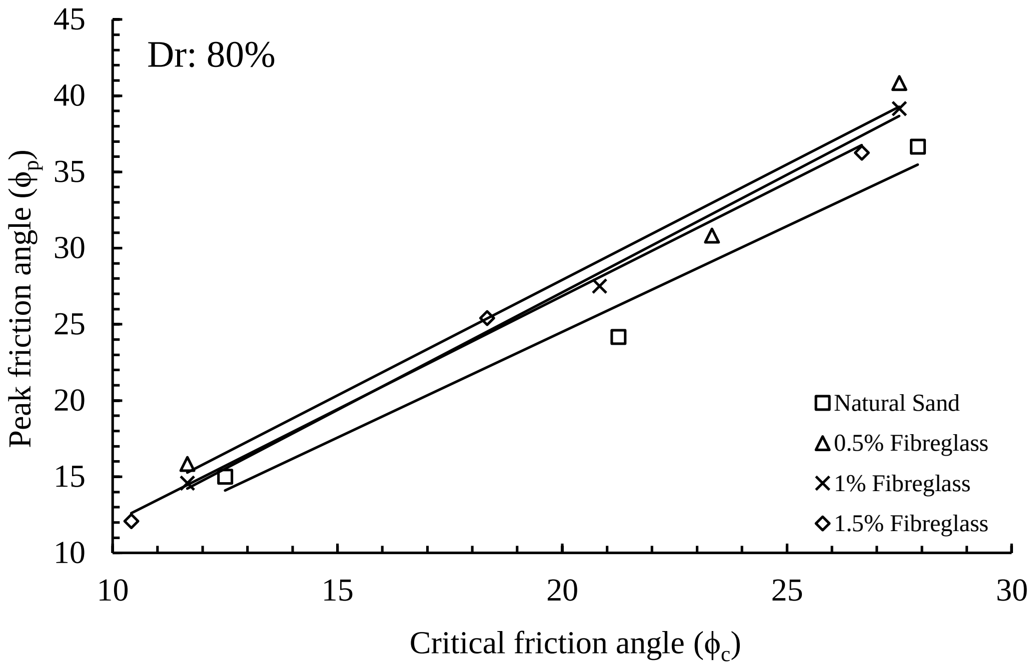

Figure 4.40. Peak friction angle- critical friction angle at 80% relative density

Figures 4.38 to 4.40 indicate the correlation between the peak friction angle and the critical friction angle. As it can be seen from the figures at 30% relative density, there is not much difference between the friction angles. As relative density increases, the difference between the natural and the reinforced soils friction angle increases slightly.

4.2.7 Dilation angle

The rate of dilation can be demonstrated by the gradient dv/dh, (where dv is the change in the height of specimen and dh is the shear displacement) the maximum rate corresponding to the peak stress. The test data on dense as well as loose sand are used to study the rate of dilation. The calculated values of dilation rates against horizontal displacement values are given in Figures 4.41 to 4.43.

The peak of dilation angle occurs at the same time with the peak value of the friction angle. Dilative soils become looser (their void ratio increases), and their rate of dilation decreases until they reach a critical void ratio. Contractive soils become denser as they shear, and their rate of contraction decreases until they reach a critical void ratio.

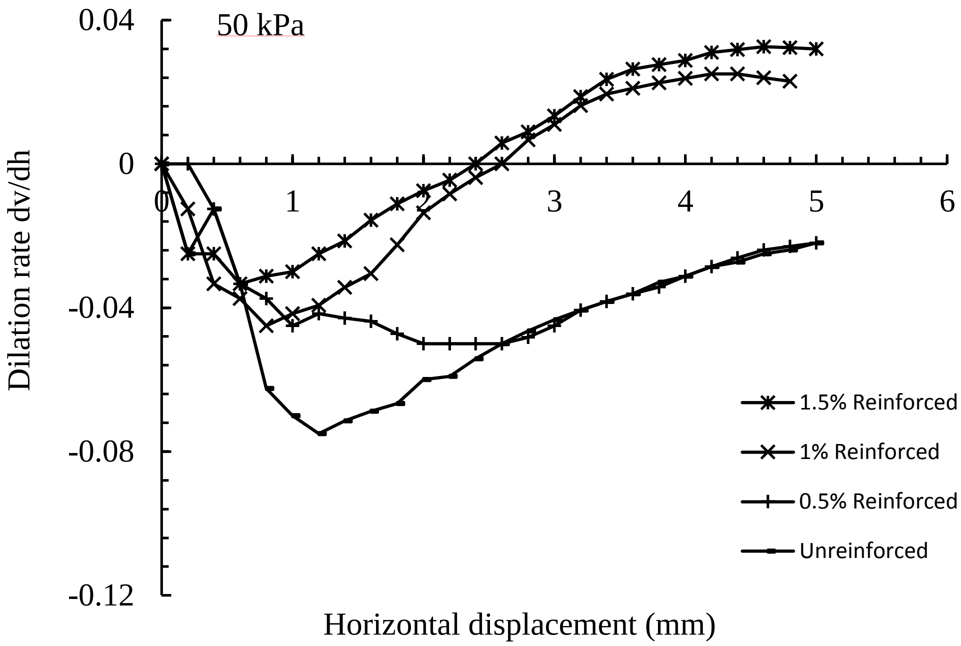

Figure 4.41. Dilation rate- horizontal displacement in dense state

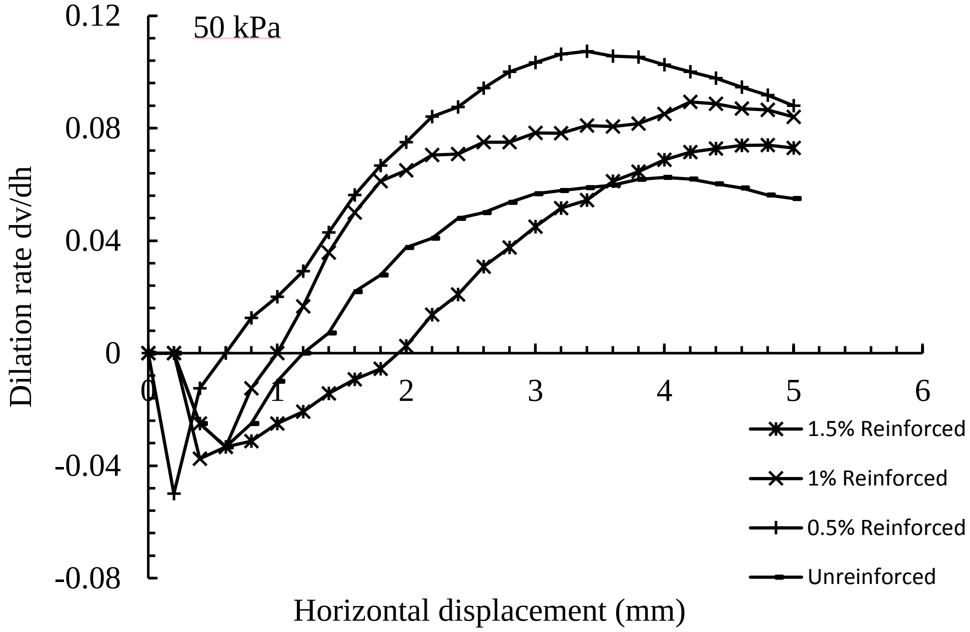

Figure 4.42. Dilation rate- horizontal displacement in medium state

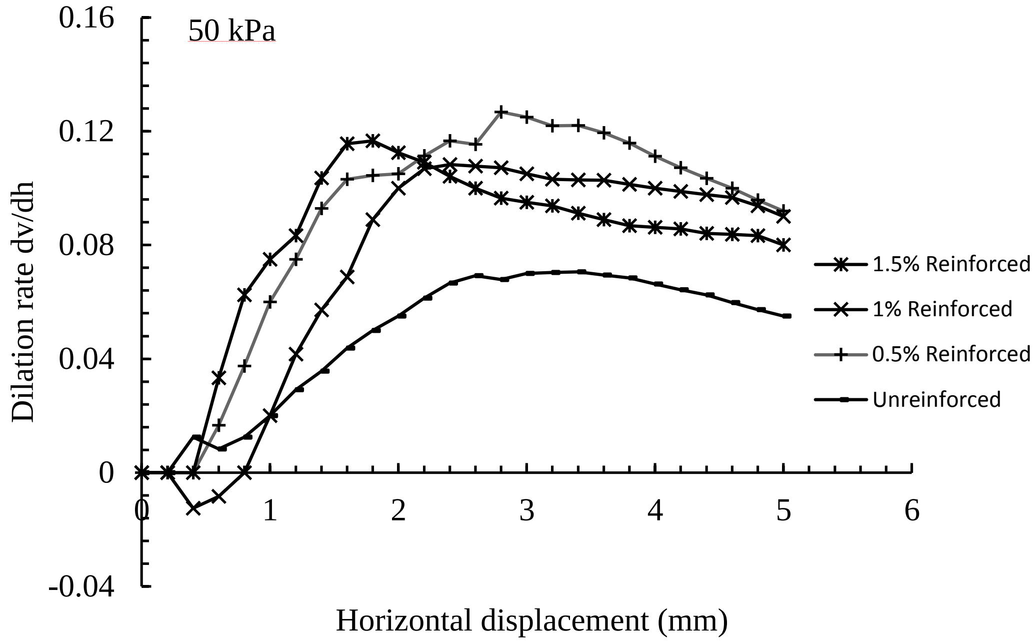

Figure 4.43. Dilation rate- horizontal displacement in dense state

The angle of dilation is the relationship between shear strain rate and volume strain rate. Normally, the angle of dilation increases with increasing relative density. Table 4.3 shows the maximum dilation angle for natural and reinforced sand at different percentages of relative density values. The values in Table 4.3 indicate that in the loose state, natural sand and sand with 0.5% fibre reinforcement have no dilation.

Table 4.3. Maximum dilation angle of natural and reinforced sand at different relative densities

Fibre content%	Dr: 30%	Dr: 60%	Dr: 80%
0	-	3.54	4
0.5	-	6.10	7.18
1	1.43	5.08	6.16
1.5	1.83	4.17	6.61

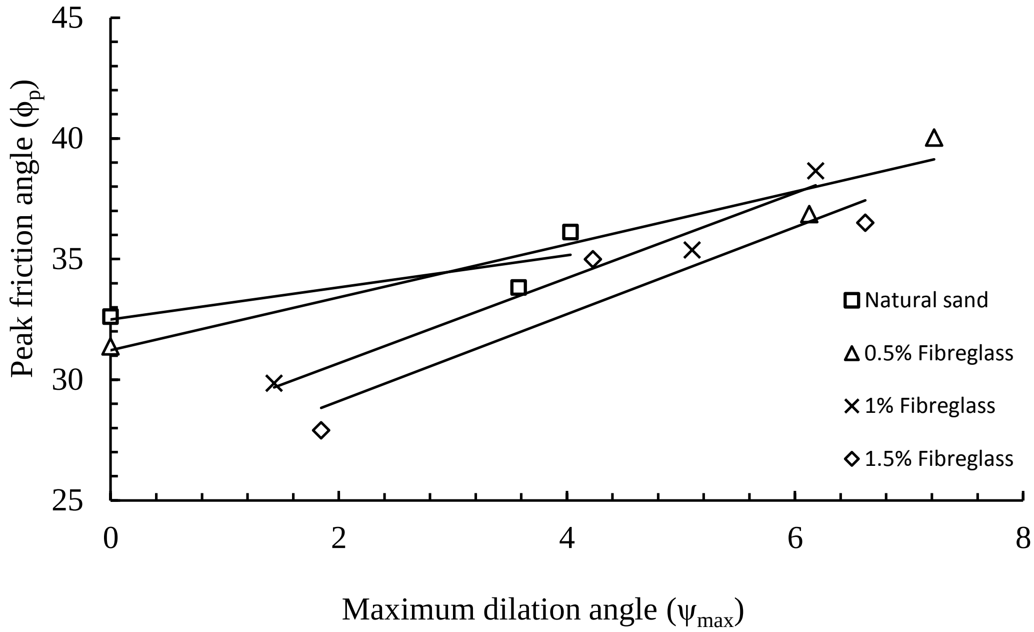

Figure 4.44: Peak friction angle versus maximum dilation angle at different relative density and fibreglass content

Figure 4.44 shows relationship between the peak friction angles with dilation angle. It can be seen that with increasing friction angle, the dilation angle increase. Furthermore, increase in the relative density increases the peak friction angle and consequently an increase in the dilation angle is obtained.

4.3 California Bearing Ration Test (CBR)

In the present study, California Bearing Ratio tests were conducted on reinforced and unreinforced sand specimens in order to see the suitability of fibreglass reinforced sand for highway and foundation construction. The reinforced sand specimens had fibre content 0.5% of the dry unit weight of the sand. The tests were conducted at three different relative densities as discussed in previous chapters: Loose, Medium and Dense state.

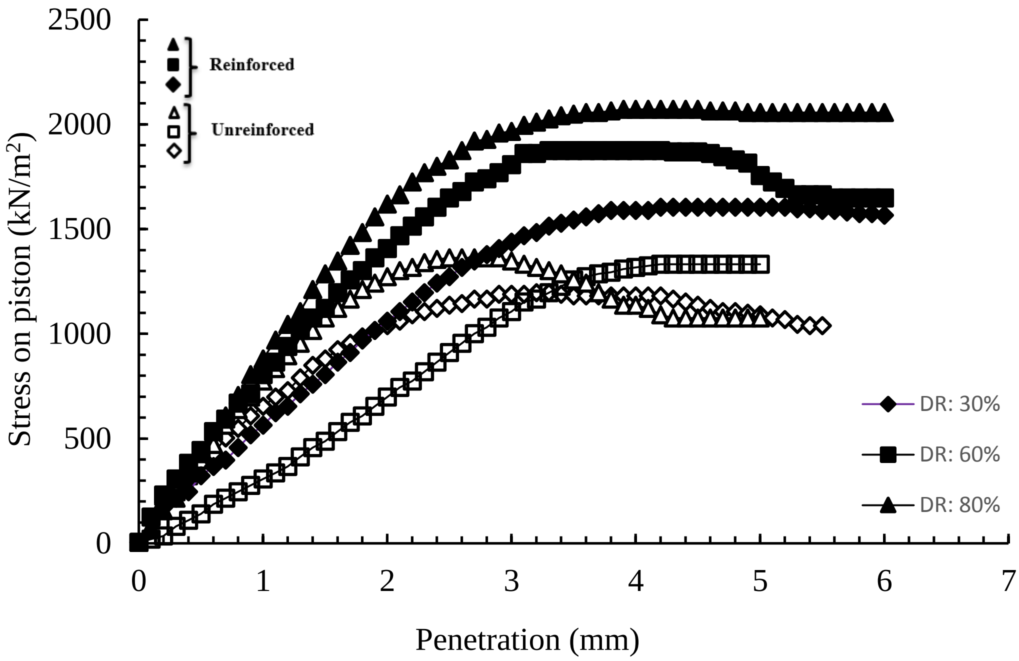

Figure 4.45. Penetration versus stress on piston values of reinforced sand and unreinforced sand

Figure 4.45 shows the results obtained from the CBR test. According to Figure 4.45, the fibreglass reinforcement of 0.5% improves the penetration resistance of the reinforced sands. The CBR values increased significantly with the addition of reinforcing fibres. In all relative density values, addition of 0.5% fibreglass significantly increased the CBR value compared to unreinforced specimens. The CBR values obtained in accordance with ASTM 698 for natural and reinforced sand are given in Figure 4.46 and 4.47.

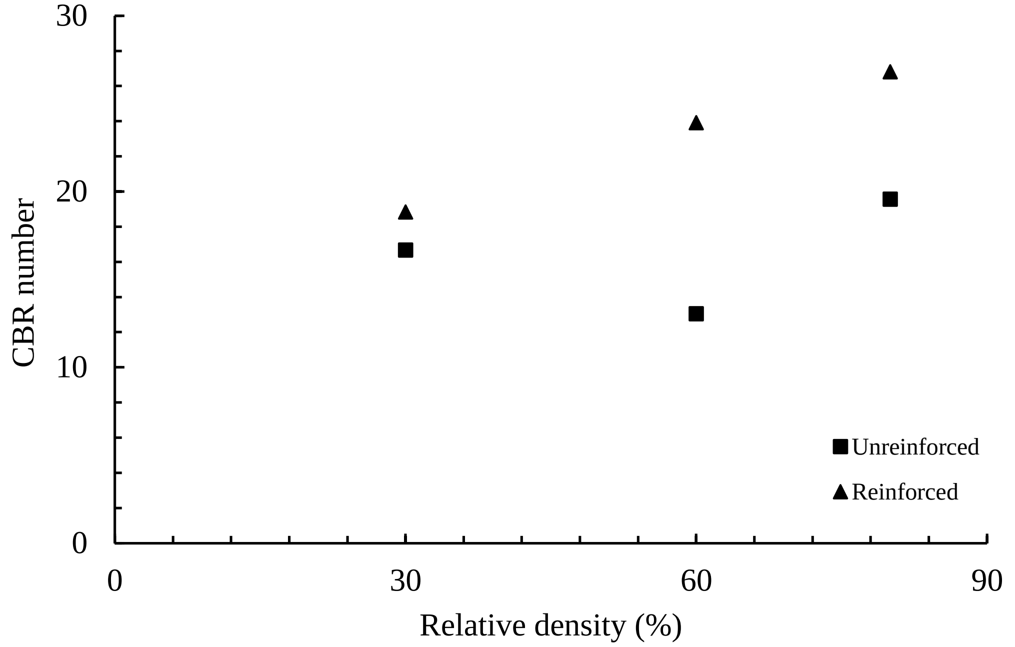

Figure 4.46. CBR value at 2.54mm, fibreglass reinforcement 0.5%

The results show that, generally the CBR values increases with increasing fibre reinforcement percent and relative density. However, in loose state, there is no significant increase in the CBR value.

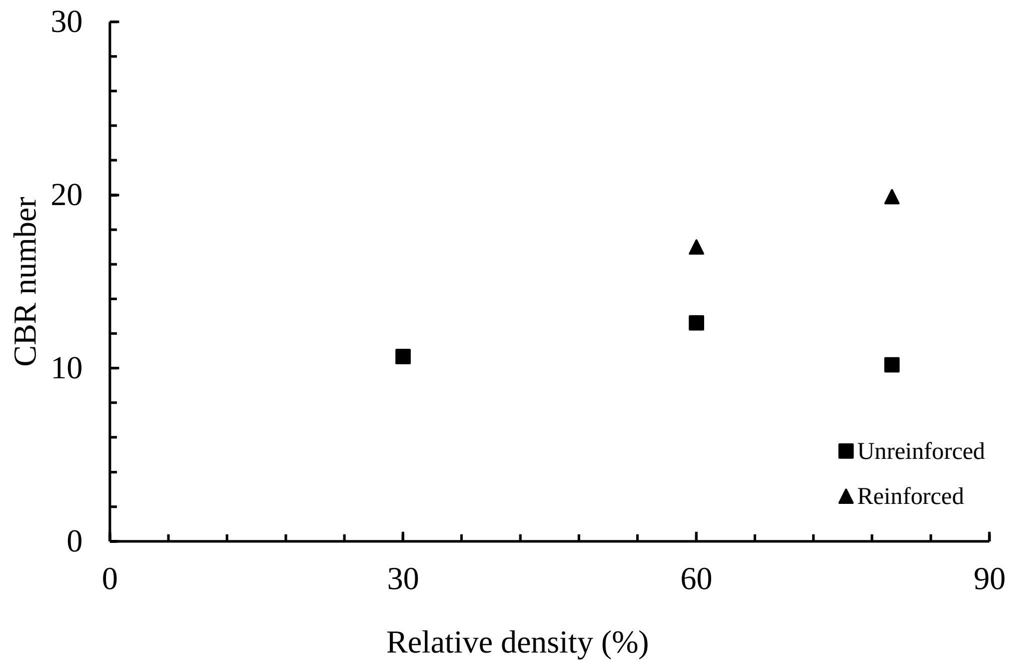

Figure 4.47. CBR value at 5.08 mm, fibreglass reinforcement 0.5%

Since the CBR value obtained at 5.08 mm penetration, where less than the values obtained at 2.54 mm penetration, the CBR value at 2.54 mm were used (ASTM 698). It can be seen from the results that the CBR value obtained for the reinforced sand at various relative density values is in the range of 10 to 20. According to U.S. Army (1960), sand with CBR value between10-40 has excellent drainage characteristics as a subgrade material.

Chapter 5

CONCLUSION AND RECOMMENDATION FOR FURTHER RESEARCH

5.1 Summary of Conclusions

The investigation of the effect of fibreglass on the shear strength of sand was the main purpose of this research. Different percentages of fibreglass were applied to Palm Beach Sand prepared at different relative densities and the results were discussed. Laboratory test results demonstrated that engineering properties of sand changed with fibreglass reinforcement. Test results indicated that fibreglass has a positive effect on the shear strength parameters of sand. Based on the laboratory study in this work, the following conclusions can be made:

- Increase in the relative density increased the shear strength of the sand and resulted in dilation. The sand in both medium and dense states showed dilation behavior.
- The initial slope of the sand reinforced with fibreglass, increased with an increase in relative density.
- Test results indicated that fibreglass reinforcement did not cause significant changes in the shear strength parameters of sand.
- Fibreglass more than 0.5% did not improve the shear strength of sand. Peak friction angle decreased with increasing fibreglass content. The maximum

improvement in shear strength was obtained for the sand at the densest state with 0.5% reinforcement at the normal stress value of 50 kPa.

- Dense sand dilated faster than medium dense sands. The angle of dilation tends to increase with increasing relative density.
- The CBR value increased with 0.5% fibreglass reinforcement. 0.5% fibreglass reinforcement improved the penetration resistance of the reinforced sands. The CBR values obtained for the fibreglass reinforced sands were between 10-40. These values indicated that the sand has an excellent drainage characteristic as a subgrade material.

5.2 Suggestions for Further Research

The aim of this study is evaluate the effect of fibreglass on the shear strength of sand with different relative density, and different fibreglass content. The suggestions for further research are as follows:

- The effect of fibreglass content on shear strength of clays should also be investigated.
- Other materials such as polymer or cement together with fibreglass can be studied and the obtained results can be compared with results obtained in this study.
- Different sizes of fibreglass can be tested.

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